DESIGN EXAMPLES

Companion to the AISC *Steel Construction Manual*

Version 15.0

AMERICAN INSTITUTE
OF
STEEL CONSTRUCTION

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PREFACE

The primary objective of this Companion is to provide guidance and additional resources of the use of the 2016 AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360-16) and the 15th Edition AISC *Steel Construction Manual*.

The Companion consists of design examples in Parts I, II and III, and design tables in Part IV. The design examples provide coverage of all applicable limit states, whether or not a particular limit state controls the design of the member or connection. In addition to the examples that demonstrate the use of the AISC *Manual* tables, design examples are provided for connection designs beyond the scope of the tables in the AISC *Manual*. These design examples are intended to demonstrate an approach to the design, and are not intended to suggest that the approach presented is the only approach. The committee responsible for the development of these design examples recognizes that designers have alternate approaches that work best for them and their projects. Design approaches that differ from those presented in these examples are considered viable as long as the AISC *Specification*, sound engineering, and project specific requirements are satisfied.

Part I of these examples is organized to correspond with the organization of the AISC *Specification*. The Chapter titles match the corresponding chapters in the AISC *Specification*.

Part II is devoted primarily to connection examples that draw on the tables from the AISC *Manual*, Part IV of this publication, recommended design procedures, and the breadth of the AISC *Specification*. The chapters of Part II are labeled II-A, II-B, II-C, etc.

Part III addresses aspects of design that are linked to the performance of a building as a whole. This includes coverage of lateral stability and second-order analysis, illustrated through a four-story braced-frame and moment-frame building.

Part IV provides additional design tables beyond what is incorporated into the AISC *Manual*.

The Design Examples are arranged with LRFD and ASD designs presented side-by-side, for consistency with the AISC *Manual*. Design with ASD and LRFD are based on the same nominal strength for each element so that the only differences between the approaches are the set of load combinations from ASCE/SEI 7-16 used for design, and whether the resistance factor for LRFD or the safety factor for ASD is used.

CONVENTIONS

The following conventions are used throughout these examples:

1. The 2016 AISC *Specification for Structural Steel Buildings* is referred to as the AISC *Specification* and the 15th Edition AISC *Steel Construction Manual,* is referred to as the AISC *Manual*.

2. The 2016 ASCE *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* is referred to as ASCE/SEI 7.

3. The source of equations or tabulated values taken from the AISC *Specification* or AISC *Manual* is noted along the right-hand edge of the page.

4. When the design process differs between LRFD and ASD, the designs equations are presented side-by-side. This rarely occurs, except when the resistance factor, ϕ, and the safety factor, Ω, are applied.

5. The results of design equations are presented to three significant figures throughout these calculations.

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Part I
Examples Based on the AISC *Specification*

This part contains design examples demonstrating select provisions of the AISC *Specification for Structural Steel Buildings*.

Chapter A
General Provisions

A1. SCOPE

These design examples are intended to illustrate the application of the 2016 AISC *Specification for Structural Steel Buildings*, ANSI/AISC 360-16 (AISC, 2016a), and the AISC *Steel Construction Manual*, 15th Edition (AISC, 2017) in low-seismic applications. For information on design applications requiring seismic detailing, see the 2016 AISC *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341-16 (AISC, 2016b) and the AISC *Seismic Design Manual*, 2nd Edition (AISC, 2012).

A2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

Section A2 includes a detailed list of the specifications, codes and standards referenced throughout the AISC *Specification*.

A3. MATERIAL

Section A3 includes a list of the steel materials that are approved for use with the AISC *Specification*. The complete ASTM standards for the most commonly used steel materials can be found in *Selected ASTM Standards for Structural Steel Fabrication* (ASTM, 2016).

A4. STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS

Section A4 requires that structural design drawings and specifications meet the requirements in the AISC *Code of Standard Practice for Steel Buildings and Bridges*, ANSI/AISC 303-16 (AISC, 2016c).

CHAPTER A REFERENCES

AISC (2012), *Seismic Design Manual*, 2nd Ed., American Institute of Steel Construction, Chicago, IL.

AISC (2016a), *Specification for Structural Steel Buildings*, ANSI/AISC 360-16, American Institute of Steel Construction, Chicago, IL.

AISC (2016b), *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341-16, American Institute of Steel Construction, Chicago, IL.

AISC (2016c), *Code of Standard Practice for Steel Buildings and Bridges*, ANSI/AISC 303-16, American Institute of Steel Construction, Chicago, IL.

AISC (2017), *Steel Construction Manual*, 15th Ed., American Institute of Steel Construction, Chicago, IL.

ASTM (2016), *Selected ASTM Standards for Structural Steel Fabrication*, ASTM International, West Conshohocken, PA.

Chapter B
Design Requirements

B1. GENERAL PROVISIONS

The AISC *Specification* requires that the design of members and connections shall be consistent with the intended behavior of the framing system and the assumptions made in the structural analysis.

B2. LOADS AND LOAD COMBINATIONS

In the absence of an applicable building code, the default load combinations to be used with the AISC *Specification* are those from *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE/SEI 7-16 (ASCE, 2016).

B3. DESIGN BASIS

Chapter B of the AISC *Specification* and Part 2 of the AISC *Manual* describe the basis of design, for both load and resistance factor design (LRFD) and allowable strength design (ASD).

AISC *Specification* Section B3.4 describes three basic types of connections: simple connections, fully restrained (FR) moment connections, and partially restrained (PR) moment connections. Several examples of the design of each of these types of connections are given in Part II of these *Design Examples*.

Information on the application of serviceability and ponding provisions may be found in AISC *Specification* Chapter L and AISC *Specification* Appendix 2, respectively, and their associated commentaries. Design examples and other useful information on this topic are given in AISC Design Guide 3, *Serviceability Design Considerations for Steel Buildings*, Second Edition (West et al., 2003).

Information on the application of fire design provisions may be found in AISC *Specification* Appendix 4 and its associated commentary. Design examples and other useful information on this topic are presented in AISC Design Guide 19, *Fire Resistance of Structural Steel Framing* (Ruddy et al., 2003).

Corrosion protection and fastener compatibility are discussed in Part 2 of the AISC *Manual*.

B4. MEMBER PROPERTIES

AISC *Specification* Tables B4.1a and B4.1b give the complete list of limiting width-to-thickness ratios for all compression and flexural members defined by the AISC *Specification*.

Except for one section, the W-shapes presented in the compression member selection tables as column sections meet the criteria as nonslender element sections. The W-shapes with a nominal depth of 8 in. or larger presented in the flexural member selection tables as beam sections meet the criteria for compact sections, except for seven specific shapes. When noncompact or slender-element sections are tabulated in the design aids, local buckling criteria are accounted for in the tabulated design values.

The shapes listing and other member design tables in the AISC *Manual* also include footnoting to highlight sections that exceed local buckling limits in their most commonly available material grades. These footnotes include the following notations for W-shapes:

[c] Shape is slender for compression with F_y = 50 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 50 ksi.

CHAPTER B REFERENCES

ASCE (2016), *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE/SEI 7-16, American Society of Civil Engineers, Reston, VA.

West, M.A., Fisher, J.M. and Griffis, L.G. (2003), *Serviceability Design Considerations for Steel Buildings*, Design Guide 3, 2nd Ed., AISC, Chicago, IL.

Ruddy, J.L., Marlo, J.P., Ioannides, S.A. and Alfawakhiri, F. (2003), *Fire Resistance of Structural Steel Framing*, Design Guide 19, AISC, Chicago, IL.

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Chapter C
Design for Stability

C1. GENERAL STABILITY REQUIREMENTS

The AISC *Specification* requires that the designer account for both the stability of the structural system as a whole and the stability of individual elements. Thus, the lateral analysis used to assess stability must include consideration of the combined effect of gravity and lateral loads, as well as member inelasticity, out-of-plumbness, out-of-straightness, and the resulting second-order effects, P-Δ and P-δ. The effects of "leaning columns" must also be considered, as illustrated in the examples in this chapter and in the four-story building design example in Part III of these *Design Examples*.

P-Δ and P-δ effects are illustrated in AISC *Specification* Commentary Figure C-C2.1. Methods for addressing stability, including P-Δ and P-δ effects, are provided in AISC *Specification* Section C2 and Appendix 7.

C2. CALCULATION OF REQUIRED STRENGTHS

The calculation of required strengths is illustrated in the examples in this chapter and in the four-story building design example in Part III of these *Design Examples*.

C3. CALCULATION OF AVAILABLE STRENGTHS

The calculation of available strengths is illustrated in the four-story building design example in Part III of these *Design Examples*.

EXAMPLE C.1A DESIGN OF A MOMENT FRAME BY THE DIRECT ANALYSIS METHOD

Given:

Determine the required strengths and effective length factors for the columns in the moment frame shown in Figure C.1A-1 for the maximum gravity load combination, using LRFD and ASD. The uniform load, w_D, includes beam self-weight and an allowance for column self-weight. Use the direct analysis method. All members are ASTM A992 material.

Columns are unbraced between the footings and roof in the *x*- and *y*-axes and have pinned bases.

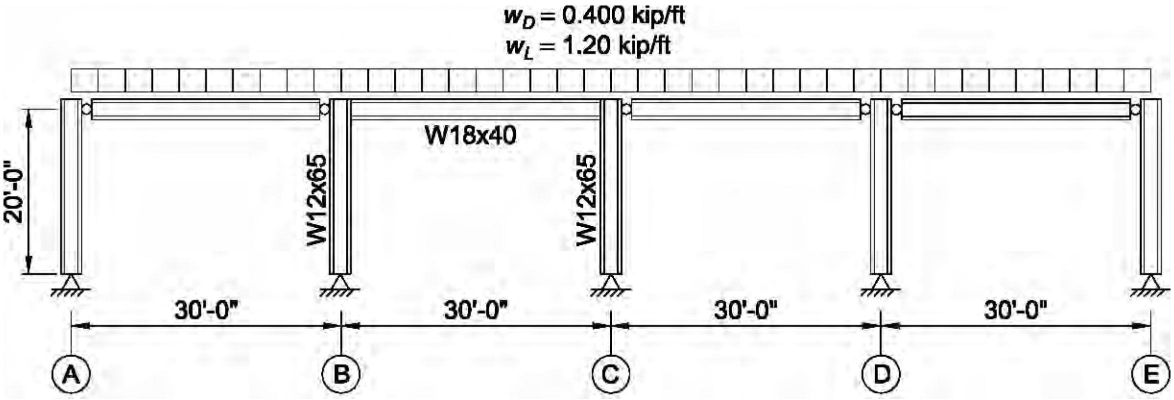


Fig. C.1A-1. Example C.1A moment frame elevation.

Solution:

From AISC *Manual* Table 1-1, the W12×65 has A = 19.1 in.^{2}

The beams from grid lines A to B and C to E and the columns at A, D and E are pinned at both ends and do not contribute to the lateral stability of the frame. There are no P-Δ effects to consider in these members and they may be designed using $L_c = L$.

The moment frame between grid lines B and C is the source of lateral stability and therefore will be evaluated using the provisions of Chapter C of the AISC *Specification*. Although the columns at grid lines A, D and E do not contribute to lateral stability, the forces required to stabilize them must be considered in the moment-frame analysis. The entire frame from grid line A to E could be modeled, but in this case the model is simplified as shown in Figure C.1A-2, in which the stability loads from the three "leaning" columns are combined into a single representative column.

From Chapter 2 of ASCE/SEI 7, the maximum gravity load combinations are:

LRFD	ASD
$w_u = 1.2D + 1.6L$ $= 1.2(0.400 \text{ kip/ft}) + 1.6(1.20 \text{ kip/ft})$ $= 2.40 \text{ kip/ft}$	$w_u = D + L$ $= 0.400 \text{ kip/ft} + 1.20 \text{ kip/ft}$ $= 1.60 \text{ kip/ft}$

Per AISC *Specification* Section C2.1(d), for LRFD, perform a second-order analysis and member strength checks using the LRFD load combinations. For ASD, perform a second-order analysis using 1.6 times the ASD load combinations and divide the analysis results by 1.6 for the ASD member strength checks.

Frame analysis gravity loads

The uniform gravity loads to be considered in a second-order analysis on the beam from B to C are:

LRFD	ASD
$w'_u = 2.40\text{ kip/ft}$	$w'_a = 1.6(1.60\text{ kip/ft})$ $= 2.56\text{ kip/ft}$

Concentrated gravity loads to be considered in a second-order analysis on the columns at B and C contributed by adjacent beams are:

LRFD	ASD
$P'_u = \dfrac{w'_u l}{2}$ $= \dfrac{(2.40\text{ kip/ft})(30.0\text{ ft})}{2}$ $= 36.0\text{ kips}$	$P'_a = \dfrac{w'_a l}{2}$ $= \dfrac{(2.56\text{ kip/ft})(30.0\text{ ft})}{2}$ $= 38.4\text{ kips}$

Concentrated gravity loads on the representative "leaning" column

The load in this column accounts for all gravity loading that is stabilized by the moment frame, but is not directly applied to it.

LRFD	ASD
$P'_{uL} = (60.0\text{ ft})(2.40\text{ kip/ft})$ $= 144\text{ kips}$	$P'_{aL} = (60.0\text{ ft})(2.56\text{ kip/ft})$ $= 154\text{ kips}$

Frame analysis notional loads

Per AISC *Specification* Section C2.2, frame out-of-plumbness must be accounted for either by explicit modeling of the assumed out-of-plumbness or by the application of notional loads. Use notional loads.

From AISC *Specification* Equation C2-1, the notional loads are:

LRFD	ASD
$\alpha = 1.0$ $Y_i = (120\text{ ft})(2.40\text{ kip/ft})$ $= 288\text{ kips}$ $N_i = 0.002\alpha Y_i$ (*Spec*. Eq. C2-1) $= 0.002(1.0)(288\text{ kips})$ $= 0.576\text{ kip}$	$\alpha = 1.6$ $Y_i = (120\text{ ft})(1.60\text{ kip/ft})$ $= 192\text{ kips}$ $N_i = 0.002\alpha Y_i$ (*Spec*. Eq. C2-1) $= 0.002(1.6)(192\text{ kips})$ $= 0.614\text{ kip}$

Summary of applied frame loads

The applied loads are shown in Figure C.1A-2.

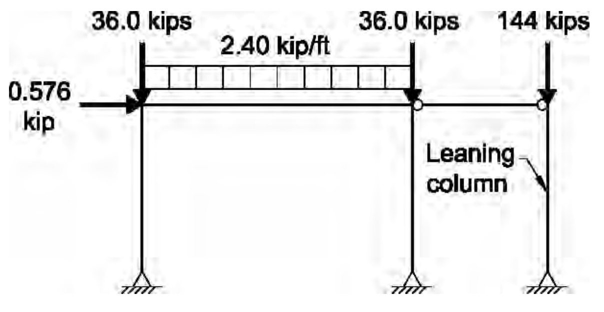
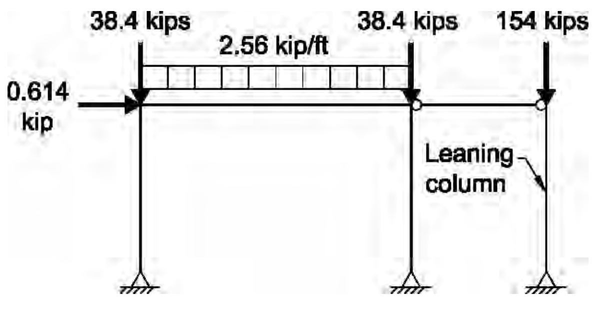
LRFD	ASD
	

Fig. C.1A-2. Applied loads on the analysis model.

Per AISC *Specification* Section C2.3, conduct the analysis using 80% of the nominal stiffnesses to account for the effects of inelasticity. Assume, subject to verification, that $\alpha P_r / P_{ns}$ is not greater than 0.5; therefore, no additional stiffness reduction is required (τ_b = 1.0).

Half of the gravity load is carried by the columns of the moment-resisting frame. Because the gravity load supported by the moment-resisting frame columns exceeds one-third of the total gravity load tributary to the frame, per AISC *Specification* Section C2.1, the effects of P-δ and P-Δ must be considered in the frame analysis. This example uses analysis software that accounts for both P-Δ and P-δ effects. (If the software used does not account for P-δ effects this may be accomplished by subdividing the columns between the footing and beam.)

Figures C.1A-3 and C.1A-4 show results from a first-order and a second-order analysis. (The first-order analysis is shown for reference only.) In each case, the drift is the average of drifts at grid lines B and C.

First-order results

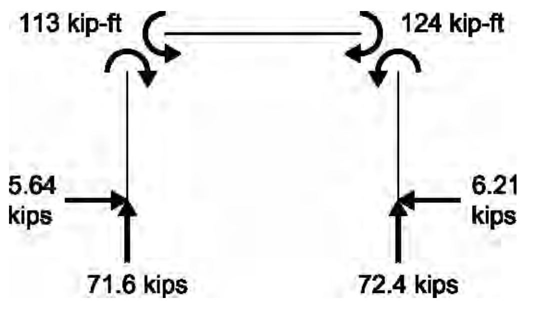
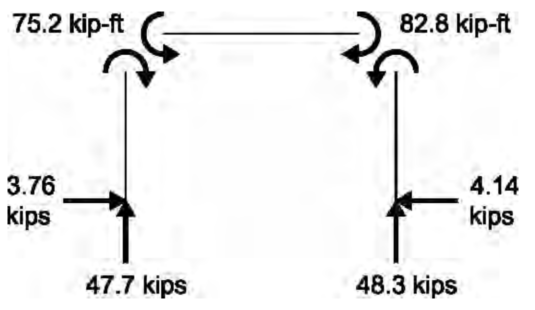
LRFD	ASD (Reactions and moments divided by 1.6)
Δ_{1st} = 0.181 in. 	Δ_{1st} = 0.193 in. (prior to dividing by 1.6) 

Fig. C.1A-3. Results of first-order analysis.

Second-order results

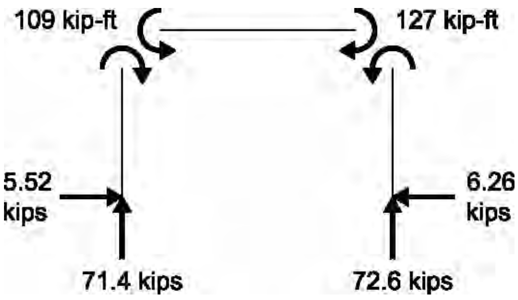
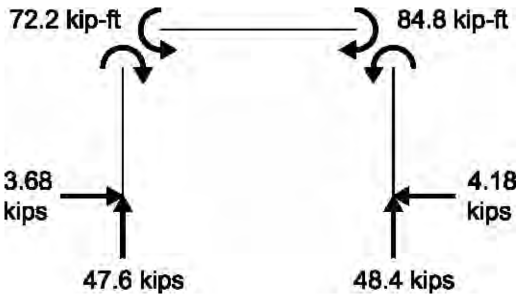
LRFD	ASD (Reactions and moments divided by 1.6)
$\Delta_{2nd} = 0.290 \text{ in.}$ Drift ratio: $\dfrac{\Delta_{2nd}}{\Delta_{1st}} = \dfrac{0.290 \text{ in.}}{0.181 \text{ in.}}$ $= 1.60$ 	$\Delta_{2nd} = 0.321 \text{ in. (prior to dividing by 1.6)}$ Drift ratio: $\dfrac{\Delta_{2nd}}{\Delta_{1st}} = \dfrac{0.321 \text{ in.}}{0.193 \text{ in.}}$ $= 1.66$ 

Fig. C.1A-4. Results of second-order analysis.

Check the assumption that $\alpha P_r / P_{ns} \le 0.5$ on the column on grid line C.

Because a W12×65 column contains no elements that are slender for uniform compression,

$$\begin{aligned} P_{ns} &= F_y A_g \\ &= (50 \text{ ksi})(19.1 \text{ in.}^2) \\ &= 955 \text{ kips} \end{aligned}$$

LRFD	ASD
$\dfrac{\alpha P_r}{P_{ns}} = \dfrac{1.0(72.6 \text{ kips})}{955 \text{ kips}}$ $= 0.0760 \le 0.5$ **o.k.**	$\dfrac{\alpha P_r}{P_{ns}} = \dfrac{1.6(48.4 \text{ kips})}{955 \text{ kips}}$ $= 0.0811 \le 0.5$ **o.k.**

The stiffness assumption used in the analysis, $\tau_b = 1.0$, is verified.

Note that the drift ratio, 1.60 (LRFD) or 1.66 (ASD), does not exceed the recommended limit of 2.5 from AISC *Specification* Commentary Section C1.

The required axial compressive strength in the columns is 72.6 kips (LRFD) or 48.4 kips (ASD). The required bending moment diagram is linear, varying from zero at the bottom to 127 kip-ft (LRFD) or 84.8 kip-ft (ASD) at the top. These required strengths apply to both columns because the notional load must be applied in each direction.

Although the second-order sway multiplier (drift ratio) is fairly large at 1.60 (LRFD) or 1.66 (ASD), the change in bending moment is small because the only sway moments are those produced by the small notional loads. For load combinations with significant gravity and lateral loadings, the increase in bending moments is larger.

Per AISC *Specification* Section C3, the effective length for flexural buckling of all members is taken as the unbraced length (K = 1.0):

L_{cx} = 20.0 ft
L_{cy} = 20.0 ft

EXAMPLE C.1B DESIGN OF A MOMENT FRAME BY THE EFFECTIVE LENGTH METHOD

Given:

Repeat Example C.1A using the effective length method.

Determine the required strengths and effective length factors for the columns in the moment frame shown in Figure C.1B-1 for the maximum gravity load combination, using LRFD and ASD. Use the effective length method.

Columns are unbraced between the footings and roof in the *x*- and *y*-axes and have pinned bases.

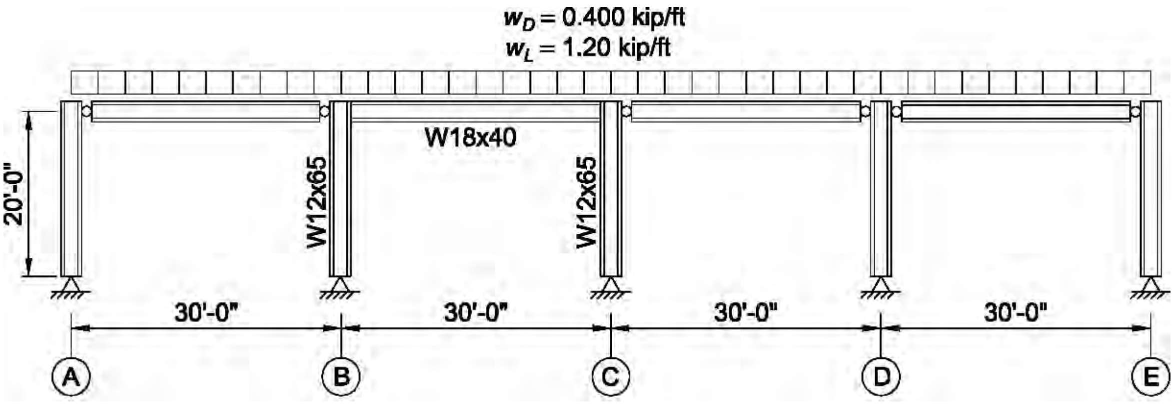


Fig. C.1B-1. Example C.1B moment frame elevation.

Solution:

From AISC *Manual* Table 1-1, the W12×65 has I_x = 533 in.^{4}

The beams from grid lines A to B and C to E and the columns at A, D and E are pinned at both ends and do not contribute to the lateral stability of the frame. There are no P-Δ effects to consider in these members and they may be designed using $L_c = L$.

The moment frame between grid lines B and C is the source of lateral stability and therefore will be evaluated using the provisions of Chapter C of the AISC *Specification*. Although the columns at grid lines A, D and E do not contribute to lateral stability, the forces required to stabilize them must be considered in the moment-frame analysis. The entire frame from grid line A to E could be modeled, but in this case the model is simplified as shown in Figure C.1B-2, in which the stability loads from the three “leaning” columns are combined into a single representative column.

Check the limitations for the use of the effective length method given in AISC *Specification* Appendix 7, Section 7.2.1:

(a) The structure supports gravity loads primarily through nominally vertical columns, walls or frames.
(b) The ratio of maximum second-order drift to the maximum first-order drift (both determined for LRFD load combinations or 1.6 times ASD load combinations, with stiffness not adjusted as specified in AISC *Specification* Section C2.3) in all stories will be assumed to be no greater than 1.5, subject to verification in the following.

From Chapter 2 of ASCE/SEI 7, the maximum gravity load combinations are:

LRFD	ASD
$w_u = 1.2D + 1.6L$ $= 1.2(0.400 \text{ kip/ft}) + 1.6(1.20 \text{ kip/ft})$ $= 2.40 \text{ kip/ft}$	$w_u = D + L$ $= 0.400 \text{ kip/ft} + 1.20 \text{ kip/ft}$ $= 1.60 \text{ kip/ft}$

Per AISC *Specification* Appendix 7, Section 7.2.2, the analysis must conform to the requirements of AISC *Specification* Section C2.1, with the exception of the stiffness reduction required by the provisions of Section C2.1(a).

Per AISC *Specification* Section C2.1(d), for LRFD perform a second-order analysis and member strength checks using the LRFD load combinations. For ASD, perform a second-order analysis at 1.6 times the ASD load combinations and divide the analysis results by 1.6 for the ASD member strength checks.

Frame analysis gravity loads

The uniform gravity loads to be considered in a second-order analysis on the beam from B to C are:

LRFD	ASD
$w'_u = 2.40 \text{ kip/ft}$	$w'_a = 1.6(1.60 \text{ kip/ft})$ $= 2.56 \text{ kip/ft}$

Concentrated gravity loads to be considered in a second-order analysis on the columns at B and C contributed by adjacent beams are:

LRFD	ASD
$P'_u = \dfrac{w'_u l}{2}$ $= \dfrac{(2.40 \text{ kip/ft})(30.0 \text{ ft})}{2}$ $= 36.0 \text{ kips}$	$P'_a = \dfrac{w'_a l}{2}$ $= \dfrac{(2.56 \text{ kip/ft})(30.0 \text{ ft})}{2}$ $= 38.4 \text{ kips}$

Concentrated gravity loads on the representative "leaning" column

The load in this column accounts for all gravity loads that is stabilized by the moment frame, but not directly applied to it.

LRFD	ASD
$P'_{uL} = (60.0 \text{ ft})(2.40 \text{ kip/ft})$ $= 144 \text{ kips}$	$P'_{aL} = (60.0 \text{ ft})(2.56 \text{ kip/ft})$ $= 154 \text{ kips}$

Frame analysis notional loads

Per AISC *Specification* Appendix 7, Section 7.2.2, frame out-of-plumbness must be accounted for by the application of notional loads in accordance with AISC *Specification* Section C2.2b. Note that notional loads need to only be applied to the gravity load combinations per AISC *Specification* Section C2.2b(d) when the requirement that $\Delta_{2nd} / \Delta_{1st} \leq 1.7$ (using stiffness adjusted as specified in Section C2.3) is satisfied. Per the User Note in AISC *Specification* Appendix 7, Section 7.2.2, Section C2.2b(d) will be satisfied in all cases where the effective length method is applicable, and therefore the notional load need only be applied in gravity-only load cases.
From AISC *Specification* Equation C2-1, the notional loads are:

LRFD	ASD
$\alpha = 1.0$	$\alpha = 1.6$
$Y_i = (120\text{ ft})(2.40\text{ kip/ft})$ $= 288\text{ kips}$	$Y_i = (120\text{ ft})(1.60\text{ kip/ft})$ $= 192\text{ kips}$
$N_i = 0.002\alpha Y_i$ (*Spec*. Eq. C2-1) $= 0.002(1.0)(288\text{ kips})$ $= 0.576\text{ kip}$	$N_i = 0.002\alpha Y_i$ (*Spec*. Eq. C2-1) $= 0.002(1.6)(192\text{ kips})$ $= 0.614\text{ kip}$

Summary of applied frame loads

The applied loads are shown in Figure C.1B-2.

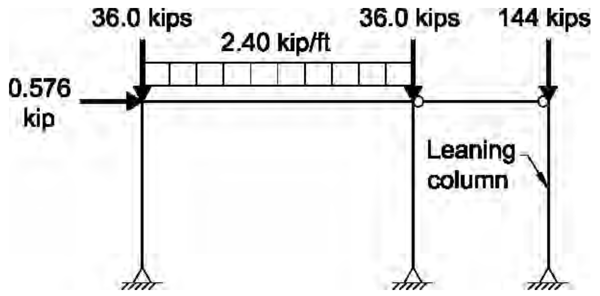
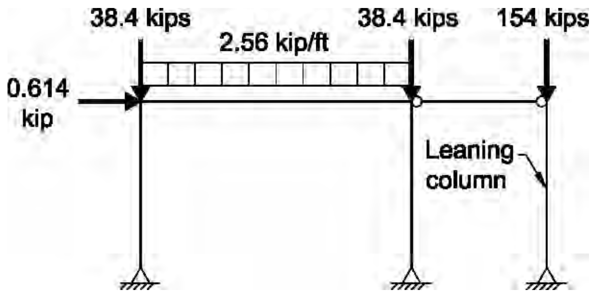
LRFD	ASD
	

Fig. C.1B-2. Applied loads on the analysis model.

Per AISC *Specification* Appendix 7, Section 7.2.2, conduct the analysis using the full nominal stiffnesses.

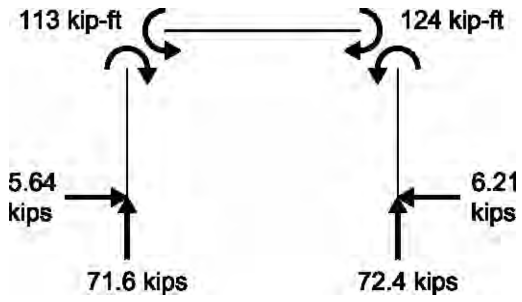
Half of the gravity load is carried by the columns of the moment-resisting frame. Because the gravity load supported by the moment-resisting frame columns exceeds one-third of the total gravity load tributary to the frame, per AISC *Specification* Section C2.1(b), the effects of *P*-δ on the response of the structure must be considered in the frame analysis. This example uses analysis software that accounts for both *P*-Δ and *P*-δ effects. When using software that does not account for *P*-δ effects, this could be accomplished by subdividing columns between the footing and beam.

Figures C.1B-3 and C.1B-4 show results from a first-order and second-order analysis. In each case, the drift is the average of drifts at grid lines B and C.

First-order results

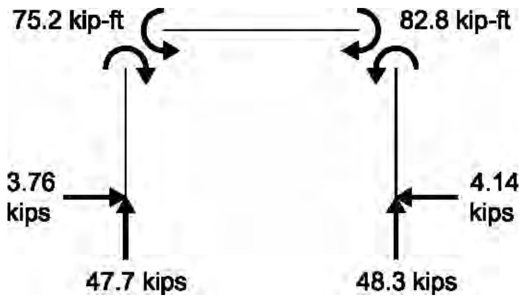
<table>
<tr><th>LRFD</th><th>ASD
(Reactions and moments divided by 1.6)</th></tr>
<tr><td>

Δ_{1st} = 0.145 in.



</td><td>

Δ_{1st} = 0.155 in. (prior to dividing by 1.6)



</td></tr>
</table>

Fig. C.1B-3. Results of first-order analysis.

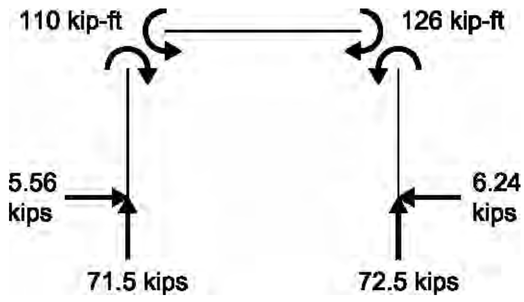
Second-order results

<table>
<tr><th>LRFD</th><th>ASD</th></tr>
<tr><td>

$\Delta_{2nd} = 0.204$ in.

Drift ratio:

$$\frac{\Delta_{2nd}}{\Delta_{1st}} = \frac{0.204 \text{ in.}}{0.145 \text{ in.}}$$
$$= 1.41$$

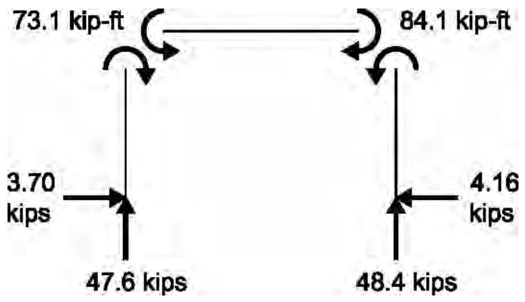


</td><td>

$\Delta_{2nd} = 0.223$ in. (prior to dividing by 1.6)

Drift ratio:

$$\frac{\Delta_{2nd}}{\Delta_{1st}} = \frac{0.223 \text{ in.}}{0.155 \text{ in.}}$$
$$= 1.44$$



</td></tr>
</table>

Fig. C-1B-4. Results of second-order analysis.

The assumption that the ratio of the maximum second-order drift to the maximum first-order drift is no greater than 1.5 is verified; therefore, the effective length method is permitted.

Although the second-order sway multiplier is fairly large at approximately 1.41 (LRFD) or 1.44 (ASD), the change in bending moment is small because the only sway moments for this load combination are those produced by the small notional loads. For load combinations with significant gravity and lateral loadings, the increase in bending moments is larger.

Calculate the in-plane effective length factor, K_x, using the "story stiffness approach" and Equation C-A-7-5 presented in AISC *Specification* Commentary Appendix 7, Section 7.2. With $K_x = K_2$:

$$K_x = \sqrt{\frac{P_{story}}{R_M P_r}\left(\frac{\pi^2 EI}{L^2}\right)\left(\frac{\Delta_H}{HL}\right)} \geq \sqrt{\frac{\pi^2 EI}{L^2}\left(\frac{\Delta_H}{1.7 H_{col} L}\right)} \qquad \text{(\textit{Spec}. Eq. C-A-7-5)}$$

Calculate the total load in all columns, P_{story}, as follows:

LRFD	ASD
$P_{story} = (2.40\text{ kip/ft})(120\text{ ft})$ $= 288\text{ kips}$	$P_{story} = (1.60\text{ kip/ft})(120\text{ ft})$ $= 192\text{ kips}$

Calculate the coefficient to account for the influence of P-δ on P-Δ, R_M, as follows, using AISC *Specification* Commentary Appendix 7, Equation C-A-7-6:

LRFD	ASD
$P_{mf} = 71.5\text{ kips} + 72.5\text{ kips}$ $= 144\text{ kips}$ $R_M = 1 - 0.15\left(P_{mf}/P_{story}\right)$ (*Spec*. Eq. C-A-7-6) $= 1 - 0.15\left(\frac{144\text{ kips}}{288\text{ kips}}\right)$ $= 0.925$	$P_{mf} = 47.6\text{ kips} + 48.4\text{ kips}$ $= 96.0\text{ kips}$ $R_M = 1 - 0.15\left(P_{mf}/P_{story}\right)$ (*Spec*. Eq. C-A-7-6) $= 1 - 0.15\left(\frac{96.0\text{ kips}}{192\text{ kips}}\right)$ $= 0.925$

Calculate the Euler buckling strength of one moment frame.

$$\begin{aligned}\frac{\pi^2 EI}{L^2} &= \frac{\pi^2 (29{,}000\text{ ksi})(533\text{ in.}^4)}{\left[(20.0\text{ ft})(12\text{ in./ft})\right]^2} \\ &= 2{,}650\text{ kips}\end{aligned}$$

From AISC *Specification* Commentary Equation C-A-7-5, for the column at line C:

LRFD	ASD
$K_x = \sqrt{\frac{P_{story}}{R_M P_r}\left(\frac{\pi^2 EI}{L^2}\right)\left(\frac{\Delta_H}{HL}\right)}$ $\geq \sqrt{\left(\frac{\pi^2 EI}{L^2}\right)\left(\frac{\Delta_H}{1.7H_{col}L}\right)}$ $= \sqrt{\left[\frac{288\text{ kips}}{(0.925)(72.5\text{ kips})}\right](2,650\text{ kips}) \times \left[\frac{0.145\text{ in.}}{(0.576\text{ kip})(20.0\text{ ft})(12\text{ in./ft})}\right]}$ $\geq \sqrt{(2,650\text{ kips}) \times \left[\frac{0.145\text{ in.}}{1.7(6.21\text{ kips})(20.0\text{ ft})(12\text{ in./ft})}\right]}$ $= 3.45 \geq 0.389$ Use $K_x = 3.45$	$K_x = \sqrt{\frac{1.6P_{story}}{R_M (1.6) P_r}\left(\frac{\pi^2 EI}{L^2}\right)\left(\frac{\Delta_H}{HL}\right)}$ $\geq \sqrt{\left(\frac{\pi^2 EI}{L^2}\right)\left(\frac{\Delta_H}{1.7(1.6)H_{col}L}\right)}$ $= \sqrt{\left[\frac{1.6(192\text{ kips})}{0.925(1.6)(48.4\text{ kips})}\right](2,650\text{ kips}) \times \left[\frac{0.155\text{ in.}}{(0.614\text{ kip})(20.0\text{ ft})(12\text{ in./ft})}\right]}$ $\geq \sqrt{(2,650\text{ kips}) \times \left[\frac{0.155\text{ in.}}{1.7(1.6)(4.14\text{ kips})(20.0\text{ ft})(12\text{ in./ft})}\right]}$ $= 3.46 \geq 0.390$ Use $K_x = 3.46$

Note that the column loads are multiplied by 1.6 for ASD in Equation C-A-7-5.

With K_x = 3.45 and K_y = 1.00, the column available strengths can be verified for the given member sizes for the second-order forces (calculations not shown), using the following effective lengths:

$$
\begin{aligned}
L_{cx} &= K_x L_x \\
&= 3.45(20.0\text{ ft}) \\
&= 69.0\text{ ft} \\
L_{cy} &= K_y L_y \\
&= 1.00(20.0\text{ ft}) \\
&= 20.0\text{ ft}
\end{aligned}
$$

EXAMPLE C.1C DESIGN OF A MOMENT FRAME BY THE FIRST-ORDER METHOD

Given:

Repeat Example C.1A using the first-order analysis method.

Determine the required strengths and effective length factors for the columns in the moment frame shown in Figure C.1C-1 for the maximum gravity load combination, using LRFD and ASD. Use the first-order analysis method as given in AISC *Specification* Appendix 7, Section 7.3.

Columns are unbraced between the footings and roof in the *x*- and *y*-axes and have pinned bases.

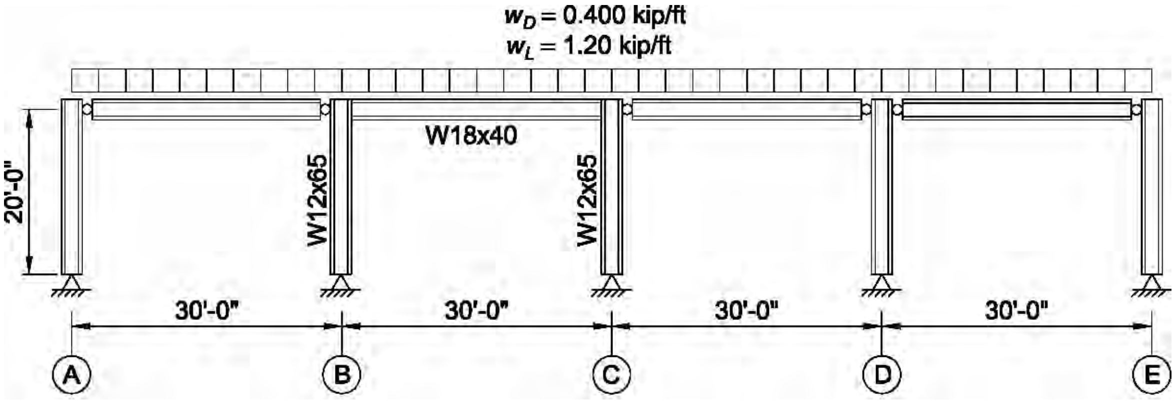


Fig. C.1C-1. Example C.1C moment frame elevation.

Solution:

From AISC *Manual* Table 1-1, the W12×65 has A = 19.1 in.^{2}

The beams from grid lines A to B and C to E and the columns at A, D and E are pinned at both ends and do not contribute to the lateral stability of the frame. There are no P-Δ effects to consider in these members and they may be designed using L_c=L.

The moment frame between grid lines B and C is the source of lateral stability and will be designed using the provisions of AISC *Specification* Appendix 7, Section 7.3. Although the columns at grid lines A, D and E do not contribute to lateral stability, the forces required to stabilize them must be considered in the moment-frame analysis. These members need not be included in the analysis model, except that the forces in the “leaning” columns must be included in the calculation of notional loads.

Check the limitations for the use of the first-order analysis method given in AISC *Specification* Appendix 7, Section 7.3.1:

(a) The structure supports gravity loads primarily through nominally vertical columns, walls or frames.
(b) The ratio of maximum second-order drift to the maximum first-order drift (both determined for LRFD load combinations or 1.6 times ASD load combinations, with stiffnesses not adjusted as specified in AISC *Specification* Section C2.3) in all stories will be assumed to be equal to or less than 1.5, subject to verification.
(c) The required axial compressive strength of all members whose flexural stiffnesses are considered to contribute to the lateral stability of the structure will be assumed to be no more than 50% of the cross-section strength, subject to verification.

Per AISC *Specification* Appendix 7, Section 7.3.2, the required strengths are determined from a first-order analysis using notional loads determined in the following, along with a B_1 multiplier to account for second-order effects, as determined from Appendix 8.

Loads

From Chapter 2 of ASCE/SEI 7, the maximum gravity load combinations are:

LRFD	ASD
$w_u = 1.2D + 1.6L$ $= 1.2(0.400 \text{ kip/ft}) + 1.6(1.20 \text{ kip/ft})$ $= 2.40 \text{ kip/ft}$	$w_u = D + L$ $= 0.400 \text{ kip/ft} + 1.20 \text{ kip/ft}$ $= 1.60 \text{ kip/ft}$

Concentrated gravity loads to be considered on the columns at B and C contributed by adjacent beams are:

LRFD	ASD
$P_u = \dfrac{w_u l}{2}$ $= \dfrac{(2.40 \text{ kip/ft})(30.0 \text{ ft})}{2}$ $= 36.0 \text{ kips}$	$P_a = \dfrac{w_a l}{2}$ $= \dfrac{(1.60 \text{ kip/ft})(30.0 \text{ ft})}{2}$ $= 24.0 \text{ kips}$

Using AISC *Specification* Appendix 7, Section 7.3.2, frame out-of-plumbness is accounted for by the application of an additional lateral load.

From AISC *Specification* Appendix Equation A-7-2, the additional lateral load is determined as follows:

LRFD	ASD
$\alpha = 1.0$	$\alpha = 1.6$
$Y_i = (120 \text{ ft})(2.40 \text{ kip/ft})$ $= 288 \text{ kips}$	$Y_i = (120 \text{ ft})(1.60 \text{ kip/ft})$ $= 192 \text{ kips}$
Δ = 0 in. (no drift for this load combination)	Δ = 0 in. (no drift for this load combination)
$L = (20.0 \text{ ft})(12 \text{ in./ft})$ $= 240 \text{ in.}$	$L = (20.0 \text{ ft})(12 \text{ in./ft})$ $= 240 \text{ in.}$
$N_i = 2.1\alpha(\Delta/L)Y_i \geq 0.0042Y_i$ (*Spec*. Eq. A-7-2) $= 2.1(1.0)\left(\dfrac{0 \text{ in.}}{240 \text{ in.}}\right)(288 \text{ kips})$ $\geq 0.0042(288 \text{ kips})$ $= 0 \text{ kip} < 1.21 \text{ kips}$	$N_i = 2.1\alpha(\Delta/L)Y_i \geq 0.0042Y_i$ (*Spec*. Eq. A-7-2) $= 2.1(1.6)\left(\dfrac{0 \text{ in.}}{240 \text{ in.}}\right)(192 \text{ kips})$ $\geq 0.0042(192 \text{ kips})$ $= 0 \text{ kip} < 0.806 \text{ kip}$
Use N_i = 1.21 kips	Use N_i = 0.806 kip

Summary of applied frame loads

The applied loads are shown in Figure C.1C-2.

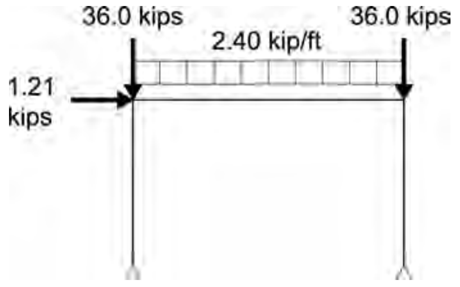
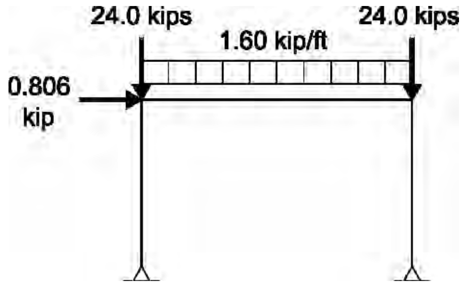
LRFD	ASD
	

Fig. C.1C-2. Applied loads on the analysis model.

Conduct the analysis using the full nominal stiffnesses, as indicated in AISC *Specification* Commentary Appendix 7, Section 7.3.

Using analysis software, the first-order results shown in Figure C.1C-3 are obtained:

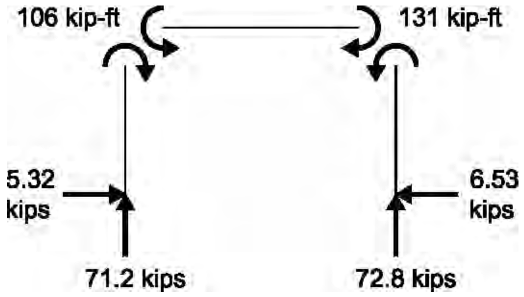
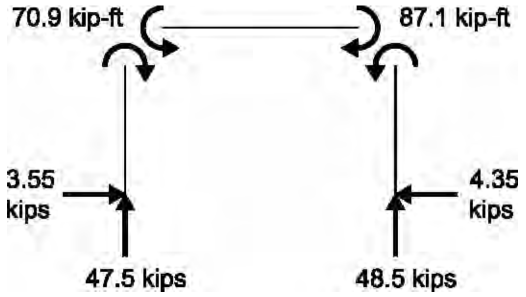
LRFD	ASD
$\Delta_{1st} = 0.304$ in. 	$\Delta_{1st} = 0.203$ in. 

Fig. C.1C-3. Results of first-order analysis.

Check the assumption that the ratio of the second-order drift to the first-order drift does not exceed 1.5. B_2 can be used to check this limit. Calculate B_2 per Appendix 8, Section 8.2.2 using the results of the first-order analysis.

LRFD	ASD
$P_{mf} = 2(36.0 \text{ kips}) + (30.0 \text{ ft})(2.40 \text{ kip/ft})$ $= 144 \text{ kips}$ $P_{story} = 144 \text{ kips} + 4(36.0 \text{ kips})$ $= 288 \text{ kips}$	$P_{mf} = 2(24.0 \text{ kips}) + (30.0 \text{ ft})(1.60 \text{ kip/ft})$ $= 96.0 \text{ kips}$ $P_{story} = 96.0 \text{ kips} + 4(24.0 \text{ kips})$ $= 192 \text{ kips}$

LRFD	ASD
$R_M = 1-0.15\left(P_{mf}/P_{story}\right)$ (*Spec*. Eq. A-8-8) $= 1-0.15\left(144\text{ kips}/288\text{ kips}\right)$ $= 0.925$	$R_M = 1-0.15\left(P_{mf}/P_{story}\right)$ (*Spec*. Eq. A-8-8) $= 1-0.15\left(96.0\text{ kips}/192\text{ kips}\right)$ $= 0.925$
$\Delta_H = 0.304\text{ in.}$	$\Delta_H = 0.203\text{ in.}$
$H = 6.53\text{ kips} - 5.32\text{ kips}$ $= 1.21\text{ kips}$	$H = 4.35\text{ kips} - 3.55\text{ kips}$ $= 0.800\text{ kip}$
$L = \left(20\text{ ft}\right)\left(12\text{ in./ft}\right)$ $= 240\text{ in.}$	$L = \left(20\text{ ft}\right)\left(12\text{ in./ft}\right)$ $= 240\text{ in.}$
$P_{e\ story} = R_M \dfrac{HL}{\Delta_H}$ (*Spec*. Eq. A-8-7) $= 0.925\dfrac{(1.21\text{ kips})\left(240\text{ in.}\right)}{0.304\text{ in.}}$ $= 884\text{ kips}$	$P_{e\ story} = R_M \dfrac{HL}{\Delta_H}$ (*Spec*. Eq. A-8-7) $= 0.925\dfrac{\left(0.800\text{ kip}\right)\left(240\text{ in.}\right)}{0.203\text{ in.}}$ $= 875\text{ kips}$
$\alpha = 1.0$	$\alpha = 1.6$
$B_2 = \dfrac{1}{1-\dfrac{\alpha P_{story}}{P_{e\ story}}} \geq 1$ (*Spec*. Eq. A-8-6) $= \dfrac{1}{1-\dfrac{1.0\left(288\text{ kips}\right)}{884\text{ kips}}} \geq 1$ $= 1.48 > 1$	$B_2 = \dfrac{1}{1-\dfrac{\alpha P_{story}}{P_{e\ story}}} \geq 1$ (*Spec*. Eq. A-8-6) $= \dfrac{1}{1-\dfrac{1.6\left(192\text{ kips}\right)}{875\text{ kips}}} \geq 1$ $= 1.54 > 1$

When a structure with a live-to-dead load ratio of 3 is analyzed by a first-order analysis the required strength for LRFD will always be 1.5 times the required strength for ASD. However, when a second-order analysis is used this ratio is not maintained. This is due to the use of the amplification factor, α, which is set equal to 1.6 for ASD, in order to capture the worst case second-order effects for any live-to-dead load ratio. Thus, in this example the limitation for applying the first-order analysis method, that the ratio of the maximum second-order drift to maximum first-order drift is not greater than 1.5, is verified for LRFD but is not verified for ASD. Therefore, for this example the first-order method is invalid for ASD and will proceed with LRFD only.

Check the assumption that $\alpha P_r \leq 0.5P_{ns}$ and, therefore, the first-order analysis method is permitted.

Because the W12×65 column does not contain elements that are slender for compression,

$$P_{ns} = F_y A_g$$

$$\begin{aligned} 0.5P_{ns} &= 0.5F_y A_g \\ &= 0.5\left(50\text{ ksi}\right)\left(19.1\text{ in.}^2\right) \\ &= 478\text{ kips} \end{aligned}$$

$$\alpha P_r = 1.0(72.8\text{ kips})$$
$$= 72.8\text{ kips} < 478\text{ kips} \quad \textbf{o.k.}\text{ (LRFD only)}$$

The assumption that the first-order analysis method can be used is verified for LRFD.

Although the second-order sway multiplier is 1.48, the change in bending moment is small because the only sway moments are those produced by the small notional loads. For load combinations with significant gravity and lateral loadings, the increase in bending moments is larger.

The column strengths can be verified after using the B_1 amplification given in Appendix 8, Section 8.2.1 to account for second-order effects (calculations not shown here). In the direction of sway, the effective length factor is taken equal to 1.00, and the column effective lengths are as follows:

$$L_{cx} = 20.0\text{ ft}$$
$$L_{cy} = 20.0\text{ ft}$$

Chapter D
Design of Members for Tension

D1. SLENDERNESS LIMITATIONS

AISC *Specification* Section D1 does not establish a slenderness limit for tension members, but recommends limiting L/r to a maximum of 300. This is not an absolute requirement. Rods and hangers are specifically excluded from this recommendation.

D2. TENSILE STRENGTH

Both tensile yielding strength and tensile rupture strength must be considered for the design of tension members. It is not unusual for tensile rupture strength to govern the design of a tension member, particularly for small members with holes or heavier sections with multiple rows of holes.

For preliminary design, tables are provided in Part 5 of the AISC *Manual* for W-shapes, L-shapes, WT-shapes, rectangular HSS, square HSS, round HSS, Pipe, and 2L-shapes. The calculations in these tables for available tensile rupture strength assume an effective area, A_e, of $0.75A_g$. The gross area, A_g, is the total cross-sectional area of the member. If the actual effective area is greater than $0.75A_g$, the tabulated values will be conservative and calculations can be performed to obtain higher available strengths. If the actual effective area is less than $0.75A_g$, the tabulated values will be unconservative and calculations are necessary to determine the available strength.

D3. EFFECTIVE NET AREA

In computing net area, A_n, AISC *Specification* Section B4.3b requires that an extra $\frac{1}{16}$ in. be added to the bolt hole diameter. A computation of the effective area for a chain of holes is presented in Example D.9.

Unless all elements of the cross section are connected, $A_e = A_nU$, where U is a reduction factor to account for shear lag. The appropriate values of U can be obtained from AISC *Specification* Table D3.1.

D4. BUILT-UP MEMBERS

The limitations for connections of built-up members are discussed in Section D4 of the AISC *Specification*.

D5. PIN-CONNECTED MEMBERS

An example of a pin-connected member is given in Example D.7.

D6. EYEBARS

An example of an eyebar is given in Example D.8. The strength of an eyebar meeting the dimensional requirements of AISC *Specification* Section D6 is governed by tensile yielding of the body.

EXAMPLE D.1 W-SHAPE TENSION MEMBER

Given:

Select an ASTM A992 W-shape with 8 in. nominal depth to carry a dead load of 30 kips and a live load of 90 kips in tension. The member is 25.0 ft long. Verify the member strength by both LRFD and ASD with the bolted end connection as shown in Figure D.1-1. Verify that the member satisfies the recommended slenderness limit. Assume that connection limit states do not govern.

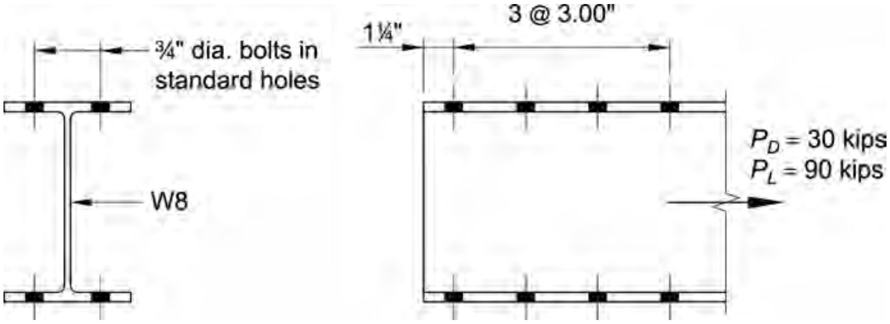


Fig D.1-1. Connection geometry for Example D.1.

Solution:

From Chapter 2 of ASCE/SEI 7, the required tensile strength is:

LRFD	ASD
$P_u = 1.2(30 \text{ kips}) + 1.6(90 \text{ kips})$ $= 180 \text{ kips}$	$P_a = 30 \text{ kips} + 90 \text{ kips}$ $= 120 \text{ kips}$

From AISC *Manual* Table 5-1, try a W8×21.

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W8×21
$A_g = 6.16$ in.^{2}
$b_f = 5.27$ in.
$t_f = 0.400$ in.
$d = 8.28$ in.
$r_y = 1.26$ in.

The WT-shape corresponding to a W8×21 is a WT4×10.5. From AISC *Manual* Table 1-8, the geometric properties are as follows:

WT4×10.5
$\bar{y} = 0.831$ in.

Tensile Yielding

From AISC *Manual* Table 5-1, the available tensile yielding strength of a W8×21 is:

LRFD	ASD
$\phi_t P_n = 277 \text{ kips} > 180 \text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_t} = 184 \text{ kips} > 120 \text{ kips}$ **o.k.**

Tensile Rupture

Verify the table assumption that $A_e / A_g \geq 0.75$ for this connection.

From the description of the element in AISC *Specification* Table D3.1, Case 7, calculate the shear lag factor, *U*, as the larger of the values from AISC *Specification* Section D3, Table D3.1 Case 2 and Case 7.

From AISC *Specification* Section D3, for open cross sections, *U* need not be less than the ratio of the gross area of the connected element(s) to the member gross area.

$$
\begin{aligned}
U &= \frac{2b_f t_f}{A_g} \\
&= \frac{2(5.27 \text{ in.})(0.400 \text{ in.})}{6.16 \text{ in.}^2} \\
&= 0.684
\end{aligned}
$$

Case 2: Determine *U* based on two WT-shapes per AISC *Specification* Commentary Figure C-D3.1, with $\bar{x} = \bar{y} = 0.831$ in. and where l is the length of connection.

$$
\begin{aligned}
U &= 1 - \frac{\bar{x}}{l} \\
&= 1 - \frac{0.831 \text{ in.}}{9.00 \text{ in.}} \\
&= 0.908
\end{aligned}
$$

Case 7:

$$
\begin{aligned}
b_f &= 5.27 \text{ in.} \\
\frac{2}{3}d &= \frac{2}{3}(8.28 \text{ in.}) \\
&= 5.52 \text{ in.}
\end{aligned}
$$

Because the flange is connected with three or more fasteners per line in the direction of loading and $b_f < \frac{2}{3}d$:

$$U = 0.85$$

Therefore, use the larger $U = 0.908$.

Calculate A_n using AISC *Specification* Section B4.3b.

$$\begin{aligned} A_n &= A_g - 4\left(d_h + \tfrac{1}{16}\text{ in.}\right)t_f \\ &= 6.16\text{ in.}^2 - 4\left(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)(0.400\text{ in.}) \\ &= 4.76\text{ in.}^2 \end{aligned}$$

Calculate A_e using AISC *Specification* Section D3.

$$\begin{aligned} A_e &= A_nU \\ &= \left(4.76\text{ in.}^2\right)(0.908) \\ &= 4.32\text{ in.}^2 \end{aligned} \qquad (\textit{Spec}.\text{ Eq. D3-1})$$

$$\begin{aligned} \frac{A_e}{A_g} &= \frac{4.32\text{ in.}^2}{6.16\text{ in.}^2} \\ &= 0.701 < 0.75 \end{aligned}$$

Because $A_e/A_g < 0.75$, the tensile rupture strength from AISC *Manual* Table 5-1 is not valid. The available tensile rupture strength is determined using AISC *Specification* Section D2 as follows:

$$\begin{aligned} P_n &= F_uA_e \\ &= (65\text{ ksi})\left(4.32\text{ in.}^2\right) \\ &= 281\text{ kips} \end{aligned} \qquad (\textit{Spec}.\text{ Eq. D2-2})$$

From AISC *Specification* Section D2, the available tensile rupture strength is:

LRFD	ASD
$\phi_t = 0.75$ $\begin{aligned} \phi_tP_n &= 0.75(281\text{ kips}) \\ &= 211\text{ kips} > 180\text{ kips} \quad \textbf{o.k.} \end{aligned}$	$\Omega_t = 2.00$ $\begin{aligned} \frac{P_n}{\Omega_t} &= \frac{281\text{ kips}}{2.00} \\ &= 141\text{ kips} > 120\text{ kips} \quad \textbf{o.k.} \end{aligned}$

Note that the W8×21 available tensile strength is governed by the tensile rupture limit state at the end connection versus the tensile yielding limit state.

See Chapter J for illustrations of connection limit state checks.

Check Recommended Slenderness Limit

$$\begin{aligned} \frac{L}{r} &= \frac{(25.0\text{ ft})(12\text{ in./ft})}{1.26\text{ in.}} \\ &= 238 < 300\text{ from AISC } \textit{Specification} \text{ Section D1} \quad \textbf{o.k.} \end{aligned}$$

EXAMPLE D.2 SINGLE-ANGLE TENSION MEMBER

Given:

Verify the tensile strength of an ASTM A36 L4×4×½ with one line of four ¾-in.-diameter bolts in standard holes, as shown in Figure D.2-1. The member carries a dead load of 20 kips and a live load of 60 kips in tension. Additionally, calculate at what length this tension member would cease to satisfy the recommended slenderness limit. Assume that connection limit states do not govern.

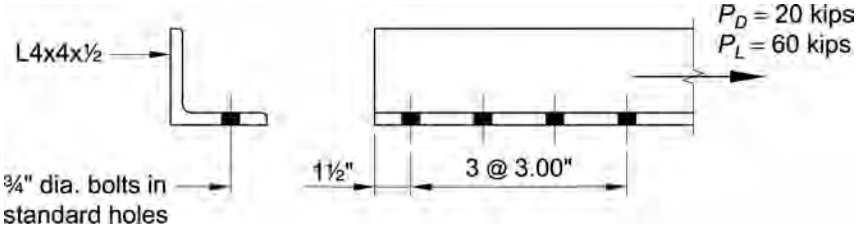


Fig. D.2-1. Connection geometry for Example D.2.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-7, the geometric properties are as follows:

L4×4×½
$A_g = 3.75 \text{ in.}^2$
$r_z = 0.776$ in.
$\bar{x} = 1.18$ in.

From Chapter 2 of ASCE/SEI 7, the required tensile strength is:

LRFD	ASD
$P_u = 1.2(20 \text{ kips}) + 1.6(60 \text{ kips})$ $= 120 \text{ kips}$	$P_a = 20 \text{ kips} + 60 \text{ kips}$ $= 80.0 \text{ kips}$

Tensile Yielding

$$P_n = F_y A_g \qquad (\textit{Spec}. \text{ Eq. D2-1})$$
$$= (36 \text{ ksi})(3.75 \text{ in.}^2)$$
$$= 135 \text{ kips}$$

From AISC *Specification* Section D2, the available tensile yielding strength is:

LRFD	ASD
$\phi_t = 0.90$ $\phi_t P_n = 0.90(135\text{ kips})$ $= 122\text{ kips} > 120\text{ kips}$ **o.k.**	$\Omega_t = 1.67$ $\dfrac{P_n}{\Omega_t} = \dfrac{135\text{ kips}}{1.67}$ $= 80.8\text{ kips} > 80.0\text{ kips}$ **o.k.**

Tensile Rupture

From the description of the element in AISC *Specification* Table D3.1 Case 8, calculate the shear lag factor, *U,* as the larger of the values from AISC *Specification* Section D3, Table D3.1 Case 2 and Case 8.

From AISC *Specification* Section D3, for open cross sections, U need not be less than the ratio of the gross area of the connected element(s) to the member gross area. Half of the member is connected, therefore, the minimum value of U is:

$$U = 0.500$$

Case 2, where l is the length of connection and $\bar{y} = \bar{x}$:

$$\begin{aligned} U &= 1 - \frac{\bar{x}}{l} \\ &= 1 - \frac{1.18\text{ in.}}{9.00\text{ in.}} \\ &= 0.869 \end{aligned}$$

Case 8, with four or more fasteners per line in the direction of loading:

$$U = 0.80$$

Therefore, use the larger $U = 0.869$.

Calculate A_n using AISC *Specification* Section B4.3b.

$$\begin{aligned} A_n &= A_g - (d_h + \tfrac{1}{16}\text{ in.})t \\ &= 3.75\text{ in.} - (\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})(\tfrac{1}{2}\text{ in.}) \\ &= 3.31\text{ in.}^2 \end{aligned}$$

Calculate A_e using AISC *Specification* Section D3.

$$\begin{aligned} A_e &= A_n U \\ &= (3.31\text{ in.}^2)(0.869) \\ &= 2.88\text{ in.}^2 \end{aligned} \qquad \text{(}\textit{Spec}\text{. Eq. D3-1)}$$

$$\begin{aligned} P_n &= F_u A_e \\ &= (58\text{ ksi})(2.88\text{ in.}^2) \\ &= 167\text{ kips} \end{aligned} \qquad \text{(}\textit{Spec}\text{. Eq. D2-2)}$$

From AISC *Specification* Section D2, the available tensile rupture strength is:

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(167 \text{ kips})$ $= 125 \text{ kips} > 120 \text{ kips}$ **o.k.**	$\Omega_t = 2.00$ $\dfrac{P_n}{\Omega_t} = \dfrac{167 \text{ kips}}{2.00}$ $= 83.5 \text{ kips} > 80.0 \text{ kips}$ **o.k.**

The L4×4×½ available tensile strength is governed by the tensile yielding limit state.

LRFD	ASD
$\phi_t P_n = 122 \text{ kips} > 120 \text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_t} = 80.8 \text{ kips} > 80.0 \text{ kips}$ **o.k.**

Recommended L_{max}

Using AISC *Specification* Section D1:

$$
\begin{aligned}
L_{max} &= 300 r_z \\
&= 300\left(\frac{0.776 \text{ in.}}{12 \text{ in./ft}}\right) \\
&= 19.4 \text{ ft}
\end{aligned}
$$

Note: The *L/r* limit is a recommendation, not a requirement.

See Chapter J for illustrations of connection limit state checks.

EXAMPLE D.3 WT-SHAPE TENSION MEMBER

Given:

An ASTM A992 WT6×20 member has a length of 30 ft and carries a dead load of 40 kips and a live load of 120 kips in tension. As shown in Figure D3-1, the end connection is fillet welded on each side for 16 in. Verify the member tensile strength by both LRFD and ASD. Assume that the gusset plate and the weld are satisfactory.

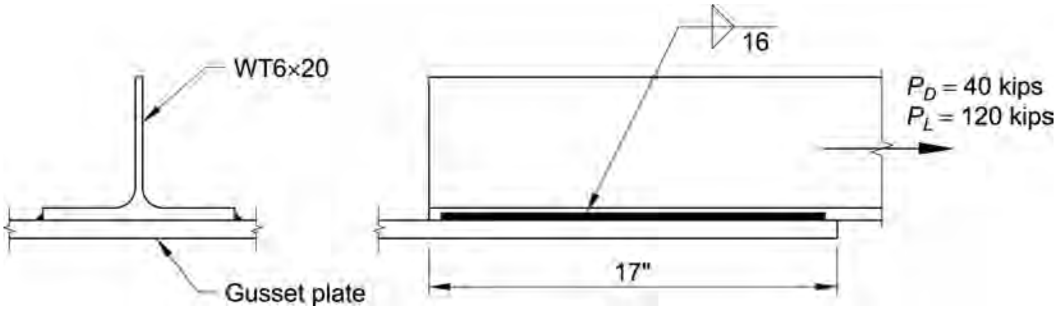


Fig. D.3-1. Connection geometry for Example D.3.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 1-8, the geometric properties are as follows:

WT6×20
A_g = 5.84 in.^{2}
b_f = 8.01 in.
t_f = 0.515 in.
r_x = 1.57 in.
$\bar{y}$ = 1.09 in.

From Chapter 2 of ASCE/SEI 7, the required tensile strength is:

LRFD	ASD
$P_u = 1.2(40 \text{ kips}) + 1.6(120 \text{ kips})$ $= 240 \text{ kips}$	$P_a = 40 \text{ kips} + 120 \text{ kips}$ $= 160 \text{ kips}$

Tensile Yielding

Check tensile yielding limit state using AISC *Manual* Table 5-3.

LRFD	ASD
$\phi_t P_n = 263 \text{ kips} > 240 \text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_t} = 175 \text{ kips} > 160 \text{ kips}$ **o.k.**

Tensile Rupture

Check tensile rupture limit state using AISC *Manual* Table 5-3.

LRFD	ASD
$\phi_t P_n = 214 \text{ kips} < 240 \text{ kips}$ **n.g.**	$\dfrac{P_n}{\Omega_t} = 142 \text{ kips} < 160 \text{ kips}$ **n.g.**

The tabulated available rupture strengths don't work and may be conservative for this case; therefore, calculate the exact solution.

Calculate U as the larger of the values from AISC *Specification* Section D3 and Table D3.1 Case 4.

From AISC *Specification* Section D3, for open cross sections, U need not be less than the ratio of the gross area of the connected element(s) to the member gross area.

$$\begin{aligned} U &= \frac{b_f t_f}{A_g} \\ &= \frac{(8.01 \text{ in.})(0.515 \text{ in.})}{5.84 \text{ in.}^2} \\ &= 0.706 \end{aligned}$$

Case 4, where l is the length of the connection and $\bar{x} = \bar{y}$:

$$\begin{aligned} U &= \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right) \\ &= \left[\frac{3(16.0 \text{ in.})^2}{3(16.0 \text{ in.})^2 + (8.01 \text{ in.})^2}\right]\left(1 - \frac{1.09 \text{ in.}}{16.0 \text{ in.}}\right) \\ &= 0.860 \end{aligned}$$

Therefore, use $U = 0.860$.

Calculate A_n using AISC *Specification* Section B4.3. Because there are no reductions due to bolt holes or notches:

$$\begin{aligned} A_n &= A_g \\ &= 5.84 \text{ in.}^2 \end{aligned}$$

Calculate A_e using AISC *Specification* Section D3.

$$\begin{aligned} A_e &= A_n U \\ &= \left(5.84 \text{ in.}^2\right)(0.860) \\ &= 5.02 \text{ in.}^2 \end{aligned}$$ (*Spec*. Eq. D3-1)

Calculate P_n.

$$\begin{aligned} P_n &= F_u A_e \\ &= (65 \text{ ksi})\left(5.02 \text{ in.}^2\right) \\ &= 326 \text{ kips} \end{aligned}$$ (*Spec*. Eq. D2-2)

From AISC *Specification* Section D2, the available tensile rupture strength is:

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(326 \text{ kips})$ $= 245 \text{ kips} > 240 \text{ kips}$ **o.k.**	$\Omega_t = 2.00$ $\dfrac{P_n}{\Omega_t} = \dfrac{326 \text{ kips}}{2.00}$ $= 163 \text{ kips} > 160 \text{ kips}$ **o.k.**

Alternately, the available tensile rupture strengths can be determined by modifying the tabulated values. The available tensile rupture strengths published in the tension member selection tables are based on the assumption that $A_e = 0.75A_g$. The actual available strengths can be determined by adjusting the values from AISC *Manual* Table 5-3 as follows:

LRFD	ASD
$\phi_t P_n = (214 \text{ kips})\left(\dfrac{A_e}{0.75A_g}\right)$ $= (214 \text{ kips})\left[\dfrac{5.02 \text{ in.}^2}{0.75(5.84 \text{ in.}^2)}\right]$ $= 245 \text{ kips} > 240 \text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_t} = (142 \text{ kips})\left(\dfrac{A_e}{0.75A_g}\right)$ $= (142 \text{ kips})\left[\dfrac{5.02 \text{ in.}^2}{0.75(5.84 \text{ in.}^2)}\right]$ $= 163 \text{ kips} > 160 \text{ kips}$ **o.k.**

Recommended Slenderness Limit

$$\frac{L}{r_x} = \frac{(30.0 \text{ ft})(12 \text{ in./ft})}{1.57 \text{ in.}}$$

$$= 229 < 300 \text{ from AISC } \textit{Specification} \text{ Section D1} \quad \textbf{o.k.}$$

Note: The L/r_x limit is a recommendation, not a requirement.

See Chapter J for illustrations of connection limit state checks.

EXAMPLE D.4 RECTANGULAR HSS TENSION MEMBER

Given:

Verify the tensile strength of an ASTM A500 Grade C HSS6×4×⅜ with a length of 30 ft. The member is carrying a dead load of 40 kips and a live load of 110 kips in tension. As shown in Figure D.4-1, the end connection is a fillet welded ½-in.-thick single concentric gusset plate with a weld length of 16 in. Assume that the gusset plate and weld are satisfactory.

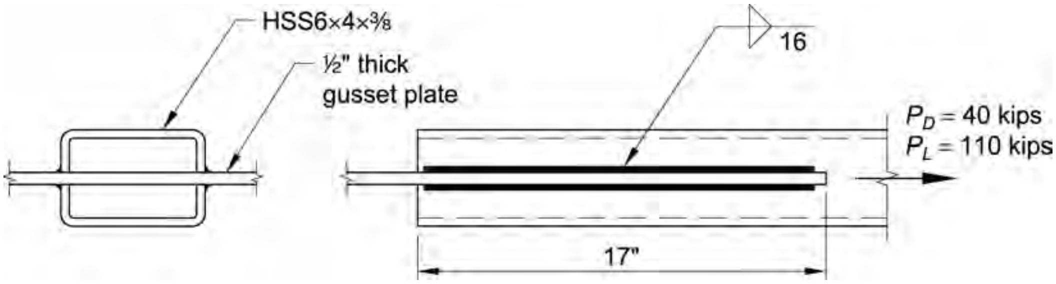


Fig. D.4-1. Connection geometry for Example D.4.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500 Grade C, rectangular HSS
$F_y = 50$ ksi
$F_u = 62$ ksi

From AISC *Manual* Table 1-11, the geometric properties are as follows:

HSS6×4×⅜
$A_g = 6.18$ in.^{2}
$r_y = 1.55$ in.
$t = 0.349$ in.

From Chapter 2 of ASCE/SEI 7, the required tensile strength is:

LRFD	ASD
$P_u = 1.2(40\text{ kips}) + 1.6(110\text{ kips})$ $= 224\text{ kips}$	$P_a = 40\text{ kips} + 110\text{ kips}$ $= 150\text{ kips}$

Tensile Yielding

Check tensile yielding limit state using AISC *Manual* Table 5-4.

LRFD	ASD
$\phi_t P_n = 278\text{ kips} > 224\text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_t} = 185\text{ kips} > 150\text{ kips}$ **o.k.**

Tensile Rupture

Check tensile rupture limit state using AISC *Manual* Table 5-4.

LRFD	ASD
$\phi_t P_n = 216 \text{ kips} < 224 \text{ kips}$ **n.g.**	$\dfrac{P_n}{\Omega_t} = 144 \text{ kips} < 150 \text{ kips}$ **n.g.**

The tabulated available rupture strengths may be conservative in this case; therefore, calculate the exact solution.

Calculate U from AISC *Specification* Section D3 and Table D3.1 Case 6.

$$\begin{aligned}\bar{x} &= \frac{B^2 + 2BH}{4(B+H)} \\ &= \frac{(4.00 \text{ in.})^2 + 2(4.00 \text{ in.})(6.00 \text{ in.})}{4(4.00 \text{ in.} + 6.00 \text{ in.})} \\ &= 1.60 \text{ in.}\end{aligned}$$

$$\begin{aligned}U &= 1 - \frac{\bar{x}}{l} \\ &= 1 - \frac{1.60 \text{ in.}}{16.0 \text{ in.}} \\ &= 0.900\end{aligned}$$

Allowing for a 1/16-in. gap in fit-up between the HSS and the gusset plate:

$$\begin{aligned}A_n &= A_g - 2(t_p + 1/16 \text{ in.})t \\ &= 6.18 \text{ in.}^2 - 2(1/2 \text{ in.} + 1/16 \text{ in.})(0.349 \text{ in.}) \\ &= 5.79 \text{ in.}^2\end{aligned}$$

Calculate A_e using AISC *Specification* Section D3.

$$\begin{aligned}A_e &= A_n U \\ &= (5.79 \text{ in.}^2)(0.900) \\ &= 5.21 \text{ in.}^2\end{aligned}$$

(*Spec.* Eq. D3-1)

Calculate P_n.

$$\begin{aligned}P_n &= F_u A_e \\ &= (62 \text{ ksi})(5.21 \text{ in.}^2) \\ &= 323 \text{ kips}\end{aligned}$$

(*Spec.* Eq. D2-2)

From AISC *Specification* Section D2, the available tensile rupture strength is:

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(323 \text{ kips})$ $= 242 \text{ kips} > 224 \text{ kips}$ **o.k.**	$\Omega_t = 2.00$ $\dfrac{P_n}{\Omega_t} = \dfrac{323 \text{ kips}}{2.00}$ $= 162 \text{ kips} > 150 \text{ kips}$ **o.k.**

The HSS available tensile strength is governed by the tensile rupture limit state.

Recommended Slenderness Limit

$$\frac{L}{r} = \frac{(30.0 \text{ ft})(12 \text{ in./ft})}{1.55 \text{ in.}}$$

$$= 232 < 300 \text{ from AISC } \textit{Specification} \text{ Section D1} \quad \textbf{o.k.}$$

Note: The L/r limit is a recommendation, not a requirement.

See Chapter J for illustrations of connection limit state checks.

EXAMPLE D.5 ROUND HSS TENSION MEMBER

Given:

Verify the tensile strength of an ASTM A500 Grade C HSS6.000×0.500 with a length of 30 ft. The member carries a dead load of 40 kips and a live load of 120 kips in tension. As shown in Figure D.5-1, the end connection is a fillet welded ½-in.-thick single concentric gusset plate with a weld length of 16 in. Assume that the gusset plate and weld are satisfactory.

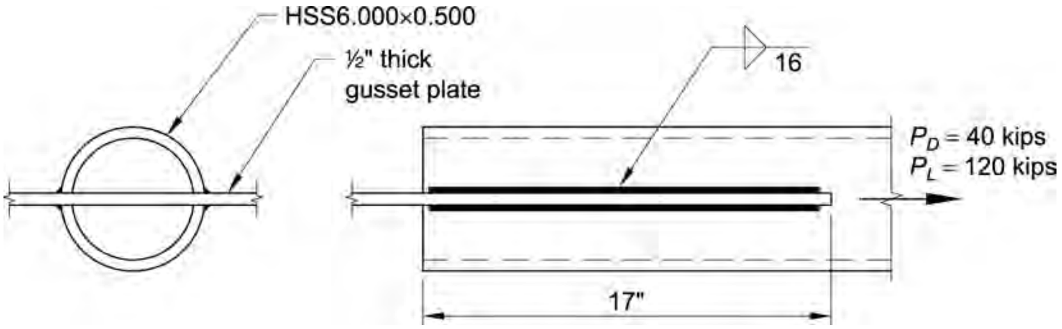


Fig. D.5-1. Connection geometry for Example D.5.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500 Grade C, round HSS
$F_y = 46$ ksi
$F_u = 62$ ksi

From AISC *Manual* Table 1-13, the geometric properties are as follows:

HSS6.000×0.500
$A_g = 8.09\text{ in.}^2$
$r = 1.96$ in.
$t = 0.465$ in.

From Chapter 2 of ASCE/SEI 7, the required tensile strength is:

LRFD	ASD
$P_u = 1.2(40\text{ kips}) + 1.6(120\text{ kips})$ $= 240\text{ kips}$	$P_a = 40\text{ kips} + 120\text{ kips}$ $= 160\text{ kips}$

Tensile Yielding

Check tensile yielding limit state using AISC *Manual* Table 5-6.

LRFD	ASD
$\phi_t P_n = 335\text{ kips} > 240\text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_t} = 223\text{ kips} > 160\text{ kips}$ **o.k.**

Tensile Rupture

Check tensile rupture limit state using AISC *Manual* Table 5-6.

LRFD	ASD
$\phi_t P_n = 282 \text{ kips} > 240 \text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_t} = 188 \text{ kips} > 160 \text{ kips}$ **o.k.**

Check that $A_e/A_g \geq 0.75$ as assumed in table.

Determine U from AISC *Specification* Table D3.1 Case 5.

$$\begin{aligned} l &= 16.0 \text{ in.} \\ D &= 6.00 \text{ in.} \\ \frac{l}{D} &= \frac{16.0 \text{ in.}}{6.00 \text{ in.}} \\ &= 2.67 > 1.3, \text{ therefore } U = 1.0 \end{aligned}$$

Allowing for a 1⁄16-in. gap in fit-up between the HSS and the gusset plate,

$$\begin{aligned} A_n &= A_g - 2\left(t_p + \tfrac{1}{16} \text{ in.}\right)t \\ &= 8.09 \text{ in.}^2 - 2\left(\tfrac{1}{2} \text{ in.} + \tfrac{1}{16} \text{ in.}\right)(0.465 \text{ in.}) \\ &= 7.57 \text{ in.}^2 \end{aligned}$$

Calculate A_e using AISC *Specification* Section D3.

$$\begin{aligned} A_e &= A_n U \\ &= \left(7.57 \text{ in.}^2\right)(1.0) \\ &= 7.57 \text{ in.}^2 \end{aligned} \qquad \text{(}\textit{Spec}\text{. Eq. D3-1)}$$

$$\begin{aligned} \frac{A_e}{A_g} &= \frac{7.57 \text{ in.}^2}{8.09 \text{ in.}^2} \\ &= 0.936 > 0.75 \quad \textbf{o.k.} \end{aligned}$$

Because AISC *Manual* Table 5-6 provides an overly conservative estimate of the available tensile rupture strength for this example, calculate P_n using AISC *Specification* Section D2.

$$\begin{aligned} P_n &= F_u A_e \\ &= (62 \text{ ksi})\left(7.57 \text{ in.}^2\right) \\ &= 469 \text{ kips} \end{aligned} \qquad \text{(Spec. Eq. D2-2)}$$

From AISC *Specification* Section D2, the available tensile rupture strength is:

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(469\text{ kips})$ $= 352\text{ kips} > 240\text{ kips}$ **o.k.**	$\Omega_t = 2.00$ $\dfrac{P_n}{\Omega_t} = \dfrac{469\text{ kips}}{2.00}$ $= 235\text{ kips} > 160\text{ kips}$ **o.k.**

The HSS available strength is governed by the tensile yielding limit state.

Recommended Slenderness Limit

$$\frac{L}{r} = \frac{(30.0\text{ ft})(12\text{ in./ft})}{1.96\text{ in.}}$$

$$= 184 < 300 \text{ from AISC } \textit{Specification} \text{ Section D1} \quad \textbf{o.k.}$$

Note: The L/r limit is a recommendation, not a requirement.

See Chapter J for illustrations of connection limit state checks.

EXAMPLE D.6 DOUBLE-ANGLE TENSION MEMBER

Given:

An ASTM A36 2L4×4×½ (⅜-in. separation) has one line of eight ¾-in.-diameter bolts in standard holes and is 25 ft in length as shown in Figure D.6-1. The double angle is carrying a dead load of 40 kips and a live load of 120 kips in tension. Verify the member tensile strength. Assume that the gusset plate and bolts are satisfactory.

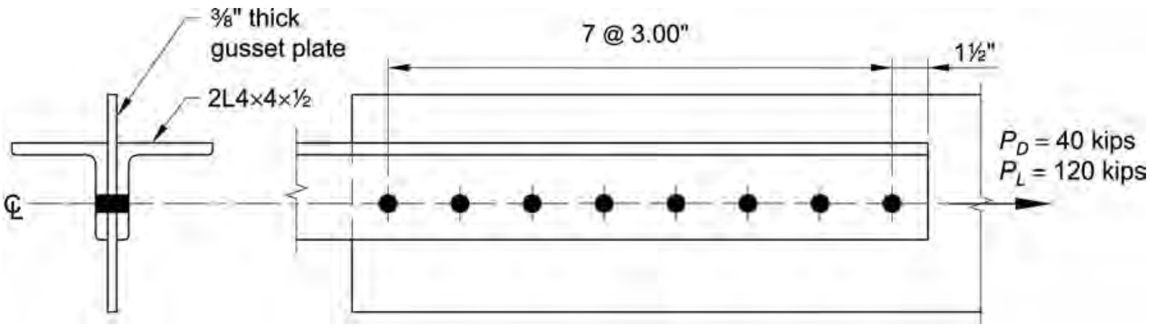


Fig. D.6-1. Connection geometry for Example D.6.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Tables 1-7 and 1-15, the geometric properties are as follows:

L4×4×½
$\bar{x} = 1.18$ in.

2L4×4×½ (s = ⅜ in.)
$A_g = 7.50$ in.^{2}
$r_y = 1.83$ in.
$r_x = 1.21$ in.

From Chapter 2 of ASCE/SEI 7, the required tensile strength is:

LRFD	ASD
$P_u = 1.2(40\text{ kips}) + 1.6(120\text{ kips})$ $= 240\text{ kips}$	$P_a = 40\text{ kips} + 120\text{ kips}$ $= 160\text{ kips}$

Tensile Yielding

Check tensile yielding limit state using AISC *Manual* Table 5-8.

LRFD	ASD
$\phi_t P_n = 243\text{ kips} > 240\text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_t} = 162\text{ kips} > 160\text{ kips}$ **o.k.**

Tensile Rupture

Determine the available tensile rupture strength using AISC *Specification* Section D2. Calculate U as the larger of the values from AISC *Specification* Section D3, Table D3.1 Case 2 and Case 8.

From AISC *Specification* Section D3, for open cross sections, U need not be less than the ratio of the gross area of the connected element(s) to the member gross area. Half of the member is connected, therefore, the minimum U value is:

$$U = 0.500$$

From Case 2, where l is the length of connection:

$$\begin{aligned} U &= 1 - \frac{\bar{x}}{l} \\ &= 1 - \frac{1.18 \text{ in.}}{21.0 \text{ in.}} \\ &= 0.944 \end{aligned}$$

From Case 8, with four or more fasteners per line in the direction of loading:

$$U = 0.80$$

Therefore, use $U = 0.944$.

Calculate A_n using AISC *Specification* Section B4.3.

$$\begin{aligned} A_n &= A_g - 2\left(d_h + \tfrac{1}{16} \text{ in.}\right)t \\ &= 7.50 \text{ in.}^2 - 2\left(\tfrac{13}{16} \text{ in.} + \tfrac{1}{16} \text{ in.}\right)\left(\tfrac{1}{2} \text{ in.}\right) \\ &= 6.63 \text{ in.}^2 \end{aligned}$$

Calculate A_e using AISC *Specification* Section D3.

$$\begin{aligned} A_e &= A_n U \\ &= \left(6.63 \text{ in.}^2\right)(0.944) \\ &= 6.26 \text{ in.}^2 \end{aligned} \qquad (\textit{Spec}.\ \text{Eq. D3-1})$$

Calculate P_n.

$$\begin{aligned} P_n &= F_u A_e \\ &= (58 \text{ ksi})\left(6.26 \text{ in.}^2\right) \\ &= 363 \text{ kips} \end{aligned} \qquad (\textit{Spec}.\ \text{Eq. D2-2})$$

From AISC *Specification* Section D2, the available tensile rupture strength is:

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(363 \text{ kips})$ $= 272 \text{ kips}$	$\Omega_t = 2.00$ $\dfrac{P_n}{\Omega_t} = \dfrac{363 \text{ kips}}{2.00}$ $= 182 \text{ kips}$

Note that AISC *Manual* Table 5-8 could also be conservatively used since $A_e \geq 0.75A_g$.

The double-angle available tensile strength is governed by the tensile yielding limit state.

LRFD	ASD
$243 \text{ kips} > 240 \text{ kips}$ **o.k.**	$162 \text{ kips} > 160 \text{ kips}$ **o.k.**

Recommended Slenderness Limit

$$\frac{L}{r_x} = \frac{(25.0 \text{ ft})(12 \text{ in./ft})}{1.21 \text{ in.}}$$

$$= 248 < 300 \text{ from AISC } \textit{Specification} \text{ Section D1} \quad \textbf{o.k.}$$

Note: From AISC *Specification* Section D4, the longitudinal spacing of connectors between components of built-up members should preferably limit the slenderness ratio in any component between the connectors to a maximum of 300.

See Chapter J for illustrations of connection limit state checks.

EXAMPLE D.7 PIN-CONNECTED TENSION MEMBER

Given:

An ASTM A36 pin-connected tension member with the dimensions shown in Figure D.7-1 carries a dead load of 4 kips and a live load of 12 kips in tension. The diameter of the pin is 1 in., in a 1⁄32-in. oversized hole. Assume that the pin itself is adequate. Verify the member tensile strength.

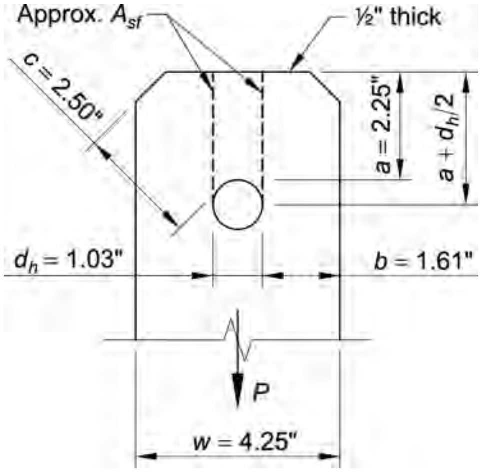


Fig. D.7-1. Connection geometry for Example D.7.

Solution:

From AISC *Manual* Table 2-5, the material properties are as follows:

Plate
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

The geometric properties of the plate are as follows:

$$\begin{aligned}
a &= 2.25 \text{ in.}\\
b &= 1.61 \text{ in.}\\
c &= 2.50 \text{ in.}\\
d &= 1.00 \text{ in.}\\
d_h &= 1.03 \text{ in.}\\
t &= \tfrac{1}{2} \text{ in.}\\
w &= 4.25 \text{ in.}
\end{aligned}$$

The requirements given in AISC *Specification* Sections D5.2(a) and D5.2(b) are satisfied by the given geometry. Requirements given in AISC *Specification* Sections D5.2(c) and D5.2(d) are checked as follows:

$$\begin{aligned}
b_e &= 2t + 0.63 \le b\\
&= 2\left(\tfrac{1}{2} \text{ in.}\right) + 0.63 \le 1.61 \text{ in.}\\
&= 1.63 \text{ in.} > 1.61 \text{ in.}
\end{aligned}$$

Therefore, use $b_e = 1.61$ in.

$$a \geq 1.33b_e$$
$$2.25\text{ in.} > 1.33(1.61\text{ in.})$$
$$2.25\text{ in.} > 2.14\text{ in.} \qquad \textbf{o.k.}$$

$$w \geq 2b_e + d$$
$$4.25\text{ in.} > 2(1.61\text{ in.}) + 1.00\text{ in.}$$
$$4.25\text{ in.} > 4.22\text{ in.} \qquad \textbf{o.k.}$$

$$c \geq a$$
$$2.50\text{ in.} > 2.25\text{ in.} \qquad \textbf{o.k.}$$

From Chapter 2 of ASCE/SEI 7, the required tensile strength is:

LRFD	ASD
$P_u = 1.2(4\text{ kips}) + 1.6(12\text{ kips})$ $= 24.0\text{ kips}$	$P_a = 4\text{ kips} + 12\text{ kips}$ $= 16.0\text{ kips}$

From AISC *Specification* Section D5.1, the available tensile strength is the lower value determined according to the limit states of tensile rupture, shear rupture, bearing and yielding.

Tensile Rupture

Calculate the available tensile rupture strength on the effective net area.

$$P_n = F_u\,(2tb_e) \qquad \text{(}\textit{Spec}\text{. Eq. D5-1)}$$
$$= (58\text{ ksi})(2)(½\text{ in.})(1.61\text{ in.})$$
$$= 93.4\text{ kips}$$

From AISC *Specification* Section D5.1, the available tensile rupture strength is:

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(93.4\text{ kips})$ $= 70.1\text{ kips}$	$\Omega_t = 2.00$ $\dfrac{P_n}{\Omega_t} = \dfrac{93.4\text{ kips}}{2.00}$ $= 46.7\text{ kips}$

Shear Rupture

From AISC *Specification* Section D5.1, the area on the shear failure path is:

$$A_{sf} = 2t\left(a + \frac{d}{2}\right)$$
$$= 2(½\text{ in.})\left[2.25\text{ in.} + \left(\frac{1.00\text{ in.}}{2}\right)\right]$$
$$= 2.75\text{ in.}^2$$

$$P_n = 0.6F_u A_{sf} \qquad \text{(}\textit{Spec}\text{. Eq. D5-2)}$$
$$= 0.6(58\text{ ksi})(2.75\text{ in.}^2)$$
$$= 95.7\text{ kips}$$

From AISC *Specification* Section D5.1, the available shear rupture strength is:

LRFD	ASD
$\phi_{sf} = 0.75$ $\phi_{sf} P_n = 0.75(95.7\text{ kips})$ $= 71.8\text{ kips}$	$\Omega_{sf} = 2.00$ $\dfrac{P_n}{\Omega_{sf}} = \dfrac{95.7\text{ kips}}{2.00}$ $= 47.9\text{ kips}$

Bearing

Determine the available bearing strength using AISC *Specification* Section J7.

$$A_{pb} = td$$
$$= (\tfrac{1}{2}\text{ in.})(1.00\text{ in.})$$
$$= 0.500\text{ in.}^2$$

$$R_n = 1.8F_y A_{pb} \qquad \text{(}\textit{Spec}\text{. Eq. J7-1)}$$
$$= 1.8(36\text{ ksi})(0.500\text{ in.}^2)$$
$$= 32.4\text{ kips}$$

From AISC *Specification* Section J7, the available bearing strength is:

LRFD	ASD
$\phi = 0.75$ $\phi P_n = 0.75(32.4\text{ kips})$ $= 24.3\text{ kips}$	$\Omega = 2.00$ $\dfrac{P_n}{\Omega} = \dfrac{32.4\text{ kips}}{2.00}$ $= 16.2\text{ kips}$

Tensile Yielding

Determine the available tensile yielding strength using AISC *Specification* Section D2(a).

$$A_g = wt$$
$$= (4.25\text{ in.})(\tfrac{1}{2}\text{ in.})$$
$$= 2.13\text{ in.}^2$$

$$P_n = F_y A_g \qquad \text{(}\textit{Spec}\text{. Eq. D2-1)}$$
$$= (36\text{ ksi})(2.13\text{ in.}^2)$$
$$= 76.7\text{ kips}$$

From AISC *Specification* Section D2, the available tensile yielding strength is:

LRFD	ASD
$\phi_t = 0.90$ $\phi_t P_n = 0.90(76.7 \text{ kips})$ $= 69.0 \text{ kips}$	$\Omega_t = 1.67$ $\dfrac{P_n}{\Omega_t} = \dfrac{76.7 \text{ kips}}{1.67}$ $= 45.9 \text{ kips}$

The available tensile strength is governed by the bearing strength limit state.

LRFD	ASD
$\phi P_n = 24.3 \text{ kips} > 24.0 \text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega} = 16.2 \text{ kips} > 16.0 \text{ kips}$ **o.k.**

EXAMPLE D.8 EYEBAR TENSION MEMBER

Given:

A ⅝-in.-thick, ASTM A36 eyebar member as shown in Figure D.8, carries a dead load of 25 kips and a live load of 15 kips in tension. The pin diameter, *d*, is 3 in. Verify the member tensile strength.

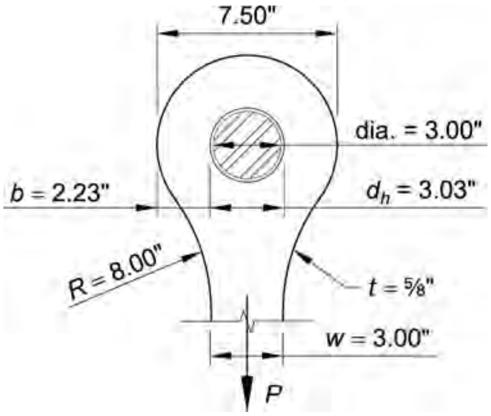


Fig. D.8-1. Connection geometry for Example D.8.

Solution:

From AISC *Manual* Table 2-5, the material properties are as follows:

Plate
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

The geometric properties of the eyebar are as follows:

$R = 8.00$ in.
$b = 2.23$ in.
$d = 3.00$ in.
$d_h = 3.03$ in.
$d_{head} = 7.50$ in.
$t =$ ⅝ in.
$w = 3.00$ in.

Check the dimensional requirement using AISC *Specification* Section D6.1.

$$w \le 8t$$
$$3.00 \text{ in.} < 8\left(\tfrac{5}{8} \text{ in.}\right)$$
$$3.00 \text{ in.} < 5.00 \text{ in.} \quad \textbf{o.k.}$$

Check the dimensional requirements using AISC *Specification* Section D6.2.

$$t \ge \tfrac{1}{2} \text{ in.}$$
$$\tfrac{5}{8} \text{ in.} > \tfrac{1}{2} \text{ in.} \quad \textbf{o.k.}$$

$$d \geq \frac{7}{8}w$$

$$3.00\text{ in.} > \frac{7}{8}(3.00\text{ in.})$$

$$3.00\text{ in.} > 2.63\text{ in.} \quad \textbf{o.k.}$$

$$d_h \leq d + \tfrac{1}{32}\text{ in.}$$

$$3.03\text{ in.} = 3.00\text{ in.} + \tfrac{1}{32}\text{ in.}$$

$$3.03\text{ in.} = 3.03\text{ in.} \quad \textbf{o.k.}$$

$$R \geq d_{head}$$

$$8.00\text{ in.} > 7.50\text{ in.} \quad \textbf{o.k.}$$

$$\frac{2}{3}w < b \leq \frac{3}{4}w$$

$$\frac{2}{3}(3.00\text{ in.}) < 2.23\text{ in.} < \frac{3}{4}(3.00\text{ in.})$$

$$2.00\text{ in.} < 2.23\text{ in.} < 2.25\text{ in.} \quad \textbf{o.k.}$$

From Chapter 2 of ASCE/SEI 7, the required tensile strength is:

LRFD	ASD
$P_u = 1.2(25\text{ kips}) + 1.6(15\text{ kips})$ $= 54.0\text{ kips}$	$P_a = 25\text{ kips} + 15\text{ kips}$ $= 40.0\text{ kips}$

Tensile Yielding

Determine the available tensile yielding strength using AISC *Specification* Section D2 at the eyebar body (at *w*).

$$\begin{aligned} A_g &= wt \\ &= (3.00\text{ in.})(\tfrac{5}{8}\text{ in.}) \\ &= 1.88\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} P_n &= F_y A_g \\ &= (36\text{ ksi})(1.88\text{ in.}^2) \\ &= 67.7\text{ kips} \end{aligned}$$ (*Spec*. Eq. D2-1)

The available tensile yielding strength is:

LRFD	ASD
$\phi_t = 0.90$ $\phi_t P_n = 0.90(67.7\text{ kips})$ $= 60.9\text{ kips} > 54.0\text{ kips}$ **o.k.**	$\Omega_t = 1.67$ $\frac{P_n}{\Omega_t} = \frac{67.7\text{ kips}}{1.67}$ $= 40.5\text{ kips} > 40.0\text{ kips}$ **o.k.**

The eyebar tension member available strength is governed by the tensile yielding limit state.

Note: The eyebar detailing limitations ensure that the tensile yielding limit state at the eyebar body will control the strength of the eyebar itself. The pin should also be checked for shear yielding, and, if the material strength is less than that of the eyebar, the bearing limit state should also be checked.

EXAMPLE D.9 PLATE WITH STAGGERED BOLTS

Given:

Compute A_n and A_e for a 14-in.-wide and ½-in.-thick plate subject to tensile loading with staggered holes as shown in Figure D.9-1.

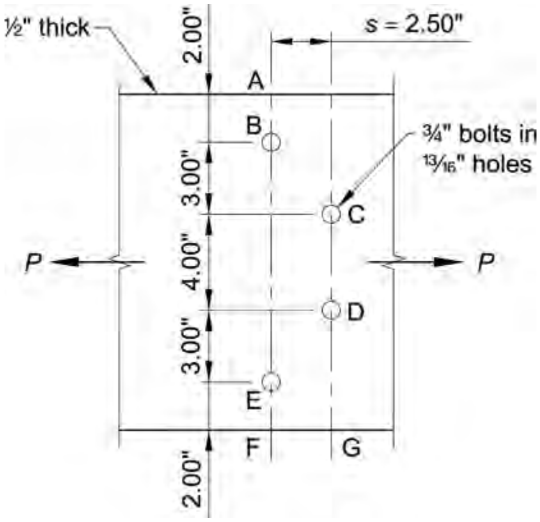


Fig. D.9-1. Connection geometry for Example D.9.

Solution:

Calculate the net hole diameter using AISC *Specification* Section B4.3b.

$$\begin{aligned} d_{net} &= d_h + \tfrac{1}{16} \text{ in.} \\ &= \tfrac{13}{16} \text{ in.} + \tfrac{1}{16} \text{ in.} \\ &= 0.875 \text{ in.} \end{aligned}$$

Compute the net width for all possible paths across the plate. Because of symmetry, many of the net widths are identical and need not be calculated.

$$w = 14.0 \text{ in.} - \Sigma d_{net} + \Sigma \frac{s^2}{4g} \quad \text{from AISC } \textit{Specification} \text{ Section B4.3b.}$$

Line A-B-E-F:

$$\begin{aligned} w &= 14.0 \text{ in.} - 2(0.875 \text{ in.}) \\ &= 12.3 \text{ in.} \end{aligned}$$

Line A-B-C-D-E-F:

$$\begin{aligned} w &= 14.0 \text{ in.} - 4(0.875 \text{ in.}) + \frac{(2.50 \text{ in.})^2}{4(3.00 \text{ in.})} + \frac{(2.50 \text{ in.})^2}{4(3.00 \text{ in.})} \\ &= 11.5 \text{ in.} \end{aligned}$$

Line A-B-C-D-G:

$$
\begin{aligned}
w &= 14.0\text{ in.} - 3(0.875\text{ in.}) + \frac{(2.50\text{ in.})^2}{4(3.00\text{ in.})} \\
&= 11.9\text{ in.}
\end{aligned}
$$

Line A-B-D-E-F:

$$
\begin{aligned}
w &= 14.0\text{ in.} - 3(0.875\text{ in.}) + \frac{(2.50\text{ in.})^2}{4(7.00\text{ in.})} + \frac{(2.50\text{ in.})^2}{4(3.00\text{ in.})} \\
&= 12.1\text{ in.}
\end{aligned}
$$

Line A-B-C-D-E-F controls the width, w, therefore:

$$
\begin{aligned}
A_n &= wt \\
&= (11.5\text{ in.})(\tfrac{1}{2}\text{ in.}) \\
&= 5.75\text{ in.}^2
\end{aligned}
$$

Calculate U.

From AISC *Specification* Table D3.1 Case 1, because tension load is transmitted to all elements by the fasteners,

$$U = 1.0$$

$$
\begin{aligned}
A_e &= A_n U && \text{(}\textit{Spec}\text{. Eq. D3-1)} \\
&= (5.75\text{ in.}^2)(1.0) \\
&= 5.75\text{ in.}^2
\end{aligned}
$$

Chapter E
Design of Members for Compression

This chapter covers the design of compression members, the most common of which are columns. The AISC *Manual* includes design tables for the following compression member types in their most commonly available grades:

- W-shapes and HP-shapes
- Rectangular, square and round HSS
- Pipes
- WT-shapes
- Double angles
- Single angles

LRFD and ASD information is presented side-by-side for quick selection, design or verification. All of the tables account for the reduced strength of sections with slender elements.

The design and selection method for both LRFD and ASD is similar to that of previous editions of the AISC *Specification*, and will provide similar designs. In this AISC *Specification*, LRFD and ASD will provide identical designs when the live load is approximately three times the dead load.

The design of built-up shapes with slender elements can be tedious and time consuming, and it is recommended that standard rolled shapes be used whenever possible.

E1. GENERAL PROVISIONS

The design compressive strength, $\phi_c P_n$, and the allowable compressive strength, P_n/Ω_c, are determined as follows:

P_n = nominal compressive strength is the lowest value obtained based on the applicable limit states of flexural buckling, torsional buckling, and flexural-torsional buckling, kips

$\phi_c = 0.90$ (LRFD) $\qquad \Omega_c = 1.67$ (ASD)

Because the critical stress, F_{cr}, is used extensively in calculations for compression members, it has been tabulated in AISC *Manual* Table 4-14 for all of the common steel yield strengths.

E2. EFFECTIVE LENGTH

In the AISC *Specification*, there is no limit on slenderness, L_c/r. Per the User Note in AISC *Specification* Section E2, it is recommended that L_c/r not exceed 200, as a practical limit based on professional judgment and construction economics.

Although there is no restriction on the unbraced length of columns, the tables of the AISC *Manual* are stopped at common or practical lengths for ordinary usage. For example, a double L3×3×¼, with a ⅜-in. separation has an r_y of 1.38 in. At a L_c/r of 200, this strut would be 23 ft long. This is thought to be a reasonable limit based on fabrication and handling requirements.

Throughout the AISC *Manual*, shapes that contain slender elements for compression when supplied in their most common material grade are footnoted with the letter “c.” For example, see a W14×22^c.

E3. FLEXURAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

Nonslender-element compression members, including nonslender built-up I-shaped columns and nonslender HSS columns, are governed by these provisions. The general design curve for critical stress versus L_c/r is shown in Figure E-1.

The term L_c is used throughout this chapter to describe the length between points that are braced against lateral and/or rotational displacement.

E4. TORSIONAL AND FLEXURAL-TORSIONAL BUCKLING OF SINGLE ANGLES AND MEMBERS WITHOUT SLENDER ELEMENTS

This section is most commonly applicable to double angles and WT sections, which are singly symmetric shapes subject to torsional and flexural-torsional buckling. The available strengths in axial compression of these shapes are tabulated in AISC *Manual* Part 4 and examples on the use of these tables have been included in this chapter for the shapes.

E5. SINGLE-ANGLE COMPRESSION MEMBERS

The available strength of single-angle compression members is tabulated in AISC *Manual* Part 4.

E6. BUILT-UP MEMBERS

The available strengths in axial compression for built-up double angles with intermediate connectors are tabulated in AISC *Manual* Part 4. There are no tables for other built-up shapes in the AISC *Manual*, due to the number of possible geometries.

E7. MEMBERS WITH SLENDER ELEMENTS

The design of these members is similar to members without slender elements except that a reduced effective area is used in lieu of the gross cross-sectional area.

The tables of AISC *Manual* Part 4 incorporate the appropriate reductions in available strength to account for slender elements.

Design examples have been included in this Chapter for built-up I-shaped members with slender webs and slender flanges**.** Examples have also been included for a double angle, WT and an HSS with slender elements.

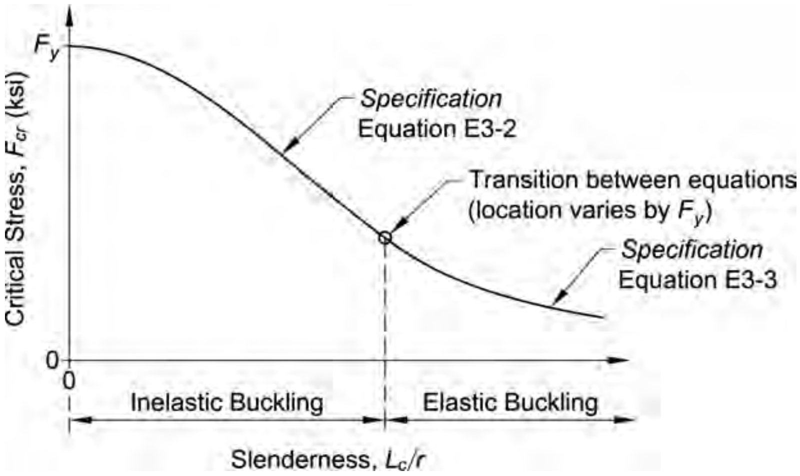


Fig. E-1. Standard column curve.

Table E-1 Limiting Values of L_c/r and F_e		
F_y, ksi	**Limiting L_c/r**	**F_e, ksi**
36	134	15.9
50	113	22.4
65	99.5	28.9
70	95.9	31.1

EXAMPLE E.1A W-SHAPE COLUMN DESIGN WITH PINNED ENDS

Given:

Select a W-shape column to carry the loading as shown in Figure E.1A. The column is pinned top and bottom in both axes. Limit the column size to a nominal 14-in. shape. A column is selected for both ASTM A992 and ASTM A913 Grade 65 material.

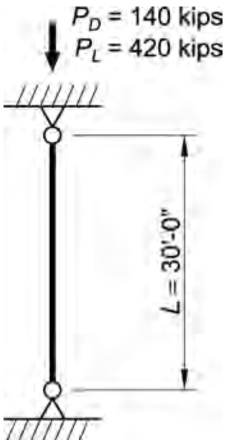


Fig. E.1A. Column loading and bracing.

Solution:

Note that ASTM A913 Grade 70 might also be used in this design. The requirement for higher preheat when welding and the need to use 90-ksi filler metals for complete-joint-penetration (CJP) welds to other 70-ksi pieces offset the advantage of the lighter column and should be considered in the selection of which grade to use.

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi

ASTM A913 Grade 65
F_y = 65 ksi

From ASCE/SEI 7, Chapter 2, the required compressive strength is:

LRFD	ASD
$P_u = 1.2(140\text{ kips}) + 1.6(420\text{ kips})$ $= 840\text{ kips}$	$P_a = 140\text{ kips} + 420\text{ kips}$ $= 560\text{ kips}$

Column Selection—ASTM A992

From AISC *Specification* Commentary Table C-A-7.1, for a pinned-pinned condition, $K_x = K_y = 1.0$. The effective length is:

$$\begin{aligned} L_c &= K_x L_x \\ &= K_y L_y \\ &= 1.0(30 \text{ ft}) \\ &= 30.0 \text{ ft} \end{aligned}$$

Because the unbraced length is the same in both the *x-x* and *y-y* directions and r_x exceeds r_y for all W-shapes, *y-y* axis bucking will govern.

Enter AISC *Manual* Table 4-1a with an effective length, L_c, of 30 ft, and proceed across the table until reaching the least weight shape with an available strength that equals or exceeds the required strength. Select a W14×132.

From AISC *Manual* Table 4-1a, the available strength for a *y-y* axis effective length of 30 ft is:

LRFD	ASD
$\phi_c P_n = 893 \text{ kips} > 840 \text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_c} = 594 \text{ kips} > 560 \text{ kips}$ **o.k.**

Column Selection–ASTM A913 Grade 65

Enter AISC *Manual* Table 4-1b with an effective length, L_c, of 30 ft, and proceed across the table until reaching the least weight shape with an available strength that equals or exceeds the required strength. Select a W14×120.

From AISC *Manual* Table 4-1b, the available strength for a *y-y* axis effective length of 30 ft is:

LRFD	ASD
$\phi_c P_n = 856 \text{ kips} > 840 \text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_c} = 569 \text{ kips} > 560 \text{ kips}$ **o.k.**

EXAMPLE E.1B W-SHAPE COLUMN DESIGN WITH INTERMEDIATE BRACING

Given:

Verify a W14×90 is adequate to carry the loading as shown in Figure E.1B. The column is pinned top and bottom in both axes and braced at the midpoint about the *y-y* axis and torsionally. The column is verified for both ASTM A992 and ASTM A913 Grade 65 material.

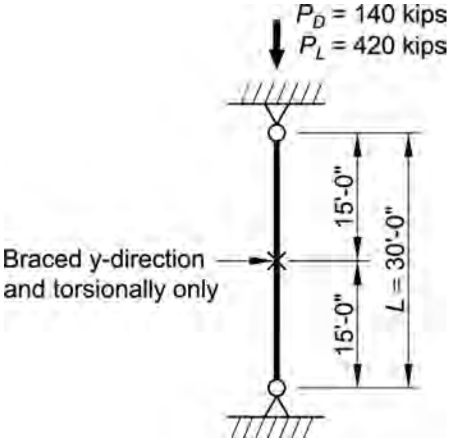


Fig. E.1B. Column loading and bracing.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi

ASTM A913 Grade 65
F_y = 65 ksi

From ASCE/SEI 7, Chapter 2, the required compressive strength is:

LRFD	ASD
$P_u = 1.2(140\text{ kips}) + 1.6(420\text{ kips})$ $= 840\text{ kips}$	$P_a = 140\text{ kips} + 420\text{ kips}$ $= 560\text{ kips}$

From AISC *Specification* Commentary Table C-A-7.1, for a pinned-pinned condition, $K_x = K_y = 1.0$. The effective length about the *y*-y axis is:

$$\begin{aligned} L_{cy} &= K_y L_y \\ &= 1.0(15\text{ ft}) \\ &= 15.0\text{ ft} \end{aligned}$$

The values tabulated in AISC *Manual* Tables 4-1a, 4-1b and 4-1c are provided for buckling in the *y-y* direction. To determine the buckling strength in the *x-x* axis, an equivalent effective length for the *y-y* axis is determined using the r_x/r_y ratio provided at the the bottom of these tables. For a W14×90, $r_x/r_y = 1.66$, and the equivalent *y-y* axis effective length for *x-x* axis buckling is computed as:

$$
\begin{aligned}
L_{cx} &= K_x L_x \\
&= 1.0(30 \text{ ft}) \\
&= 30.0 \text{ ft}
\end{aligned}
$$

$$
\begin{aligned}
L_{cy\,eq} &= \frac{L_{cx}}{r_x/r_y} \qquad \text{(}\textit{Manual}\text{ Eq. 4-1)} \\
&= \frac{30.0 \text{ ft}}{1.66} \\
&= 18.1 \text{ ft}
\end{aligned}
$$

Because 18.1 ft > 15.0 ft, the available compressive strength is governed by the *x-x* axis flexural buckling limit state.

Available Compressive Strength—ASTM A992

The available strength of a W14×90 is determined using AISC *Manual* Table 4-1a, conservatively using an unbraced length of L_c = 19.0 ft.

LRFD	ASD
$\phi_c P_n = 903 \text{ kips} > 840 \text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_c} = 601 \text{ kips} > 560 \text{ kips}$ **o.k.**

Available Compressive Strength—ASTM 913 Grade 65

The available strength of a W14×90 is determined using AISC *Manual* Table 4-1b, conservatively using an unbraced length of L_c = 19.0 ft.

LRFD	ASD
$\phi_c P_n = 1{,}080 \text{ kips} > 840 \text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_c} = 719 \text{ kips} > 560 \text{ kips}$ **o.k.**

The available strengths of the columns described in Examples E.1A and E.1B are easily selected directly from the AISC *Manual* Tables. The available strengths can also be determined as shown in the following Examples E.1C and E.1D.

EXAMPLE E.1C W-SHAPE AVAILABLE STRENGTH CALCULATION

Given:

Calculate the available strength of the column sizes selected in Example E.1A with unbraced lengths of 30 ft in both axes. The material properties and loads are as given in Example E.1A.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi

ASTM A913 Grade 65
F_y = 65 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W14×120
A_g = 35.3 in.^{2}
r_x = 6.24 in.
r_y = 3.74 in.

W14×132
A_g = 38.8 in.^{2}
r_x = 6.28 in.
r_y = 3.76 in.

Column Compressive Strength—ASTM A992

Slenderness Check

From AISC *Specification* Commentary Table C-A-7.1, for a pinned-pinned condition, $K_x = K_y = 1.0$. The effective length about the *y*-y axis is:

$$\begin{aligned} L_{cy} &= K_y L_y \\ &= 1.0(30 \text{ ft}) \\ &= 30.0 \text{ ft} \end{aligned}$$

Because the unbraced length for the W14×132 column is the same for both axes, the *y*-*y* axis will govern.

$$\begin{aligned} \frac{L_{cy}}{r_y} &= \frac{(30.0 \text{ ft})(12 \text{ in./ft})}{3.76 \text{ in.}} \\ &= 95.7 \end{aligned}$$

Critical Stress

For F_y = 50 ksi, the available critical stresses, $\phi_c F_{cr}$ and F_{cr}/Ω_c for L_c/r = 95.7 are interpolated from AISC *Manual* Table 4-14 as follows. The available critical stress can also be determined as shown in Example E.1D.

LRFD	ASD
$\phi_c F_{cr} = 23.0 \text{ ksi}$	$\dfrac{F_{cr}}{\Omega_c} = 15.4 \text{ ksi}$

From AISC *Specification* Equation E3-1, the available compressive strength of the W14×132 column is:

LRFD	ASD
$\phi_c P_n = (\phi_c F_{cr}) A_g$ $= (23.0 \text{ ksi})(38.8 \text{ in.}^2)$ $= 892 \text{ kips} > 840 \text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_c} = \left(\dfrac{F_{cr}}{\Omega_c}\right) A_g$ $= (15.4 \text{ ksi})(38.8 \text{ in.}^2)$ $= 598 \text{ kips} > 560 \text{ kips}$ **o.k.**

Column Compressive Strength—ASTM A913 Grade 65

Slenderness Check

From AISC *Specification* Commentary Table C-A-7.1, for a pinned-pinned condition, $K_x = K_y = 1.0$. The effective length about the *y*-y axis is:

$$\begin{aligned} L_{cy} &= K_y L_y \\ &= 1.0(30 \text{ ft}) \\ &= 30.0 \text{ ft} \end{aligned}$$

Because the unbraced length for the W14×120 column is the same for both axes, the *y*-*y* axis will govern.

$$\begin{aligned} \frac{L_{cy}}{r_y} &= \frac{(30.0 \text{ ft})(12 \text{ in./ft})}{3.74 \text{ in.}} \\ &= 96.3 \end{aligned}$$

Critical Stress

For F_y = 65 ksi, the available critical stresses, $\phi_c F_{cr}$ and F_{cr}/Ω_c for L_c/r = 96.3 are interpolated from AISC *Manual* Table 4-14 as follows. The available critical stress can also be determined as shown in Example E.1D.

LRFD	ASD
$\phi_c F_{cr} = 24.3 \text{ ksi}$	$\dfrac{F_{cr}}{\Omega_c} = 16.1 \text{ ksi}$

From AISC *Specification* Equation E3-1, the available compressive strength of the W14×120 column is:

LRFD	ASD
$\phi_c P_n = (\phi_c F_{cr}) A_g$ $= (24.3 \text{ ksi})(35.3 \text{ in.}^2)$ $= 858 \text{ kips} > 840 \text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_c} = \left(\dfrac{F_{cr}}{\Omega_c}\right) A_g$ $= (16.1 \text{ ksi})(35.3 \text{ in.}^2)$ $= 568 \text{ kips} > 560 \text{ kips}$ **o.k.**

Note that the calculated values are approximately equal to the tabulated values.

EXAMPLE E.1D W-SHAPE AVAILABLE STRENGTH CALCULATION

Given:

Calculate the available strength of a W14×90 with a *x-x* axis unbraced length of 30 ft and *y-y* axis and torsional unbraced lengths of 15 ft. The material properties and loads are as given in Example E.1A.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi

ASTM A913 Grade 65
F_y = 65 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W14×90
A_g = 26.5 in.^{2}
r_x = 6.14 in.
r_y = 3.70 in.

$$\frac{b_f}{2t_f} = 10.2$$

$$\frac{h}{t_w} = 25.9$$

Slenderness Check

From AISC *Specification* Commentary Table C-A-7.1, for a pinned-pinned condition, $K_x = K_y = 1.0$.

$$\begin{aligned} L_{cx} &= K_x L_x \\ &= 1.0(30 \text{ ft}) \\ &= 30.0 \text{ ft} \end{aligned}$$

$$\begin{aligned} \frac{L_{cx}}{r_x} &= \frac{(30.0 \text{ ft})(12 \text{ in./ft})}{6.14 \text{ in.}} \\ &= 58.6 \quad \textbf{governs} \end{aligned}$$

$$\begin{aligned} L_{cy} &= K_y L_y \\ &= 1.0(15 \text{ ft}) \\ &= 15.0 \text{ ft} \end{aligned}$$

$$\begin{aligned} \frac{L_{cy}}{r_y} &= \frac{(15.0 \text{ ft})(12 \text{ in./ft})}{3.70 \text{ in.}} \\ &= 48.6 \end{aligned}$$

Column Compressive Strength—ASTM A992

Width-to-Thickness Ratio

The width-to-thickness ratio of the flanges of the W14×90 is:

$$\frac{b_f}{2t_f} = 10.2$$

From AISC *Specification* Table B4.1a, Case 1, the limiting width-to-thickness ratio of the flanges is:

$$\begin{aligned} 0.56\sqrt{\frac{E}{F_y}} &= 0.56\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} \\ &= 13.5 > 10.2;\ \text{therefore, the flanges are nonslender} \end{aligned}$$

The width-to-thickness ratio of the web of the W14×90 is:

$$\frac{h}{t_w} = 25.9$$

From AISC *Specification* Table B4.1a, Case 5, the limiting width-to-thickness ratio of the web is:

$$\begin{aligned} 1.49\sqrt{\frac{E}{F_y}} &= 1.49\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} \\ &= 35.9 > 25.9;\ \text{therefore, the web is nonslender} \end{aligned}$$

Because the web and flanges are nonslender, the limit state of local buckling does not apply.

Critical Stresses

The available critical stresses may be interpolated from AISC *Manual* Table 4-14 or calculated directly as follows.

Calculate the elastic critical buckling stress, F_e, according to AISC *Specification* Section E3. As noted in AISC *Specification* Commentary Section E4, torsional buckling of symmetric shapes is a failure mode usually not considered in the design of hot-rolled columns. This failure mode generally does not govern unless the section is manufactured from relatively thin plates or a torsional unbraced length significantly larger than the y-y axis flexural unbraced length is present.

$$\begin{aligned} F_e &= \frac{\pi^2 E}{\left(\frac{L_c}{r}\right)^2} \qquad \text{(\textit{Spec}. Eq. E3-4)} \\ &= \frac{\pi^2 (29{,}000\text{ ksi})}{(58.6)^2} \\ &= 83.3\text{ ksi} \end{aligned}$$

Calculate the flexural buckling stress, F_{cr}.

$$\begin{aligned} 4.71\sqrt{\frac{E}{F_y}} &= 4.71\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} \\ &= 113 \end{aligned}$$

Because $\frac{L_c}{r} = 58.6 < 113$,

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y \qquad (\textit{Spec}.\ \text{Eq. E3-2})$$

$$= \left(0.658^{\frac{50\text{ ksi}}{83.3\text{ ksi}}}\right)(50\text{ ksi})$$

$$= 38.9\text{ ksi}$$

Nominal Compressive Strength

$$P_n = F_{cr}A_g \qquad (\textit{Spec}.\ \text{Eq. E3-1})$$

$$= (38.9\text{ ksi})\left(26.5\text{ in.}^2\right)$$

$$= 1{,}030\text{ kips}$$

From AISC *Specification* Section E1, the available compressive strength is:

LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(1{,}030\text{ kips})$ $= 927\text{ kips} > 840\text{ kips}$ **o.k.**	$\Omega_c = 1.67$ $\frac{P_n}{\Omega_c} = \frac{1{,}030\text{ kips}}{1.67}$ $= 617\text{ kips} > 560\text{ kips}$ **o.k.**

Column Compressive Strength—ASTM A913 Grade 65

Width-to-Thickness Ratio

The width-to-thickness ratio of the flanges of the W14×90 is:

$$\frac{b_f}{2t_f} = 10.2$$

From AISC *Specification* Table B4.1a, Case 1, the limiting width-to-thickness ratio of the flanges is:

$$0.56\sqrt{\frac{E}{F_y}} = 0.56\sqrt{\frac{29{,}000\text{ ksi}}{65\text{ ksi}}}$$

$$= 11.8 > 10.2;\ \text{therefore, the flanges are nonslender}$$

The width-to-thickness ratio of the web of the W14×90 is:

$$\frac{h}{t_w} = 25.9$$

From AISC *Specification* Table B4.1a, Case 5, the limiting width-to-thickness ratio of the web is:

$$1.49\sqrt{\frac{E}{F_y}} = 1.49\sqrt{\frac{29{,}000\text{ ksi}}{65\text{ ksi}}}$$
$$= 31.5 > 25.9;\ \text{therefore, the web is nonslender}$$

Because the web and flanges are nonslender, the limit state of local buckling does not apply.

Critical Stress

$$F_e = 83.3\text{ ksi (calculated previously)}$$

Calculate the flexural buckling stress, F_{cr}.

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29{,}000\text{ ksi}}{65\text{ ksi}}}$$
$$= 99.5$$

Because $\dfrac{L_c}{r} = 58.6 < 99.5$,

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y \qquad (\textit{Spec}.\text{ Eq. E3-2})$$
$$= \left(0.658^{\frac{65\text{ ksi}}{83.3\text{ ksi}}}\right)(65\text{ ksi})$$
$$= 46.9\text{ ksi}$$

Nominal Compressive Strength

$$P_n = F_{cr}A_g \qquad (\textit{Spec}.\text{ Eq. E3-1})$$
$$= (46.9\text{ ksi})(26.5\text{ in.}^2)$$
$$= 1{,}240\text{ kips}$$

From AISC *Specification* Section E1, the available compressive strength is:

LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(1{,}240\text{ kips})$ $= 1{,}120\text{ kips} > 840\text{ kips}$ **o.k.**	$\Omega_c = 1.67$ $\dfrac{P_n}{\Omega_c} = \dfrac{1{,}240\text{ kips}}{1.67}$ $= 743\text{ kips} > 560\text{ kips}$ **o.k.**

EXAMPLE E.2 BUILT-UP COLUMN WITH A SLENDER WEB

Given:

Verify that a built-up, ASTM A572 Grade 50 column with PL1 in. × 8 in. flanges and a PL¼ in. × 15 in. web, as shown in Figure E2-1, is sufficient to carry a dead load of 70 kips and live load of 210 kips in axial compression. The column's unbraced length is 15 ft and the ends are pinned in both axes.

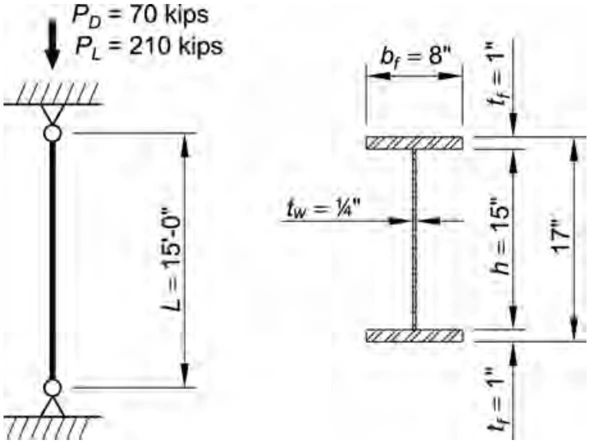


Fig. E.2-1. Column geometry for Example E.2.

Solution:

From AISC *Manual* Table 2-5, the material properties are as follows:

Built-Up Column
ASTM A572 Grade 50
F_y = 50 ksi
F_u = 65 ksi

The geometric properties are as follows:

Built-Up Column
d = 17.0 in.
b_f = 8.00 in.
t_f = 1.00 in.
h = 15.0 in.
t_w = ¼ in.

From ASCE/SEI 7, Chapter 2, the required compressive strength is:

LRFD	ASD
$P_u = 1.2(70\text{ kips}) + 1.6(210\text{ kips})$ $= 420\text{ kips}$	$P_a = 70\text{ kips} + 210\text{ kips}$ $= 280\text{ kips}$

Built-Up Section Properties (ignoring fillet welds)

$$\begin{aligned}A_g &= 2b_f t_f + ht_w\\ &= 2(8.00\text{ in.})(1.00\text{ in.}) + (15.0\text{ in.})(\tfrac{1}{4}\text{ in.})\\ &= 19.8\text{ in.}^2\end{aligned}$$

$$\begin{aligned}I_y &= \sum\frac{bh^3}{12}\\ &= 2\left[\frac{(1.00\text{ in.})(8.00\text{ in.})^3}{12}\right] + \frac{(15.0\text{ in.})(\tfrac{1}{4}\text{ in.})^3}{12}\\ &= 85.4\text{ in.}^4\end{aligned}$$

$$\begin{aligned}r_y &= \sqrt{\frac{I_y}{A}}\\ &= \sqrt{\frac{85.4\text{ in.}^4}{19.8\text{ in.}^2}}\\ &= 2.08\text{ in.}\end{aligned}$$

$$\begin{aligned}I_x &= \sum Ad^2 + \sum\frac{bh^3}{12}\\ &= 2\left[(8.00\text{ in.}^2)(8.00\text{ in.})^2\right] + \frac{(\tfrac{1}{4}\text{ in.})(15.0\text{ in.})^3}{12} + 2\left[\frac{(8.00\text{ in.})(1.00\text{ in.})^3}{12}\right]\\ &= 1{,}100\text{ in.}^4\end{aligned}$$

Elastic Flexural Buckling Stress

From AISC *Specification* Commentary Table C-A-7.1, for a pinned-pinned condition, K_y = 1.0.

Because the unbraced length is the same for both axes, the *y-y* axis will govern by inspection. With $L_{cy} = K_yL_y$ = 1.0(15 ft) = 15.0 ft:

$$\begin{aligned}\frac{L_{cy}}{r_y} &= \frac{(15.0\text{ ft})(12\text{ in./ft})}{2.08\text{ in.}}\\ &= 86.5\end{aligned}$$

$$\begin{aligned}F_e &= \frac{\pi^2 E}{\left(\dfrac{L_{cy}}{r_y}\right)^2} \qquad \text{(from } \textit{Spec}\text{. Eq. E3-4)}\\ &= \frac{\pi^2(29{,}000\text{ ksi})}{(86.5)^2}\\ &= 38.3\text{ ksi}\end{aligned}$$

Elastic Critical Torsional Buckling Stress

Note: Torsional buckling generally will not govern for doubly symmetric members if $L_{cy} \geq L_{cz}$; however, the check is included here to illustrate the calculation.

From the User Note in AISC *Specification* Section E4:

$$
\begin{aligned}
C_w &= \frac{I_y h_o^2}{4} \\
&= \frac{\left(85.4\text{ in.}^4\right)\left(16.0\text{ in.}\right)^2}{4} \\
&= 5{,}470\text{ in.}^6
\end{aligned}
$$

From AISC Design Guide 9, Equation 3.4:

$$
\begin{aligned}
J &= \sum \frac{bt^3}{3} \\
&= 2\left[\frac{\left(8.00\text{ in.}\right)\left(1.00\text{ in.}\right)^3}{3}\right] + \frac{\left(15.0\text{ in.}\right)\left(\tfrac{1}{4}\text{ in.}\right)^3}{3} \\
&= 5.41\text{ in.}^4
\end{aligned}
$$

$$
F_e = \left(\frac{\pi^2 E C_w}{L_{cz}^{\;2}} + GJ\right)\frac{1}{I_x + I_y} \qquad (\textit{Spec}.\text{ Eq. E4-2})
$$

$$
\begin{aligned}
&= \left\{\frac{\pi^2\left(29{,}000\text{ ksi}\right)\left(5{,}470\text{ in.}^6\right)}{\left[1.0\left(15\text{ ft}\right)\left(12\text{ in./ft}\right)\right]^2} + \left(11{,}200\text{ ksi}\right)\left(5.41\text{in.}^4\right)\right\}\left(\frac{1}{1{,}100\text{ in.}^4 + 85.4\text{ in.}^4}\right) \\
&= 91.9\text{ ksi} > 38.3\text{ ksi}
\end{aligned}
$$

Therefore, the flexural buckling limit state controls.

Use F_e = 38.3 ksi.

Flexural Buckling Stress

$$
\begin{aligned}
\frac{F_y}{F_e} &= \frac{50\text{ ksi}}{38.3\text{ ksi}} \\
&= 1.31
\end{aligned}
$$

Because $\frac{F_y}{F_e} < 2.25$,

$$
F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y \qquad (\textit{Spec}.\text{ Eq. E3-2})
$$

$$
\begin{aligned}
&= \left(0.658^{1.31}\right)\left(50\text{ ksi}\right) \\
&= 28.9\text{ ksi}
\end{aligned}
$$

Slenderness

Check for slender flanges using AISC *Specification* Table B4.1a.

Calculate k_c using AISC *Specification* Table B4.1a, note [a].

$$\begin{aligned} k_c &= \frac{4}{\sqrt{h/t_w}} \\ &= \frac{4}{\sqrt{\dfrac{15.0\text{ in.}}{\tfrac{1}{4}\text{ in.}}}} \\ &= 0.516,\ \text{which is between 0.35 and 0.76.} \end{aligned}$$

For the flanges:

$$\begin{aligned} \lambda &= \frac{b}{t} \\ &= \frac{4.00\text{ in.}}{1.00\text{ in.}} \\ &= 4.00 \end{aligned}$$

Determine the flange limiting slenderness ratio, λ_r, from AISC *Specification* Table B4.1a, Case 2:

$$\begin{aligned} \lambda_r &= 0.64\sqrt{\frac{k_c E}{F_y}} \\ &= 0.64\sqrt{\frac{0.516(29{,}000\text{ ksi})}{50\text{ ksi}}} \\ &= 11.1 \end{aligned}$$

Because $\lambda < \lambda_r$, the flanges are not slender and there is no reduction in effective area due to local buckling of the flanges.

Check for a slender web, and then determine the effective area for compression, A_e, using AISC *Specification* Section E7.1.

$$\begin{aligned} \lambda &= \frac{h}{t_w} \\ &= \frac{15.0\text{ in.}}{\tfrac{1}{4}\text{ in.}} \\ &= 60.0 \end{aligned}$$

Determine the slender web limit from AISC *Specification* Table B4.1a, Case 5:

$$\begin{aligned} \lambda_r &= 1.49\sqrt{\frac{E}{F_y}} \\ &= 1.49\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} \\ &= 35.9 \end{aligned}$$

Because $\lambda > \lambda_r$, the web is slender.

Determine the slenderness limit from AISC *Specification* Section E7.1 for a fully effective element:

$$\lambda_r\sqrt{\frac{F_y}{F_{cr}}} = 35.9\sqrt{\frac{50\text{ ksi}}{28.9\text{ ksi}}}$$
$$= 47.2$$

Because $\lambda > \lambda_r\sqrt{\frac{F_y}{F_{cr}}}$, the effective width is determined from AISC *Specification* Equation E7-3. Determine the effective width imperfection adjustment factors from AISC *Specification* Table E7.1, Case (a):

$c_1 = 0.18$
$c_2 = 1.31$

The elastic local buckling stress is:

$$F_{el} = \left(c_2\frac{\lambda_r}{\lambda}\right)^2 F_y \qquad (\textit{Spec}.\text{ Eq. E7-5})$$
$$= \left[1.31\left(\frac{35.9}{60.0}\right)\right]^2 (50\text{ ksi})$$
$$= 30.7\text{ ksi}$$

Determine the effective width of the web and the resulting effective area:

$$h_e = h\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} \qquad (\text{from }\textit{Spec}.\text{ Eq. E7-3})$$
$$= (15.0\text{ in.})\left(1 - 0.18\sqrt{\frac{30.7\text{ ksi}}{28.9\text{ ksi}}}\right)\sqrt{\frac{30.7\text{ ksi}}{28.9\text{ ksi}}}$$
$$= 12.6\text{ in.}$$

$$A_e = A_g - (h - h_e)t_w$$
$$= 19.8\text{ in.}^2 - (15.0\text{ in.} - 12.6\text{ in.})(\tfrac{1}{4}\text{ in.})$$
$$= 19.2\text{ in.}^2$$

Available Compressive Strength

$$P_n = F_{cr}A_e \qquad (\textit{Spec}.\text{ Eq. E7-1})$$
$$= (28.9\text{ ksi})(19.2\text{ in.}^2)$$
$$= 555\text{ kips}$$

From AISC *Specification* Section E1, the available compressive strength is:

LRFD	ASD
$\phi_c = 0.90$	$\Omega_c = 1.67$
$\phi_c P_n = 0.90(555\text{ kips})$ $= 500\text{ kips} > 420\text{ kips}$ **o.k.**	$\frac{P_n}{\Omega_c} = \frac{555\text{ kips}}{1.67}$ $= 332\text{ kips} > 280\text{ kips}$ **o.k.**

EXAMPLE E.3 BUILT-UP COLUMN WITH SLENDER FLANGES

Given:

Determine if a built-up, ASTM A572 Grade 50 column with PL⅜ in. × 10½ in. flanges and a PL¼ in. × 7¼ in. web, as shown in Figure E.3-1, has sufficient available strength to carry a dead load of 40 kips and a live load of 120 kips in axial compression. The column's unbraced length is 15 ft and the ends are pinned in both axes.

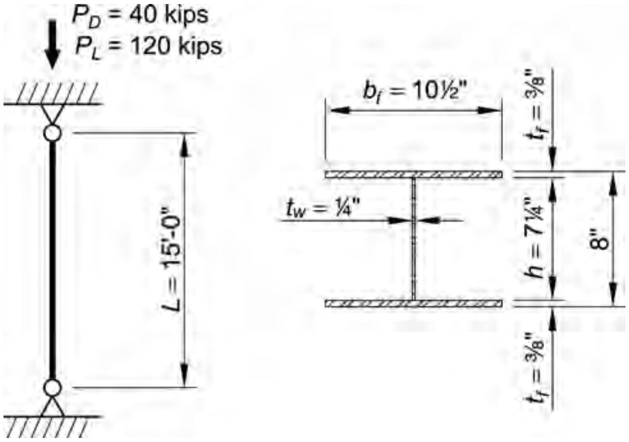


Fig. E.3-1. Column geometry for Example E.3.

Solution:

From AISC *Manual* Table 2-5, the material properties are as follows:

Built-Up Column
ASTM A572 Grade 50
F_y = 50 ksi
F_u = 65 ksi

The geometric properties are as follows:

Built-Up Column
d = 8.00 in.
b_f = 10½ in.
t_f = ⅜ in.
h = 7¼ in.
t_w = ¼ in.

From ASCE/SEI 7, Chapter 2, the required compressive strength is:

LRFD	ASD
$P_u = 1.2(40\text{ kips}) + 1.6(120\text{ kips})$ $= 240\text{ kips}$	$P_a = 40\text{ kips} + 120\text{ kips}$ $= 160\text{ kips}$

Built-Up Section Properties (ignoring fillet welds)

$$\begin{aligned} A_g &= 2(10\tfrac{1}{2}\text{ in.})(\tfrac{3}{8}\text{ in.}) + (7\tfrac{1}{4}\text{ in.})(\tfrac{1}{4}\text{ in.}) \\ &= 9.69\text{ in.}^2 \end{aligned}$$

Because the unbraced length is the same for both axes, the weak axis will govern.

$$\begin{aligned} I_y &= \sum \frac{bh^3}{12} \\ &= 2\left[\frac{(\tfrac{3}{8}\text{ in.})(10\tfrac{1}{2}\text{ in.})^3}{12}\right] + \frac{(7\tfrac{1}{4}\text{ in.})(\tfrac{1}{4}\text{ in.})^3}{12} \\ &= 72.4\text{ in.}^4 \end{aligned}$$

$$\begin{aligned} r_y &= \sqrt{\frac{I_y}{A_g}} \\ &= \sqrt{\frac{72.4\text{ in.}^4}{9.69\text{ in.}^2}} \\ &= 2.73\text{ in.} \end{aligned}$$

$$\begin{aligned} I_x &= \sum Ad^2 + \sum \frac{bh^3}{12} \\ &= 2\left[(10\tfrac{1}{2}\text{ in.})(\tfrac{3}{8}\text{ in.})(3.81\text{ in.})^2\right] + \frac{(\tfrac{1}{4}\text{ in.})(7\tfrac{1}{4}\text{ in.})^3}{12} + 2\left[\frac{(10\tfrac{1}{2}\text{ in.})(\tfrac{3}{8}\text{ in.})^3}{12}\right] \\ &= 122\text{ in.}^4 \end{aligned}$$

Web Slenderness

Determine the limiting slenderness ratio, λ_r, from AISC *Specification* Table B4.1a, Case 5:

$$\begin{aligned} \lambda_r &= 1.49\sqrt{\frac{E}{F_y}} \\ &= 1.49\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} \\ &= 35.9 \end{aligned}$$

$$\begin{aligned} \lambda &= \frac{h}{t_w} \\ &= \frac{7\tfrac{1}{4}\text{ in.}}{\tfrac{1}{4}\text{ in.}} \\ &= 29.0 \end{aligned}$$

Because $\lambda < \lambda_r$, the web is not slender.

Note that the fillet welds are ignored in the calculation of h for built up sections.

Flange Slenderness

Calculate k_c using AISC *Specification* Table B4.1a, note [a]:

$$\begin{aligned} k_c &= \frac{4}{\sqrt{h/t_w}} \\ &= \frac{4}{\sqrt{\dfrac{7\frac{1}{4}\text{ in.}}{\frac{1}{4}\text{ in.}}}} \\ &= 0.743,\ \text{which is between 0.35 and 0.76} \end{aligned}$$

Determine the limiting slenderness ratio, λ_r, from AISC *Specification* Table B4.1a, Case 2:

$$\begin{aligned} \lambda_r &= 0.64\sqrt{\frac{k_c E}{F_y}} \\ &= 0.64\sqrt{\frac{0.743(29{,}000\text{ ksi})}{50\text{ ksi}}} \\ &= 13.3 \end{aligned}$$

$$\begin{aligned} \lambda &= \frac{b}{t} \\ &= \frac{5.25\text{ in.}}{\frac{3}{8}\text{ in.}} \\ &= 14.0 \end{aligned}$$

Because $\lambda > \lambda_r$, the flanges are slender.

For compression members with slender elements, AISC *Specification* Section E7 applies. The nominal compressive strength, P_n, is determined based on the limit states of flexural, torsional and flexural-torsional buckling. Depending on the slenderness of the column, AISC *Specification* Equation E3-2 or E3-3 applies. F_e is used in both equations and is calculated as the lesser of AISC *Specification* Equations E3-4 and E4-2.

From AISC *Specification* Commentary Table C-A-7.1, for a pinned-pinned condition, K = 1.0.

Because the unbraced length is the same for both axes, the weak axis will govern. With $L_{cy} = K_yL_y = 1.0(15\text{ ft}) = 15.0\text{ ft}$:

$$\begin{aligned} \frac{L_{cy}}{r_y} &= \frac{(15.0\text{ ft})(12\text{ in./ft})}{2.73\text{ in.}} \\ &= 65.9 \end{aligned}$$

Elastic Critical Stress, F_e, for Flexural Buckling

$$\begin{aligned} F_e &= \frac{\pi^2 E}{\left(\dfrac{L_{cy}}{r_y}\right)^2} \qquad \text{(from } \textit{Spec}\text{. Eq. E3-4)} \\ &= \frac{\pi^2(29{,}000\text{ ksi})}{(65.9)^2} \\ &= 65.9\text{ ksi} \end{aligned}$$

Elastic Critical Stress, F_e, for Torsional Buckling

Note: This limit state is not likely to govern, but the check is included here for completeness.

From the User Note in AISC *Specification* Section E4:

$$\begin{aligned} C_w &= \frac{I_y h_o^2}{4} \\ &= \frac{\left(72.4\text{ in.}^4\right)\left(7.63\text{ in.}\right)^2}{4} \\ &= 1{,}050\text{ in.}^6 \end{aligned}$$

From AISC Design Guide 9, Equation 3.4:

$$\begin{aligned} J &= \sum \frac{bt^3}{3} \\ &= \frac{2\left(10\tfrac{1}{2}\text{ in.}\right)\left(\tfrac{3}{8}\text{ in.}\right)^3 + \left(7\tfrac{1}{4}\text{ in.}\right)\left(\tfrac{1}{4}\text{ in.}\right)^3}{3} \\ &= 0.407\text{ in.}^4 \end{aligned}$$

With $L_{cz} = K_z L_z = 1.0(15\text{ ft}) = 15\text{ ft}$:

$$F_e = \left(\frac{\pi^2 E C_w}{L_{cz}^2} + GJ\right)\frac{1}{I_x + I_y} \qquad \text{(}\textit{Spec}\text{. Eq. E4-2)}$$

$$\begin{aligned} &= \left\{\frac{\pi^2\left(29{,}000\text{ ksi}\right)\left(1{,}050\text{ in.}^6\right)}{\left[\left(15\text{ ft}\right)\left(12\text{ in./ft}\right)\right]^2} + \left(11{,}200\text{ ksi}\right)\left(0.407\text{ in.}^4\right)\right\}\left(\frac{1}{122\text{ in.}^4 + 72.4\text{ in.}^4}\right) \\ &= 71.2\text{ ksi} > 65.9\text{ ksi} \end{aligned}$$

Therefore, use $F_e = 65.9$ ksi.

Flexural Buckling Stress

$$\begin{aligned} \frac{F_y}{F_e} &= \frac{50\text{ ksi}}{65.9\text{ ksi}} \\ &= 0.759 \end{aligned}$$

Because $\dfrac{F_y}{F_e} < 2.25$:

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y \qquad \text{(}\textit{Spec}\text{. Eq. E3-2)}$$

$$\begin{aligned} &= \left(0.658^{0.759}\right)\left(50\text{ ksi}\right) \\ &= 36.4\text{ ksi} \end{aligned}$$

Effective Area, A_e

The effective area, A_e, is the summation of the effective areas of the cross section based on the reduced effective widths, b_e or h_e. Since the web is nonslender, there is no reduction in the effective area due to web local buckling and $h_e = h$.

Determine the slender web limit from AISC *Specification* Section E7.1.

$$\begin{aligned}\lambda_r\sqrt{\frac{F_y}{F_{cr}}} &= 13.3\sqrt{\frac{50\text{ ksi}}{36.4\text{ ksi}}}\\ &= 15.6\end{aligned}$$

Because $\lambda < \lambda_r\sqrt{\frac{F_y}{F_{cr}}}$ for all elements,

$$b_e = b \qquad \text{(\textit{Spec}. Eq. E7-2)}$$

Therefore, $A_e = A_g$.

Available Compressive Strength

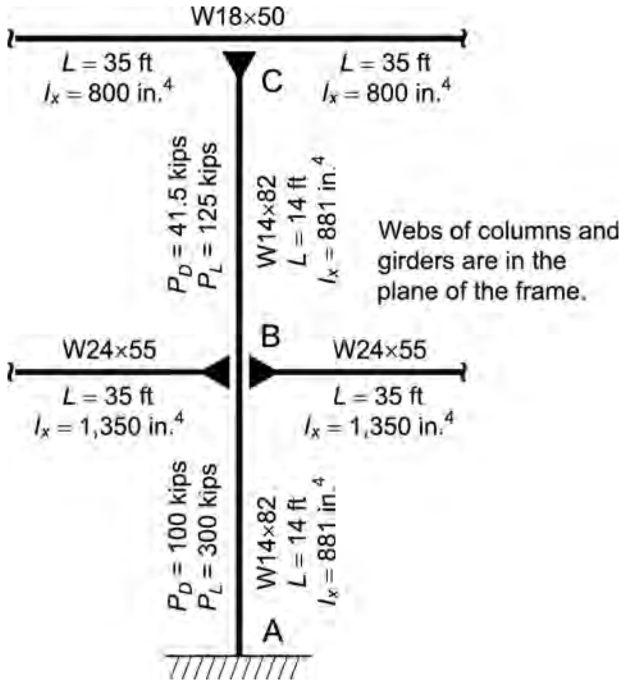
$$\begin{aligned}P_n &= F_{cr}A_e\\ &= (36.4\text{ ksi})(9.69\text{ in.}^2) \qquad \text{(\textit{Spec}. Eq. E7-1)}\\ &= 353\text{ kips}\end{aligned}$$

From AISC *Specification* Section E1, the available compressive strength is:

LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(353\text{ kips})$ $= 318\text{ kips} > 240\text{ kips}$ **o.k.**	$\Omega_c = 1.67$ $\frac{P_n}{\Omega_c} = \frac{353\text{ kips}}{1.67}$ $= 211\text{ kips} > 160\text{ kips}$ **o.k.**

Note: Built-up sections are generally more expensive than standard rolled shapes; therefore, a standard compact shape, such as a W8×35 might be a better choice even if the weight is somewhat higher. This selection could be taken directly from AISC *Manual* Table 4-1a.

EXAMPLE E.4A W-SHAPE COMPRESSION MEMBER (MOMENT FRAME)



This example is primarily intended to illustrate the use of the alignment chart for sidesway uninhibited columns in conjunction with the effective length method.

Given:

The member sizes shown for the moment frame illustrated here (sidesway uninhibited in the plane of the frame) have been determined to be adequate for lateral loads. The material for both the column and the girders is ASTM A992. The loads shown at each level are the accumulated dead loads and live loads at that story. The column is fixed at the base about the *x-x* axis of the column.

Determine if the column is adequate to support the gravity loads shown. Assume the column is continuously supported in the transverse direction (the *y-y* axis of the column).

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W18×50
$I_x = 800$ in.^{4}

W24×55
$I_x = 1,350$ in.^{4}

W14×82
$A_g = 24.0$ in.^{2}
$I_x = 881$ in.^{4}

Column B-C

From ASCE/SEI 7, Chapter 2, the required compressive strength for the column between the roof and floor is:

LRFD	ASD
$P_u = 1.2(41.5\text{ kips}) + 1.6(125\text{ kips})$ $= 250\text{ kips}$	$P_a = 41.5\text{ kips} + 125\text{ kips}$ $= 167\text{ kips}$

Effective Length Factor

Using the effective length method, the effective length factor is determined using AISC *Specification* Commentary Appendix 7, Section 7.2. As discussed there, column inelasticity should be addressed by incorporating the stiffness reduction parameter, τ_b. Determine G_{top} and G_{bottom} accounting for column inelasticity by replacing $E_{col}I_{col}$ with $\tau_b(E_{col}I_{col})$. Calculate the stiffness reduction parameter, τ_b, for the column B-C using AISC *Manual* Table 4-13.

LRFD	ASD
$\frac{P_u}{A_g} = \frac{250\text{ kips}}{24.0\text{ in.}^2}$ $= 10.4\text{ ksi}$ $\tau_b = 1.00$	$\frac{P_a}{A_g} = \frac{167\text{ kips}}{24.0\text{ in.}^2}$ $= 6.96\text{ ksi}$ $\tau_b = 1.00$

Therefore, no reduction in stiffness for inelastic buckling will be required.

Determine G_{top} and G_{bottom}.

$$G_{top} = \tau_b\left[\frac{\sum(EI/L)_{col}}{\sum(EI/L)_g}\right] \qquad \text{(from } \textit{Spec.}\text{ Comm. Eq. C-A-7-3)}$$

$$= 1.00\left\{\frac{\left[\frac{(29{,}000\text{ ksi})(881\text{ in.}^4)}{14.0\text{ ft}}\right]}{2\left[\frac{(29{,}000\text{ ksi})(800\text{ in.}^4)}{35.0\text{ ft}}\right]}\right\}$$

$$= 1.38$$

$$G_{bottom} = \tau_b\left[\frac{\sum(EI/L)_{col}}{\sum(EI/L)_g}\right] \qquad \text{(from } \textit{Spec.}\text{ Comm. Eq. C-A-7-3)}$$

$$= 1.00\left\{\frac{2\left[\frac{(29{,}000\text{ ksi})(881\text{ in.}^4)}{14.0\text{ ft}}\right]}{2\left[\frac{(29{,}000\text{ ksi})(1{,}350\text{ in.}^4)}{35.0\text{ ft}}\right]}\right\}$$

$$= 1.63$$

From the alignment chart, AISC *Specification* Commentary Figure C-A-7.2, K is slightly less than 1.5; therefore use $K = 1.5$. Because the column available strength tables are based on the L_c about the y-y axis, the equivalent effective column length of the upper segment for use in the table is:

$$L_{cx} = (KL)_x$$
$$= 1.5(14\text{ ft})$$
$$= 21.0\text{ ft}$$

From AISC *Manual* Table 4-1a, for a W14×82:

$$\frac{r_x}{r_y} = 2.44$$

$$\begin{aligned} L_c &= \frac{L_{cx}}{\left(\dfrac{r_x}{r_y}\right)} \\ &= \frac{21.0 \text{ ft}}{2.44} \\ &= 8.61 \text{ ft} \end{aligned}$$

Take the available strength of the W14×82 from AISC *Manual* Table 4-1a.

At L_c = 9 ft, the available strength in axial compression is:

LRFD	ASD
$\phi_c P_n = 940 \text{ kips} > 250 \text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_c} = 626 \text{ kips} > 167 \text{ kips}$ **o.k.**

Column A-B

From Chapter 2 of ASCE/SEI 7, the required compressive strength for the column between the floor and the foundation is:

LRFD	ASD
$P_u = 1.2(100 \text{ kips}) + 1.6(300 \text{ kips})$ $= 600 \text{ kips}$	$P_a = 100 \text{ kips} + 300 \text{ kips}$ $= 400 \text{ kips}$

Effective Length Factor

Determine the stiffness reduction parameter, τ_b, for column A-B using AISC *Manual* Table 4-13.

LRFD	ASD
$\dfrac{P_u}{A_g} = \dfrac{600 \text{ kips}}{24.0 \text{ in.}^2}$ $= 25.0 \text{ ksi}$ $\tau_b = 1.00$	$\dfrac{P_a}{A_g} = \dfrac{400 \text{ kips}}{24.0 \text{ in.}^2}$ $= 16.7 \text{ ksi}$ $\tau_b = 0.994$

Use τ_b = 0.994.

$$G_{top} = \tau_b \left[\frac{\Sigma(EI/L)_{col}}{\Sigma(EI/L)_g} \right] \qquad \text{(from } \textit{Spec.} \text{ Comm. Eq. C-A-7-3)}$$

$$= 0.994 \left\{ \frac{2\left[\frac{(29{,}000 \text{ ksi})(881 \text{ in.}^4)}{14.0 \text{ ft}} \right]}{2\left[\frac{(29{,}000 \text{ ksi})(1{,}350 \text{ in.}^4)}{35.0 \text{ ft}} \right]} \right\}$$

$$= 1.62$$

$G_{bottom} = 1.0$ (fixed), from AISC *Specification* Commentary Appendix 7, Section 7.2

From the alignment chart, AISC *Specification* Commentary Figure C-A-7.2, K is approximately 1.4. Because the column available strength tables are based on L_c about the y-y axis, the effective column length of the lower segment for use in the table is:

$$\begin{aligned} L_{cx} &= (KL)_x \\ &= 1.4(14 \text{ ft}) \\ &= 19.6 \text{ ft} \end{aligned}$$

$$\begin{aligned} L_c &= \frac{L_{cx}}{\left(\dfrac{r_x}{r_y}\right)} \\ &= \frac{19.6 \text{ ft}}{2.44} \\ &= 8.03 \text{ ft} \end{aligned}$$

Take the available strength of the W14×82 from AISC *Manual* Table 4-1a.

At L_c = 9 ft, (conservative) the available strength in axial compression is:

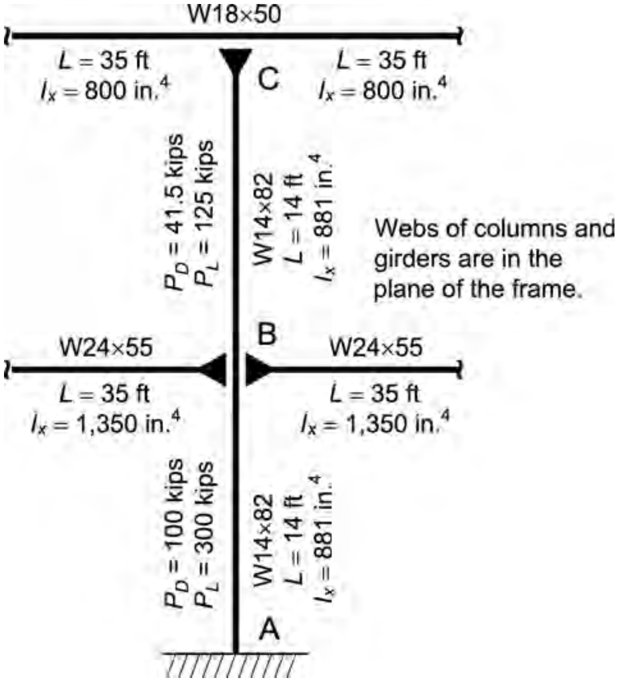
LRFD	ASD
$\phi_c P_n = 940 \text{ kips} > 600 \text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_c} = 626 \text{ kips} > 400 \text{ kips}$ **o.k.**

A more accurate strength could be determined by interpolation from AISC *Manual* Table 4-1a.

EXAMPLE E.4B W-SHAPE COMPRESSION MEMBER (MOMENT FRAME)

Given:

Using the effective length method, determine the available strength of the column shown subject to the same gravity loads shown in Example E.4A with the column pinned at the base about the *x*-*x* axis. All other assumptions remain the same.



Solution:

As determined in Example E.4A, for the column segment B-C between the roof and the floor, the column strength is adequate.

As determined in Example E.4A, for the column segment A-B between the floor and the foundation,

$$G_{top} = 1.62$$

At the base,

$G_{bottom} = 10$ (pinned) from AISC *Specification* Commentary Appendix 7, Section 7.2

Note: this is the only change in the analysis.

From the alignment chart, AISC *Specification* Commentary Figure C-A-7.2, K is approximately equal to 2.0. Because the column available strength tables are based on the effective length, L_c, about the *y*-*y* axis, the effective column length of the segment A-B for use in the table is:

$$\begin{aligned} L_{cx} &= (KL)_x \\ &= 2.0(14\text{ ft}) \\ &= 28.0\text{ ft} \end{aligned}$$

From AISC *Manual* Table 4-1a, for a W14×82:

$$\frac{r_x}{r_y} = 2.44$$

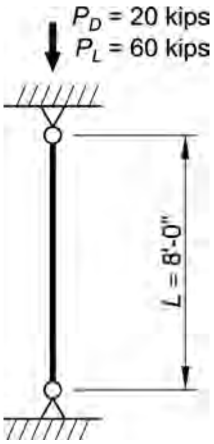
$$
\begin{aligned}
L_c &= \frac{L_{cx}}{\left(\dfrac{r_x}{r_y}\right)} \\
&= \frac{28.0\text{ ft}}{2.44} \\
&= 11.5\text{ ft}
\end{aligned}
$$

Interpolate the available strength of the W14×82 from AISC *Manual* Table 4-1a.

LRFD	ASD
$\phi_c P_n = 861\text{ kips} > 600\text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_c} = 573\text{ kips} > 400\text{ kips}$ **o.k.**

EXAMPLE E.5 DOUBLE-ANGLE COMPRESSION MEMBER WITHOUT SLENDER ELEMENTS

Given:



Verify the strength of a 2L4×3½×⅜ LLBB (¾-in. separation) strut, ASTM A36, with a length of 8 ft and pinned ends carrying an axial dead load of 20 kips and live load of 60 kips. Also, calculate the required number of pretensioned bolted or welded intermediate connectors required. The solution will be provided using:

(1) AISC *Manual* Tables
(2) Calculations using AISC *Specification* provisions

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Tables 1-7 and 1-15, the geometric properties are as follows:

L4×3½×⅜
r_z = 0.719 in.

2L4×3½×⅜ LLBB
r_x = 1.25 in.
r_y = 1.55 in. for ⅜-in. separation
r_y = 1.69 in. for ¾-in. separation

From ASCE/SEI 7, Chapter 2, the required compressive strength is:

LRFD	ASD
$P_u = 1.2(20\text{ kips}) + 1.6(60\text{ kips})$ $= 120\text{ kips}$	$P_a = 20\text{ kips} + 60\text{ kips}$ $= 80.0\text{ kips}$

(1) AISC *Manual* Table Solution

From AISC *Specification* Commentary Table C-A-7.1, for a pinned-pinned condition, K = 1.0. Therefore, $L_{cx} = L_{cy} = KL$ = 1.0(8 ft) = 8.00 ft. The available strength in axial compression is taken from the upper (X-X Axis) portion of AISC *Manual* Table 4-9:

LRFD	ASD
$\phi_c P_n = 127\text{ kips} > 120\text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_c} = 84.7\text{ kips} > 80.0\text{ kips}$ **o.k.**

For buckling about the y-y axis, the values are tabulated for a separation of ⅜ in.

To adjust to a spacing of ¾ in., L_{cy} is multiplied by the ratio of the r_y for a ⅜-in. separation to the r_y for a ¾-in. separation, where $L_{cy} = K_yL_y$ = 1.0(8 ft) = 8.00 ft . Thus:

$$\begin{aligned} L_{cy} &= (8.00\text{ ft})\left(\frac{1.55\text{ in.}}{1.69\text{ in.}}\right) \\ &= 7.34\text{ ft} \end{aligned}$$

The calculation of the equivalent L_{cy} in the preceding text is a simplified approximation of AISC *Specification* Section E6.1. To ensure a conservative adjustment for a ¾-in. separation, take L_{cy} = 8 ft. The available strength in axial compression is taken from the lower (Y-Y Axis) portion of AISC *Manual* Table 4-9 as:

LRFD	ASD
$\phi_c P_n = 132 \text{ kips} > 120 \text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_c} = 87.9 \text{ kips} > 80.0 \text{ kips}$ **o.k.**

Therefore, *x-x* axis flexural buckling governs.

Intermediate Connectors

From AISC *Manual* Table 4-9, at least two welded or pretensioned bolted intermediate connectors are required. This can be verified as follows:

$$\begin{aligned} a &= \text{ distance between connectors} \\ &= \frac{(8.00 \text{ ft})(12 \text{ in./ft})}{3 \text{ spaces}} \\ &= 32.0 \text{ in.} \end{aligned}$$

From AISC *Specification* Section E6.2, the effective slenderness ratio of the individual components of the built-up member based upon the distance between intermediate connectors, *a*, must not exceed three-fourths of the governing slenderness ratio of the built-up member.

Therefore, $\dfrac{a}{r_i} \le \dfrac{3}{4}\left(\dfrac{L_c}{r}\right)_{max}$.

Solving for *a* gives:

$$a \le \frac{3r_i\left(\dfrac{L_c}{r}\right)_{max}}{4}$$

$$\begin{aligned} \frac{L_{cx}}{r_x} &= \frac{(8.00 \text{ ft})(12 \text{ in./ft})}{1.25 \text{ in.}} \\ &= 76.8 \quad \textbf{controls} \end{aligned}$$

$$\begin{aligned} \frac{L_{cy}}{r_y} &= \frac{(8.00 \text{ ft})(12 \text{ in./ft})}{1.69 \text{ in.}} \\ &= 56.8 \end{aligned}$$

$$\begin{aligned} a &= \frac{3r_z\left(\dfrac{L_c}{r}\right)_{max}}{4} \\ &= \frac{3(0.719 \text{ in.})(76.8)}{4} \\ &= 41.4 \text{ in.} \end{aligned}$$

Therefore, two welded or pretensioned bolted connectors are adequate since 32.0 in. < 41.4 in.

Note that one connector would not be adequate as 48.0 in. > 41.4 in. Available strength can also be determined by hand calculations, as demonstrated in the following.

(2) Calculations Using AISC *Specification* Provisions

From AISC *Manual* Tables 1-7 and 1-15, the geometric properties are as follows:

L4×3½×⅜
$J = 0.132 \text{ in.}^4$

2L4×3½×⅜ LLBB (¾ in. separation)
$A_g = 5.36 \text{ in.}^2$
$r_y = 1.69 \text{ in.}$
$\bar{r}_o = 2.33 \text{ in.}$
$H = 0.813$

Slenderness Check

$$\begin{aligned}\lambda &= \frac{b}{t}\\ &= \frac{4.00 \text{ in.}}{3/8 \text{ in.}}\\ &= 10.7\end{aligned}$$

Determine the limiting slenderness ratio, λ_r, from AISC *Specification* Table B4.1a, Case 3:

$$\begin{aligned}\lambda_r &= 0.45\sqrt{\frac{E}{F_y}}\\ &= 0.45\sqrt{\frac{29{,}000 \text{ ksi}}{36 \text{ ksi}}}\\ &= 12.8\end{aligned}$$

$\lambda < \lambda_r$; therefore, there are no slender elements.

For double-angle compression members without slender elements, AISC *Specification* Sections E3, E4 and E6 apply.

The nominal compressive strength, P_n, is determined based on the limit states of flexural, torsional and flexural-torsional buckling.

Flexural Buckling about the x-x Axis

$$\begin{aligned}\frac{L_{cx}}{r_x} &= \frac{(8.00 \text{ ft})(12 \text{ in./ft})}{1.25 \text{ in.}}\\ &= 76.8\end{aligned}$$

$$F_{ex} = \frac{\pi^2 E}{\left(\dfrac{L_{cx}}{r_x}\right)^2} \qquad \text{(}\textit{Spec}\text{. Eq. E4-5)}$$

$$= \frac{\pi^2 (29{,}000 \text{ ksi})}{(76.8)^2}$$

$$= 48.5 \text{ ksi}$$

Flexural Buckling about the y-y Axis

$$\frac{L_{cy}}{r_y} = \frac{(8.00 \text{ ft})(12 \text{ in./ft})}{1.69 \text{ in.}}$$

$$= 56.8$$

Using AISC *Specification* Section E6, compute the modified L_c/r for built up members with pretensioned bolted or welded connectors. Assume two connectors are required.

$$a = \frac{(8.00 \text{ ft})(12 \text{ in./ft})}{3}$$

$$= 32.0 \text{ in.}$$

$$r_i = r_z \text{ (single angle)}$$

$$= 0.719 \text{ in.}$$

$$\frac{a}{r_i} = \frac{32.0 \text{ in.}}{0.719 \text{ in.}}$$

$$= 44.5 > 40$$

Therefore:

$$\left(\frac{L_c}{r}\right)_m = \sqrt{\left(\frac{L_c}{r}\right)_o^2 + \left(\frac{K_i a}{r_i}\right)^2} \qquad \text{(}\textit{Spec}\text{. Eq. E6-2b)}$$

where $K_i = 0.50$ for angles back-to-back

$$\left(\frac{L_c}{r}\right)_m = \sqrt{(56.8)^2 + \left[\frac{0.50(32.0 \text{ in.})}{0.719 \text{ in.}}\right]^2}$$

$$= 61.0$$

$$F_{ey} = \frac{\pi^2 E}{\left(\dfrac{L_{cy}}{r_y}\right)^2} \qquad \text{(}\textit{Spec}\text{. Eq. E4-6)}$$

$$= \frac{\pi^2 (29{,}000 \text{ ksi})}{(61.0)^2}$$

$$= 76.9 \text{ ksi}$$

Torsional and Flexural-Torsional Buckling

For nonslender double-angle compression members, AISC *Specification* Equation E4-3 applies. Per the User Note for AISC *Specification* Section E4, the term with C_w is omitted when computing F_{ez} and x_o is taken as zero. The flexural buckling term about the *y-y* axis, F_{ey}, was computed in the preceding section.

$$F_{ez} = \left(\frac{\pi^2 E C_w}{L_{cz}^2} + GJ\right)\frac{1}{A_g \bar{r}_o^2} \qquad (\textit{Spec}.\ \text{Eq. E4-7})$$

$$= \left[0 + (11{,}200\text{ ksi})(0.132\text{ in.}^4)(2\text{ angles})\right]\frac{1}{(5.36\text{ in.}^2)(2.33\text{ in.})^2}$$

$$= 102\text{ ksi}$$

$$F_e = \left(\frac{F_{ey}+F_{ez}}{2H}\right)\left[1-\sqrt{1-\frac{4F_{ey}F_{ez}H}{(F_{ey}+F_{ez})^2}}\right] \qquad (\textit{Spec}.\ \text{Eq. E4-3})$$

$$= \left[\frac{76.9\text{ ksi}+102\text{ ksi}}{2(0.813)}\right]\left[1-\sqrt{1-\frac{4(76.9\text{ ksi})(102\text{ ksi})(0.813)}{(76.9\text{ ksi}+102\text{ ksi})^2}}\right]$$

$$= 60.5\text{ ksi}$$

Critical Buckling Stress

The critical buckling stress for the member could be controlled by flexural buckling about either the *x-x* axis or *y-y* axis, F_{ex} or F_{ey}, respectively. Note that AISC *Specification* Equations E4-5 and E4-6 reflect the same buckling modes as calculated in AISC *Specification* Equation E3-4. Or, the critical buckling stress for the member could be controlled by torsional or flexural-torsional buckling calculated per AISC *Specification* Equation E4-3. In this example, F_e calculated in accordance with AISC *Specification* Equation E4-5 (or Equation E3-4) is less than that calculated in accordance with AISC *Specification* Equation E4-3 or E4-6, and controls. Therefore:

$$F_e = 48.5\text{ ksi}$$

$$\frac{F_y}{F_e} = \frac{36\text{ ksi}}{48.5\text{ ksi}}$$

$$= 0.742$$

Per the AISC *Specification* User Note for Section E3, the two inequalities for calculating limits of applicability of Sections E3(a) and E3(b) provide the same result for flexural buckling only. When the elastic buckling stress, F_e, is controlled by torsional or flexural-torsional buckling, the L_c/r limits would not be applicable unless an equivalent L_c/r ratio is first calculated by substituting the governing F_e into AISC *Specification* Equation E3-4 and solving for L_c/r. The F_y/F_e limits may be used regardless of which buckling mode governs.

Because $\dfrac{F_y}{F_e} < 2.25$:

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y \qquad (\textit{Spec}.\ \text{Eq. E3-2})$$

$$= \left(0.658^{0.742}\right)(36\text{ ksi})$$

$$= 26.4\text{ ksi}$$

Available Compressive Strength

$$P_n = F_{cr} A_g \qquad \text{(}\textit{Spec}\text{. Eq. E3-1, Eq. E4-1)}$$

$$= (26.4 \text{ ksi})(5.36 \text{ in.}^2)$$

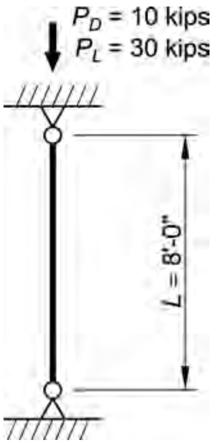
$$= 142 \text{ kips}$$

From AISC *Specification* Section E1, the available compressive strength is:

LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(142 \text{ kips})$ $= 128 \text{ kips} > 120 \text{ kips}$ **o.k.**	$\Omega_c = 1.67$ $\dfrac{P_n}{\Omega_c} = \dfrac{142 \text{ kips}}{1.67}$ $= 85.0 \text{ kips} > 80.0 \text{ kips}$ **o.k.**

EXAMPLE E.6 DOUBLE-ANGLE COMPRESSION MEMBER WITH SLENDER ELEMENTS

Given:



Determine if a 2L5×3×¼ LLBB (¾-in. separation) strut, ASTM A36, with a length of 8 ft and pinned ends has sufficient available strength to support a dead load of 10 kips and live load of 30 kips in axial compression. Also, calculate the required number of pretensioned bolted or welded intermediate connectors. The solution will be provided using:

(1) AISC *Manual* Tables
(2) Calculations using AISC *Specification* provisions

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Tables 1-7 and 1-15, the geometric properties are as follows:

L5×3×¼
r_z = 0.652 in.

2L5×3×¼ LLBB
r_x = 1.62 in.
r_y = 1.19 in. for ⅜-in. separation
r_y = 1.33 in. for ¾-in. separation

From ASCE/SEI 7, Chapter 2, the required compressive strength is:

LRFD	ASD
$P_u = 1.2(10\text{ kips}) + 1.6(30\text{ kips})$ $= 60.0\text{ kips}$	$P_a = 10\text{ kips} + 30\text{ kips}$ $= 40.0\text{ kips}$

(1) AISC *Manual* Table Solution

From AISC *Specification* Commentary Table C-A-7.1, for a pinned-pinned condition, K = 1.0. Therefore, $L_{cx} = L_{cy} = KL$ = 1.0(8 ft) = 8.00 ft. The available strength in axial compression is taken from the upper (X-X Axis) portion of AISC *Manual* Table 4-9:

LRFD	ASD
$\phi_c P_{nx} = 91.2\text{ kips} > 60.0\text{ kips}$ **o.k.**	$\dfrac{P_{nx}}{\Omega_c} = 60.7\text{ kips} > 40.0\text{ kips}$ **o.k.**

For buckling about the y-y axis, the tabulated values are based on a separation of ⅜ in. To adjust for a spacing of ¾ in., L_{cy} is multiplied by the ratio of r_y for a ⅜-in. separation to r_y for a ¾-in. separation.

$$\begin{aligned} L_{cy} &= (8.00\text{ ft})\left(\frac{1.19\text{ in.}}{1.33\text{ in.}}\right) \\ &= 7.16\text{ ft} \end{aligned}$$

This calculation of the equivalent L_{cy} does not completely take into account the effect of AISC *Specification* Section E6.1 and is slightly unconservative.

From the lower portion of AISC *Manual* Table 4-9, interpolate for a value at L_{cy} = 7.16 ft.

The available strength in compression is:

LRFD	ASD
$\phi_c P_{ny} = 68.3 \text{ kips} > 60.0 \text{ kips}$ **o.k.**	$\dfrac{P_{ny}}{\Omega_c} = 45.4 \text{ kips} > 40.0 \text{ kips}$ **o.k.**

These strengths are approximate due to the linear interpolation from the table and the approximate value of the equivalent L_{cy} noted in the preceding text. These can be compared to the more accurate values calculated in detail as follows.

Intermediate Connectors

From AISC *Manual* Table 4-9, it is determined that at least two welded or pretensioned bolted intermediate connectors are required. This can be confirmed by calculation, as follows:

$$\begin{aligned} a &= \text{ distance between connectors} \\ &= \frac{(8.00 \text{ ft})(12 \text{ in./ft})}{3 \text{ spaces}} \\ &= 32.0 \text{ in.} \end{aligned}$$

From AISC *Specification* Section E6.2, the effective slenderness ratio of the individual components of the built-up member based upon the distance between intermediate connectors, a, must not exceed three-fourths of the governing slenderness ratio of the built-up member.

Therefore, $\dfrac{a}{r_i} \le \dfrac{3}{4}\left(\dfrac{L_c}{r}\right)_{max}$.

Solving for a gives:

$$a \le \frac{3r_i\left(\dfrac{L_c}{r}\right)_{max}}{4}$$

$$\begin{aligned} r_i &= r_z \\ &= 0.652 \text{ in.} \end{aligned}$$

$$\begin{aligned} \frac{L_{cx}}{r_x} &= \frac{(8.00 \text{ ft})(12 \text{ in./ft})}{1.62 \text{ in.}} \\ &= 59.3 \end{aligned}$$

$$\begin{aligned} \frac{L_{cy}}{r_y} &= \frac{(8.00 \text{ ft})(12 \text{ in./ft})}{1.33 \text{ in.}} \\ &= 72.2 \quad \textbf{controls} \end{aligned}$$

$$a = \frac{3r_z\left(\frac{L_c}{r}\right)_{max}}{4}$$
$$= \frac{3(0.652\text{ in.})(72.2)}{4}$$
$$= 35.3\text{ in.}$$

Therefore, two welded or pretensioned bolted connectors are adequate since 32.0 in. < 35.3 in.

Available strength can also be determined by hand calculations, as determined in the following.

(2) Calculations Using AISC *Specification* Provisions

From AISC *Manual* Tables 1-7 and 1-15, the geometric properties are as follows.

L5×3×¼
J = 0.0438 in.^{4}
r_z = 0.652 in.

2L5×3×¼ LLBB
A_g = 3.88 in.^{2}
r_x = 1.62 in.
r_y = 1.33 in. for ¾-in. separation
$\bar{r}_o$ = 2.59 in.
H = 0.657

Slenderness Check

For the 5-in. leg:

$$\lambda = \frac{b}{t}$$
$$= \frac{5.00\text{ in.}}{\frac{1}{4}\text{ in.}}$$
$$= 20.0$$

For the 3-in. leg:

$$\lambda = \frac{b}{t}$$
$$= \frac{3.00\text{ in.}}{\frac{1}{4}\text{ in.}}$$
$$= 12.0$$

Calculate the limiting slenderness ratio, λ_r, from AISC *Specification* Table B4.1a, Case 3:

$$\lambda_r = 0.45\sqrt{\frac{E}{F_y}}$$
$$= 0.45\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}}$$
$$= 12.8$$

For the longer leg, $\lambda > \lambda_r$, and therefore it is classified as a slender element. For the shorter leg, $\lambda < \lambda_r$, and therefore it is classified as a nonslender element.

For a double-angle compression member with slender elements, AISC *Specification* Section E7 applies. The nominal compressive strength, P_n, is determined based on the limit states of flexural, torsional and flexural-torsional buckling. A_e will be determined by AISC *Specification* Section E7.1.

Elastic Buckling Stress about the x-x Axis

With $L_{cx} = K_x L_x$ = 1.0(8 ft) = 8.00 ft:

$$\begin{aligned}\frac{L_{cx}}{r_x} &= \frac{(8.00\text{ ft})(12\text{ in./ft})}{1.62\text{ in.}}\\ &= 59.3\end{aligned}$$

$$\begin{aligned}F_{ex} &= \frac{\pi^2 E}{\left(\dfrac{L_{cx}}{r_x}\right)^2}\\ &= \frac{\pi^2(29{,}000\text{ ksi})}{(59.3)^2}\\ &= 81.4\end{aligned}$$

(*Spec*. Eq. 3-4 or E4-5)

Elastic Buckling Stress about the y-y Axis

With $L_{cy} = K_y L_y$ = 1.0(8 ft) = 8.00 ft:

$$\begin{aligned}\frac{L_{cy}}{r_y} &= \frac{(8.00\text{ ft})(12\text{ in./ft})}{1.33\text{ in.}}\\ &= 72.2\end{aligned}$$

Using AISC *Specification* Section E6, compute the modified L_{cy}/r_y for built-up members with pretensioned bolted or welded connectors. Assuming two connectors are required:

$$\begin{aligned}a &= \frac{(8.00\text{ ft})(12\text{ in./ft})}{3}\\ &= 32.0\text{ in.}\end{aligned}$$

$$\begin{aligned}r_i &= r_z\ \text{(single angle)}\\ &= 0.652\text{ in.}\end{aligned}$$

$$\begin{aligned}\frac{a}{r_i} &= \frac{32.0\text{ in.}}{0.652\text{ in.}}\\ &= 49.1 > 40\end{aligned}$$

Therefore:

$$\left(\frac{L_c}{r}\right)_m = \sqrt{\left(\frac{L_c}{r}\right)_o^2 + \left(\frac{K_i a}{r_i}\right)^2}$$

(*Spec*. Eq. E6-2b)

where K_i = 0.50 for angles back-to-back

$$\left(\frac{L_c}{r}\right)_m = \sqrt{(72.2)^2 + \left[\frac{0.50(32.0\text{ in.})}{0.652\text{ in.}}\right]^2}$$
$$= 76.3$$

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{L_{cy}}{r_y}\right)^2} \qquad \text{(}\textit{Spec}\text{. Eq. E3-4 or E4-6)}$$
$$= \frac{\pi^2(29{,}000\text{ ksi})}{(76.3)^2}$$
$$= 49.2\text{ ksi}$$

Torsional and Flexural-Torsional Elastic Buckling Stress

Per the User Note in AISC *Specification* Section E4, the term with C_w is omitted when computing F_{ez}, and x_o is taken as zero. The flexural buckling term about the *y-y* axis, F_{ey}, was computed in the preceding section.

$$F_{ez} = \left(\frac{\pi^2 EC_w}{L_{cz}^2} + GJ\right)\frac{1}{A_g \bar{r}_o^2} \qquad \text{(}\textit{Spec}\text{. Eq. E4-7)}$$
$$= \left[0 + (11{,}200\text{ ksi})(0.0438\text{ in.}^4)(2\text{ angles})\right]\frac{1}{(3.88\text{ in.}^2)(2.59\text{ in.})^2}$$
$$= 37.7\text{ ksi}$$

$$F_e = \left(\frac{F_{ey}+F_{ez}}{2H}\right)\left[1-\sqrt{1-\frac{4F_{ey}F_{ez}H}{(F_{ey}+F_{ez})^2}}\right] \qquad \text{(}\textit{Spec}\text{. Eq. E4-3)}$$
$$= \left[\frac{49.2\text{ ksi}+37.7\text{ ksi}}{2(0.657)}\right]\left[1-\sqrt{1-\frac{4(49.2\text{ ksi})(37.7\text{ ksi})(0.657)}{(49.2\text{ ksi}+37.7\text{ ksi})^2}}\right]$$
$$= 26.8\text{ ksi} \quad \textbf{controls}$$

Critical Buckling Stress

The critical buckling stress for the member could be controlled by flexural buckling about either the *x-x* axis or *y-y* axis, F_{ex} or F_{ey}, respectively. Note that AISC *Specification* Equations E4-5 and E4-6 reflect the same buckling modes as calculated in AISC *Specification* Equation E3-4. Or, the critical buckling stress for the member could be controlled by torsional or flexural-torsional buckling calculated per AISC *Specification* Equation E4-3. In this example, F_e calculated in accordance with AISC *Specification* Equation E4-3 is less than that calculated in accordance with AISC *Specification* Equation E4-5 or E4-6, and controls. Therefore:

$$F_e = 26.8\text{ ksi}$$

$$\frac{F_y}{F_e} = \frac{36\text{ ksi}}{26.8\text{ ksi}}$$
$$= 1.34$$

Per the AISC *Specification* User Note for Section E3, the two inequalities for calculating limits of applicability of Sections E3(a) and E3(b) provide the same result for flexural buckling only. When the elastic buckling stress, F_e, is controlled by torsional or flexural-torsional buckling, the L_c/r limits would not be applicable unless an equivalent L_c/r ratio is first calculated by substituting the governing F_e into AISC *Specification* Equation E3-4 and solving for L_c/r. The F_y/F_e limits may be used regardless of which buckling mode governs.

Because $\dfrac{F_y}{F_e} < 2.25$:

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}} \right) F_y \qquad \text{(}\textit{Spec}\text{. Eq. E3-2)}$$

$$= \left(0.658^{1.34}\right)(36\text{ ksi})$$

$$= 20.5\text{ ksi}$$

Effective Area

Determine the limits of applicability for local buckling in accordance with AISC *Specification* Section E7.1. The shorter leg was shown previously to be nonslender and therefore no reduction in effective area due to local buckling of the shorter leg is required. The longer leg was shown previously to be slender and therefore the limits of AISC *Specification* Section E7.1 need to be evaluated.

$$\lambda = 20.0$$

$$\lambda_r \sqrt{\frac{F_y}{F_{cr}}} = 12.8\sqrt{\frac{36\text{ ksi}}{20.5\text{ ksi}}}$$

$$= 17.0$$

Because $\lambda > \lambda_r \sqrt{\dfrac{F_y}{F_{cr}}}$, determine the effective width imperfection adjustment factors per AISC *Specification* Table E7.1, Case (c).

$$c_1 = 0.22$$
$$c_2 = 1.49$$

Determine the elastic local buckling stress from AISC *Specification* Section E7.1.

$$F_{el} = \left(c_2 \frac{\lambda_r}{\lambda} \right)^2 F_y \qquad \text{(}\textit{Spec}\text{. Eq. E7-5)}$$

$$= \left[1.49\left(\frac{12.8}{20.0} \right) \right]^2 (36\text{ ksi})$$

$$= 32.7\text{ ksi}$$

Determine the effective width of the angle leg and the resulting effective area.

$$b_e = b\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} \qquad \text{(}\textit{Spec}\text{. Eq. E7-3)}$$

$$= (5.00\text{ in.})\left(1 - 0.22\sqrt{\frac{32.7\text{ ksi}}{20.5\text{ ksi}}}\right)\sqrt{\frac{32.7\text{ ksi}}{20.5\text{ ksi}}}$$

$$= 4.56\text{ in.}$$

$$A_e = A_g - t\sum(b - b_e)$$

$$= \left(3.88\text{ in.}^2\right) - (¼\text{ in.})(5.00\text{ in.} - 4.56\text{ in.})(2\text{ angles})$$

$$= 3.66\text{ in.}^2$$

Available Compressive Strength

$$P_n = F_{cr}A_e \qquad \text{(}\textit{Spec}\text{. Eq. E7-1)}$$

$$= (20.5\text{ ksi})\left(3.66\text{ in.}^2\right)$$

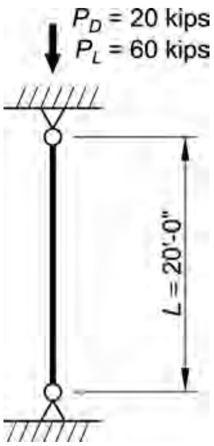
$$= 75.0\text{ kips}$$

From AISC *Specification* Section E1, the available compressive strength is:

LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(75.0\text{ kips})$ $= 67.5\text{ kips} > 60.0\text{ kips}$ **o.k.**	$\Omega_c = 1.67$ $\frac{P_n}{\Omega_c} = \frac{75.0\text{ kips}}{1.67}$ $= 44.9\text{ kips} > 40.0\text{ kips}$ **o.k.**

EXAMPLE E.7 WT COMPRESSION MEMBER WITHOUT SLENDER ELEMENTS

Given:



Select an ASTM A992 nonslender WT-shape compression member with a length of 20 ft to support a dead load of 20 kips and live load of 60 kips in axial compression. The ends are pinned. The solution will be provided using:

(1) AISC *Manual* Tables
(2) Calculations using AISC *Specification* provisions

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From ASCE/SEI 7, Chapter 2, the required compressive strength is:

LRFD	ASD
$P_u = 1.2(20\text{ kips}) + 1.6(60\text{ kips})$ $= 120\text{ kips}$	$P_a = 20\text{ kips} + 60\text{ kips}$ $= 80.0\text{ kips}$

(1) AISC *Manual* Table Solution

From AISC *Specification* Commentary Table C-A-7.1, for a pinned-pinned condition, K = 1.0. Therefore, L_{cx} = L_{cy} = KL = 1.0(20 ft) = 20.0 ft.

Select the lightest nonslender member from AISC *Manual* Table 4-7 with sufficient available strength about both the *x-x* axis (upper portion of the table) and the *y-y* axis (lower portion of the table) to support the required strength.

Try a WT7×34.

The available strength in compression is:

LRFD	ASD
$\phi_c P_{nx} = 128\text{ kips} > 120\text{ kips}$ **o.k. controls**	$\dfrac{P_{nx}}{\Omega_c} = 85.5\text{ kips} > 80.0\text{ kips}$ **o.k. controls**
$\phi_c P_{ny} = 222\text{ kips} > 120\text{ kips}$ **o.k.**	$\dfrac{P_{ny}}{\Omega_c} = 147\text{ kips} > 80.0\text{ kips}$ **o.k.**

Available strength can also be determined by hand calculations, as demonstrated in the following.

(2) Calculation Using AISC *Specification* Provisions

From AISC *Manual* Table 1-8, the geometric properties are as follows.

WT7×34
A_g = 10.0 in.^{2}
r_x = 1.81 in.
r_y = 2.46 in.
J = 1.50 in.^{4}

$\bar{y}$ = 1.29 in.
I_x = 32.6 in.^{4}
I_y = 60.7 in.^{4}
d = 7.02 in.
t_w = 0.415 in.
b_f = 10.0 in.
t_f = 0.720 in.

Stem Slenderness Check

$$\begin{aligned}\lambda &= \frac{d}{t_w}\\ &= \frac{7.02\text{ in.}}{0.415\text{ in.}}\\ &= 16.9\end{aligned}$$

Determine the stem limiting slenderness ratio, λ_r, from AISC *Specification* Table B4.1a, Case 4:

$$\begin{aligned}\lambda_r &= 0.75\sqrt{\frac{E}{F_y}}\\ &= 0.75\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}\\ &= 18.1\end{aligned}$$

$\lambda < \lambda_r$; therefore, the stem is not slender

Flange Slenderness Check

$$\begin{aligned}\lambda &= \frac{b_f}{2t_f}\\ &= \frac{10.0\text{ in.}}{2(0.720\text{ in.})}\\ &= 6.94\end{aligned}$$

Determine the flange limiting slenderness ratio, λ_r, from AISC *Specification* Table B4.1a, Case 1:

$$\begin{aligned}\lambda_r &= 0.56\sqrt{\frac{E}{F_y}}\\ &= 0.56\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}\\ &= 13.5\end{aligned}$$

$\lambda < \lambda_r$; therefore, the flange is not slender

There are no slender elements.

For compression members without slender elements, AISC *Specification* Sections E3 and E4 apply. The nominal compressive strength, P_n, is determined based on the limit states of flexural, torsional and flexural-torsional buckling.

Elastic Flexural Buckling Stress about the x-x Axis

$$\frac{L_{cx}}{r_x}=\frac{(20.0\text{ ft})(12\text{ in./ft})}{1.81\text{ in.}}$$
$$=133$$

$$F_{ex}=\frac{\pi^2 E}{\left(\dfrac{L_{cx}}{r_x}\right)^2}$$ (*Spec*. Eq. E3-4 or E4-5)

$$=\frac{\pi^2(29{,}000\text{ ksi})}{(133)^2}$$
$$=16.2\text{ ksi}\quad\textbf{controls}$$

Elastic Flexural Buckling Stress about the y-y Axis

$$\frac{L_{cy}}{r_y}=\frac{(20.0\text{ ft})(12\text{ in./ft})}{2.46\text{ in.}}$$
$$=97.6$$

$$F_{ey}=\frac{\pi^2 E}{\left(\dfrac{L_{cy}}{r_y}\right)^2}$$ (*Spec*. Eq. E3-4 or E4-6)

$$=\frac{\pi^2(29{,}000\text{ ksi})}{(97.6)^2}$$
$$=30.0\text{ ksi}$$

Torsional and Flexural-Torsional Elastic Buckling Stress

Because the WT7×34 section does not have any slender elements, AISC *Specification* Section E4 will be applicable for torsional and flexural-torsional buckling. F_e will be calculated using AISC *Specification* Equation E4-3. Per the User Note for AISC *Specification* Section E4, the term with C_w is omitted when computing F_{ez}, and x_o is taken as zero. The flexural buckling term about the *y-y* axis, F_{ey}, was computed in the preceding section.

$$x_o=0$$

$$y_o=\bar{y}-\frac{t_f}{2}$$
$$=1.29\text{ in.}-\frac{0.720\text{ in.}}{2}$$
$$=0.930\text{ in.}$$

$$\bar{r}_o^{\,2} = x_o^{\,2} + y_o^{\,2} + \frac{I_x + I_y}{A_g} \qquad \text{(\textit{Spec}. Eq. E4-9)}$$

$$= 0 + (0.930\text{ in.})^2 + \frac{32.6\text{ in.}^4 + 60.7\text{ in.}^4}{10.0\text{ in.}^2}$$

$$= 10.2\text{ in.}^2$$

$$F_{ez} = \left(\frac{\pi^2 E C_w}{L_{cz}^{\,2}} + GJ\right)\frac{1}{A_g \bar{r}_o^{\,2}} \qquad \text{(\textit{Spec}. Eq. E4-7)}$$

$$= \left[0 + (11{,}200\text{ ksi})\left(1.50\text{ in.}^4\right)\right]\frac{1}{\left(10.0\text{ in.}^2\right)\left(10.2\text{ in.}^2\right)}$$

$$= 165\text{ ksi}$$

$$H = 1 - \frac{x_o^{\,2} + y_o^{\,2}}{\bar{r}_o^{\,2}} \qquad \text{(\textit{Spec}. Eq. E4-8)}$$

$$= 1 - \frac{0 + (0.930\text{ in.})^2}{10.2\text{ in.}^2}$$

$$= 0.915$$

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{\left(F_{ey} + F_{ez}\right)^2}}\right] \qquad \text{(\textit{Spec}. Eq. E4-3)}$$

$$= \left[\frac{30.0\text{ ksi} + 165\text{ ksi}}{2(0.915)}\right]\left[1 - \sqrt{1 - \frac{4(30.0\text{ ksi})(165\text{ ksi})(0.915)}{(30.0\text{ ksi} + 165\text{ ksi})^2}}\right]$$

$$= 29.5\text{ ksi}$$

Critical Buckling Stress

The critical buckling stress for the member could be controlled by flexural buckling about either the *x-x* axis or *y-y* axis, F_{ex} or F_{ey}, respectively. Note that AISC *Specification* Equations E4-5 and E4-6 reflect the same buckling modes as calculated in AISC *Specification* Equation E3-4. Or, the critical buckling stress for the member could be controlled by torsional or flexural-torsional buckling calculated per AISC *Specification* Equation E4-3. In this example, F_e calculated in accordance with AISC *Specification* Equation E4-5 is less than that calculated in accordance with AISC *Specification* Equation E4-3 or E4-6 and controls. Therefore:

$$F_e = 16.2\text{ ksi}$$

$$\frac{F_y}{F_e} = \frac{50\text{ ksi}}{16.2\text{ ksi}}$$

$$= 3.09$$

Per the AISC *Specification* User Note for Section E3, the two inequalities for calculating limits of applicability of Sections E3(a) and E3(b) provide the same result for flexural buckling only. When the elastic buckling stress, F_e, is controlled by torsional or flexural-torsional buckling, the L_c/r limits would not be applicable unless an equivalent L_c/r ratio is first calculated by substituting the governing F_e into AISC *Specification* Equation E3-4 and solving for L_c/r. The F_y/F_e limits may be used regardless of which buckling mode governs.

Because $\dfrac{F_y}{F_e} > 2.25$:

$$\begin{aligned} F_{cr} &= 0.877F_e \\ &= 0.877(16.2\text{ ksi}) \\ &= 14.2\text{ ksi} \end{aligned} \qquad \text{(}\textit{Spec}\text{. Eq. E3-3)}$$

Available Compressive Strength

$$\begin{aligned} P_n &= F_{cr}A_g \\ &= (14.2\text{ ksi})\left(10.0\text{ in.}^2\right) \\ &= 142\text{ kips} \end{aligned} \qquad \text{(}\textit{Spec}\text{. Eq. E3-1)}$$

From AISC *Specification* Section E1, the available compressive strength is:

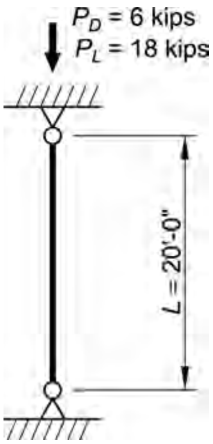
LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(142\text{ kips})$ $= 128\text{ kips} > 120\text{ kips}$ **o.k.**	$\Omega_c = 1.67$ $\dfrac{P_n}{\Omega_c} = \dfrac{142\text{ kips}}{1.67}$ $= 85.0\text{ kips} > 80.0\text{ kips}$ **o.k.**

EXAMPLE E.8 WT COMPRESSION MEMBER WITH SLENDER ELEMENTS

Given:

Select an ASTM A992 WT-shape compression member with a length of 20 ft to support a dead load of 6 kips and live load of 18 kips in axial compression. The ends are pinned. The solution will be provided using:

(1) AISC *Manual* Tables
(2) Calculations using AISC *Specification* provisions



Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From ASCE/SEI 7, Chapter 2 , the required compressive strength is:

LRFD	ASD
$P_u = 1.2(6\text{ kips}) + 1.6(18\text{ kips})$ $= 36.0\text{ kips}$	$P_a = 6\text{ kips} + 18\text{ kips}$ $= 24.0\text{ kips}$

(1) AISC *Manual* Table Solution

From AISC *Specification* Commentary Table C-A-7.1, for a pinned-pinned condition, K = 1.0. Therefore, L_{cx} = L_{cy} = KL = 1.0(20 ft) = 20.0 ft.

Select the lightest member from AISC *Manual* Table 4-7 with sufficient available strength about the both the x-x axis (upper portion of the table) and the y-y axis (lower portion of the table) to support the required strength.

Try a WT7×15.

The available strength in axial compression from AISC *Manual* Table 4-7 is:

LRFD	ASD
$\phi_c P_{nx} = 74.3\text{ kips} > 36.0\text{ kips}$ **o.k.**	$\dfrac{P_{nx}}{\Omega_c} = 49.4\text{ kips} > 24.0\text{ kips}$ **o.k.**
$\phi_c P_{ny} = 36.6\text{ kips} > 36.0\text{ kips}$ **o.k.** **controls**	$\dfrac{P_{ny}}{\Omega_c} = 24.4\text{ kips} > 24.0\text{ kips}$ **o.k.** **controls**

Available strength can also be determined by hand calculations, as demonstrated in the following.

(2) Calculation Using AISC *Specification* Provisions

From AISC *Manual* Table 1-8, the geometric properties are as follows:

WT7×15
A_g = 4.42 in.^{2}
r_x = 2.07 in.
r_y = 1.49 in.

J = 0.190 in.^{4}
$\bar{y}$ = 1.58 in.
I_x = 19.0 in.^{4}
I_y = 9.79 in.^{4}
d = 6.92 in.
t_w = 0.270 in.
b_f = 6.73 in.
t_f = 0.385 in.

Stem Slenderness Check

$$\begin{aligned}\lambda &= \frac{d}{t_w} \\ &= \frac{6.92\text{ in.}}{0.270\text{ in.}} \\ &= 25.6\end{aligned}$$

Determine stem limiting slenderness ratio, λ_r, from AISC *Specification* Table B4.1a, Case 4:

$$\begin{aligned}\lambda_r &= 0.75\sqrt{\frac{E}{F_y}} \\ &= 0.75\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} \\ &= 18.1\end{aligned}$$

$\lambda > \lambda_r$; therefore, the stem is slender

Flange Slenderness Check

$$\begin{aligned}\lambda &= \frac{b_f}{2t_f} \\ &= \frac{6.73\text{ in.}}{2(0.385\text{ in.})} \\ &= 8.74\end{aligned}$$

Determine flange limiting slenderness ratio, λ_r, from AISC *Specification* Table B4.1a, Case 1:

$$\begin{aligned}\lambda_r &= 0.56\sqrt{\frac{E}{F_y}} \\ &= 0.56\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} \\ &= 13.5\end{aligned}$$

$\lambda < \lambda_r$; therefore, the flange is not slender

Because this WT7×15 has a slender web, AISC *Specification* Section E7 is applicable. The nominal compressive strength, P_n, is determined based on the limit states of flexural, torsional and flexural-torsional buckling.

Elastic Flexural Buckling Stress about the x-x Axis

$$\begin{aligned}\frac{L_{cx}}{r_x} &= \frac{(20.0\text{ ft})(12\text{ in./ft})}{2.07\text{ in.}}\\ &= 116\end{aligned}$$

$$\begin{aligned}F_{ex} &= \frac{\pi^2 E}{\left(\dfrac{L_{cx}}{r_x}\right)^2}\\ &= \frac{\pi^2(29{,}000\text{ ksi})}{(116)^2}\\ &= 21.3\end{aligned}$$ (*Spec*. Eq. E3-4 or E4-5)

Elastic Flexural Buckling Stress about the y-y Axis

$$\begin{aligned}\frac{L_{cy}}{r_y} &= \frac{(20.0\text{ ft})(12\text{ in./ft})}{1.49\text{ in.}}\\ &= 161\end{aligned}$$

$$\begin{aligned}F_{ey} &= \frac{\pi^2 E}{\left(\dfrac{L_{cy}}{r_y}\right)^2}\\ &= \frac{\pi^2(29{,}000\text{ ksi})}{(161)^2}\\ &= 11.0\text{ ksi}\end{aligned}$$ (*Spec*. Eq. E3-4 or E4-6)

Torsional and Flexural-Torsional Elastic Buckling Stress

F_e will be calculated using AISC *Specification* Equation E4-3. Per the User Note for AISC *Specification* Section E4, the term with C_w is omitted when computing F_{ez}, and x_o is taken as zero. The flexural buckling term about the *y-y* axis, F_{ey}, was computed in the preceding section.

$$x_o = 0$$

$$\begin{aligned}y_o &= \bar{y} - \frac{t_f}{2}\\ &= 1.58\text{ in.} - \frac{0.385\text{ in.}}{2}\\ &= 1.39\text{ in.}\end{aligned}$$

$$\begin{aligned}\bar{r}_o^{\,2} &= x_o^{\,2} + y_o^{\,2} + \frac{I_x + I_y}{A_g}\\ &= 0 + (1.39\text{ in.})^2 + \frac{19.0\text{ in.}^4 + 9.79\text{ in.}^4}{4.42\text{ in.}^2}\\ &= 8.45\text{ in.}^2\end{aligned}$$ (*Spec*. Eq. E4-9)

$$F_{ez} = \left(\frac{\pi^2 E C_w}{L_{cz}^{\,2}} + GJ\right)\frac{1}{A_g \bar{r}_o^{\,2}} \qquad (\textit{Spec}.\ \text{Eq. E4-7})$$

$$= \left[0 + (11{,}200\text{ ksi})(0.190\text{ in.}^4)\right]\frac{1}{(4.42\text{ in.}^2)(8.45\text{ in.}^2)}$$

$$= 57.0\text{ ksi}$$

$$H = 1 - \frac{x_o^{\,2} + y_o^{\,2}}{\bar{r}_o^{\,2}} \qquad (\textit{Spec}.\ \text{Eq. E4-8})$$

$$= 1 - \frac{0 + (1.39\text{ in.})^2}{8.45\text{ in.}^2}$$

$$= 0.771$$

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2}}\right] \qquad (\textit{Spec}.\ \text{Eq. E4-3})$$

$$= \left[\frac{11.0\text{ ksi} + 57.0\text{ ksi}}{2(0.771)}\right]\left[1 - \sqrt{1 - \frac{4(11.0\text{ ksi})(57.0\text{ ksi})(0.771)}{(11.0\text{ ksi} + 57.0\text{ ksi})^2}}\right]$$

$$= 10.5\text{ ksi} \quad \textbf{controls}$$

Critical Buckling Stress

The critical buckling stress for the member could be controlled by flexural buckling about either the *x-x* axis or *y-y* axis, F_{ex} or F_{ey}, respectively. Note that AISC *Specification* Equations E4-5 and E4-6 reflect the same buckling modes as calculated in AISC *Specification* Equation E3-4. Or, the critical buckling stress for the member could be controlled by torsional or flexural-torsional buckling calculated per AISC *Specification* Equation E4-3. In this example, F_e calculated in accordance with AISC *Specification* Equation E4-3 is less than that calculated in accordance with AISC *Specification* Equation E4-5 or E4-6 and controls. Therefore:

$$F_e = 10.5\text{ ksi}$$

$$\frac{F_y}{F_e} = \frac{50\text{ ksi}}{10.5\text{ ksi}}$$

$$= 4.76$$

Per the AISC *Specification* User Note for Section E3, the two inequalities for calculating limits of applicability of Sections E3(a) and E3(b) provide the same result for flexural buckling only. When the elastic buckling stress, F_e, is controlled by torsional or flexural-torsional buckling, the L_c/r limits would not be applicable unless an equivalent L_c/r ratio is first calculated by substituting the governing F_e into AISC *Specification* Equation E3-4 and solving for L_c/r. The F_y/F_e limits may be used regardless of which buckling mode governs.

Because $\dfrac{F_y}{F_e} > 2.25$:

$$F_{cr} = 0.877F_e \qquad (\textit{Spec}.\ \text{Eq. E3-3})$$

$$= 0.877(10.5\text{ ksi})$$

$$= 9.21\text{ ksi}$$

Effective Area

Because this section was found to have a slender element, the limits of AISC *Specification* Section E7.1 must be evaluated to determine if there is a reduction in effective area due to local buckling. Since the flange was found to not be slender, no reduction in effective area due to local buckling in the flange is required. Only a reduction in effective area due to local buckling in the stem may be required.

$$\lambda = 25.6$$

$$\begin{aligned}\lambda_r\sqrt{\frac{F_y}{F_{cr}}} &= 18.1\sqrt{\frac{50\text{ ksi}}{9.21\text{ ksi}}}\\ &= 42.2\end{aligned}$$

Because $\lambda < \lambda_r\sqrt{\frac{F_y}{F_{cr}}}$,

$$b_e = b \qquad \text{(}\textit{Spec}\text{. Eq. E7-2)}$$

There is no reduction in effective area due to local buckling of the stem at the critical stress level and $A_e = A_g$.

Available Compressive Strength

$$\begin{aligned}P_n &= F_{cr}A_e \qquad \text{(}\textit{Spec}\text{. Eq. E7-1)}\\ &= (9.21\text{ ksi})(4.42\text{ in.}^2)\\ &= 40.7\text{ kips}\end{aligned}$$

From AISC *Specification* Section E1, the available compressive strength is:

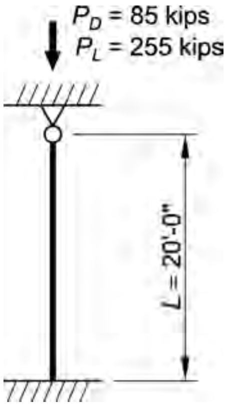
LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(40.7\text{ kips})$ $= 36.6\text{ kips} > 36.0\text{ kips}$ **o.k.**	$\Omega_c = 1.67$ $\frac{P_n}{\Omega_c} = \frac{40.7\text{ kips}}{1.67}$ $= 24.4\text{ kips} > 24.0\text{ kips}$ **o.k.**

EXAMPLE E.9 RECTANGULAR HSS COMPRESSION MEMBER WITHOUT SLENDER ELEMENTS

Given:

Select an ASTM A500 Grade C rectangular HSS compression member, with a length of 20 ft, to support a dead load of 85 kips and live load of 255 kips in axial compression. The base is fixed and the top is pinned. The solution will be provided using:

(1) AISC *Manual* Tables
(2) Calculations using AISC *Specification* provisions



Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500 Grade C, rectangular HSS
F_y = 50 ksi
F_u = 62 ksi

From ASCE/SEI 7, Chapter 2, the required compressive strength is:

LRFD	ASD
$P_u = 1.2(85\text{ kips}) + 1.6(255\text{ kips})$ $= 510\text{ kips}$	$P_a = 85\text{ kips} + 255\text{ kips}$ $= 340\text{ kips}$

(1) AISC *Manual* Table Solution

From AISC *Specification* Commentary Table C-A-7.1, for a fixed-pinned condition, $K_x = K_y = 0.80$.

$$\begin{aligned} L_c &= K_x L_x \\ &= K_y L_y \\ &= 0.80(20\text{ ft}) \\ &= 16.0\text{ ft} \end{aligned}$$

Enter AISC *Manual* Table 4-3 for rectangular sections.

Try a HSS12×10×⅜.

From AISC *Manual* Table 4-3, the available strength in axial compression is:

LRFD	ASD
$\phi_c P_n = 556\text{ kips} > 510\text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_c} = 370\text{ kips} > 340\text{ kips}$ **o.k.**

Available strength can also be determined by hand calculations, as demonstrated in the following.

(2) Calculation Using AISC *Specification* Provisions

From AISC *Manual* Table 1-11, the geometric properties are as follows:

HSS12×10×⅜

A_g = 14.6 in.^{2}
t = 0.349 in.
r_x = 4.61 in.
r_y = 4.01 in.
b/t = 25.7
h/t = 31.4

Slenderness Check

Determine the wall limiting slenderness ratio, λ_r, from AISC *Specification* Table B4.1a, Case 6:

$$\begin{aligned}\lambda_r &= 1.40\sqrt{\frac{E}{F_y}}\\ &= 1.40\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}\\ &= 33.7\end{aligned}$$

For the narrow side:

$$\lambda = b/t = 25.7$$

For the wide side:

$$\lambda = h/t = 31.4$$

$\lambda < \lambda_r$; therefore, the section does not contain slender elements.

Elastic Buckling Stress

Because $r_y < r_x$ and $L_{cx} = L_{cy}$, r_y will govern the available strength.

Determine the applicable equation:

$$\begin{aligned}\frac{L_{cy}}{r_y} &= \frac{(16.0\text{ ft})(12\text{ in./ft})}{4.01\text{ in.}}\\ &= 47.9\end{aligned}$$

$$\begin{aligned}4.71\sqrt{\frac{E}{F_y}} &= 4.71\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}\\ &= 113 \geq 47.9\end{aligned}$$

Therefore, use AISC *Specification* Equation E3-2.

$$\begin{aligned}F_e &= \frac{\pi^2 E}{\left(\dfrac{L_c}{r}\right)^2} \qquad (\textit{Spec}.\text{ Eq. E3-4})\\ &= \frac{\pi^2(29{,}000\text{ ksi})}{(47.9)^2}\\ &= 125\text{ ksi}\end{aligned}$$

Critical Buckling Stress

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) F_y \qquad \text{(}\textit{Spec}\text{. Eq. E3-2)}$$

$$= \left(0.658^{\frac{50\text{ ksi}}{125\text{ ksi}}}\right)(50\text{ ksi})$$

$$= 42.3\text{ ksi}$$

Available Compressive Strength

$$P_n = F_{cr} A_g \qquad \text{(}\textit{Spec}\text{. Eq. E3-1)}$$

$$= (42.3\text{ ksi})(14.6\text{ in.}^2)$$

$$= 618\text{ kips}$$

From AISC *Specification* Section E1, the available compressive strength is:

LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(618\text{ kips})$ $= 556\text{ kips} > 510\text{ kips}$ **o.k.**	$\Omega_c = 1.67$ $\dfrac{P_n}{\Omega_c} = \dfrac{618\text{ kips}}{1.67}$ $= 370\text{ kips} > 340\text{ kips}$ **o.k.**

EXAMPLE E.10 RECTANGULAR HSS COMPRESSION MEMBER WITH SLENDER ELEMENTS

Given:

Using the AISC *Specification* provisions, calculate the available strength of a HSS12×8×3⁄16 compression member with an effective length of L_c = 24 ft with respect to both axes. Use ASTM A500 Grade C.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500 Grade C, rectangular HSS
F_y = 50 ksi
F_u = 62 ksi

From AISC *Manual* Table 1-11 the geometric properties of an HSS12×8×3⁄16 are as follows:

$A = 6.76 \text{ in.}^2$
$t = 0.174 \text{ in.}$
$r_x = 4.56 \text{ in.}$
$r_y = 3.35 \text{ in.}$
$\dfrac{b}{t} = 43.0$
$\dfrac{h}{t} = 66.0$

Slenderness Check

Calculate the limiting slenderness ratio, λ_r, from AISC *Specification* Table B4.1a, Case 6 for walls of rectangular HSS.

$$\begin{aligned}\lambda_r &= 1.40\sqrt{\frac{E}{F_y}}\\ &= 1.40\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}}\\ &= 33.7\end{aligned}$$

Determine the width-to-thickness ratios of the HSS walls.

For the narrow side:

$$\begin{aligned}\lambda &= \frac{b}{t}\\ &= 43.0 > \lambda_r = 33.7\end{aligned}$$

For the wide side:

$$\begin{aligned}\lambda &= \frac{h}{t}\\ &= 66.0 > \lambda_r = 33.7\end{aligned}$$

All walls of the HSS12×8×3⁄16 are slender elements and the provisions of AISC *Specification* Section E7 apply.

Critical Stress, F_{cr}

From AISC *Specification* Section E7, the critical stress, F_{cr}, is calculated using the gross section properties and following the provisions of AISC *Specification* Section E3. The effective slenderness ratio about the y-axis will control. From AISC *Specification* Commentary Table C-A-7.1, for a pinned-pinned condition, K = 1.0. Therefore, $L_{cy} = K_yL_y$ = 1.0(24 ft) = 24.0 ft.

$$\begin{aligned}\left(\frac{L_c}{r}\right)_{max} &= \frac{L_{cy}}{r_y}\\ &= \frac{(24.0\text{ ft})(12\text{ in./ft})}{3.35\text{ in.}}\\ &= 86.0\end{aligned}$$

$$\begin{aligned}4.71\sqrt{\frac{E}{F_y}} &= 4.71\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}\\ &= 113 \geq 86.0\end{aligned}$$

Therefore, use AISC *Specification* Equation E3-2.

$$\begin{aligned}F_e &= \frac{\pi^2 E}{\left(\frac{L_c}{r}\right)^2} \qquad \text{(\textit{Spec}. Eq. E3-4)}\\ &= \frac{\pi^2(29{,}000\text{ ksi})}{(86.0)^2}\\ &= 38.7\text{ ksi}\end{aligned}$$

$$\begin{aligned}F_{cr} &= \left(0.658^{\frac{F_y}{F_e}}\right)F_y \qquad \text{(\textit{Spec}. Eq. E3-2)}\\ &= \left[0.658^{\left(\frac{50\text{ ksi}}{38.7\text{ ksi}}\right)}\right](50\text{ ksi})\\ &= 29.1\text{ ksi}\end{aligned}$$

Effective Area, A_e

Compute the effective wall widths, h_e and b_e, in accordance with AISC *Specification* Section E7.1. Compare λ for each wall with the following limit to determine if a local buckling reduction applies.

$$\begin{aligned}\lambda_r\sqrt{\frac{F_y}{F_{cr}}} &= 33.7\sqrt{\frac{50\text{ ksi}}{29.1\text{ ksi}}}\\ &= 44.2\end{aligned}$$

For the narrow walls:

$$\begin{aligned}\lambda &= \frac{b}{t}\\ &= 43.0 < 44.2\end{aligned}$$

Therefore, the narrow wall width does not need to be reduced ($b_e = b$) per AISC *Specification* Equation E7-2.

For the wide walls:

$$\begin{aligned}\lambda &= \frac{h}{t}\\ &= 66.0 > 44.2\end{aligned}$$

Therefore, use AISC *Specification* Equation E7-3, with $h = \left(\frac{h}{t}\right)t = (66.0)(0.174\text{ in.}) = 11.5\text{ in.}$

The effective width imperfection adjustment factors, c_1 and c_2, are selected from AISC *Specification* Table E7.1, Case (b):

$$\begin{aligned}c_1 &= 0.20\\ c_2 &= 1.38\end{aligned}$$

$$\begin{aligned}F_{el} &= \left(c_2\frac{\lambda_r}{\lambda}\right)^2 F_y &&\text{(}Spec.\text{ Eq. E7-5)}\\ &= \left[1.38\left(\frac{33.7}{66.0}\right)\right]^2(50\text{ ksi})\\ &= 24.8\text{ ksi}\end{aligned}$$

$$\begin{aligned}h_e &= h\left(1-c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} &&\text{(}Spec.\text{ Eq. E7-3)}\\ &= (11.5\text{ in.})\left(1-0.20\sqrt{\frac{24.8\text{ ksi}}{29.1\text{ ksi}}}\right)\sqrt{\frac{24.8\text{ ksi}}{29.1\text{ ksi}}}\\ &= 8.66\text{ in.}\end{aligned}$$

The effective area, A_e, is determined using the effective width h_e = 8.66 in. and the design wall thickness t = 0.174 in. As shown in Figure E.10-1, $h - h_e$ is the width of the wall segments that must be reduced from the gross area, A, to compute the effective area, A_e. Note that a similar deduction would be required for the narrow walls if $b_e < b$.

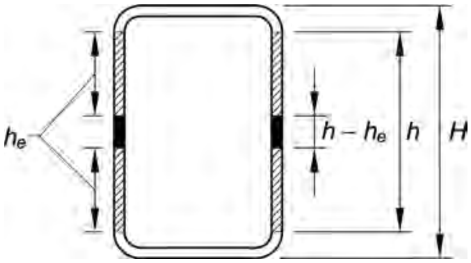


Fig. E.10-1. HSS Effective Area.

$$\begin{aligned}A_e &= A - 2(h - h_e)t\\ &= 6.76\text{ in.}^2 - 2(11.5\text{ in.} - 8.66\text{ in.})(0.174\text{ in.})\\ &= 5.77\text{ in.}^2\end{aligned}$$

Available Compressive Strength

The effective area is used to compute nominal compressive strength:

$$P_n = F_{cr} A_e \qquad (Spec.\ \text{Eq. E7-1})$$

$$= (29.1\text{ ksi})(5.77\text{ in.}^2)$$

$$= 168\text{ kips}$$

From AISC *Specification* Section E1, the available compressive strength is:

LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(168\text{ kips})$ $= 151\text{ kips}$	$\Omega_c = 1.67$ $\dfrac{P_n}{\Omega_c} = \dfrac{168\text{ kips}}{1.67}$ $= 101\text{ kips}$

Discussion

The width-to-thickness criterion, $\lambda_r = 1.40\sqrt{\dfrac{E}{F_y}}$ for HSS in Table B4.1a is based on the assumption that the element will be stressed to F_y. If the critical flexural buckling stress is less than F_y, which it always is for compression members of reasonable length, wall local buckling may or may not occur before member flexural buckling occurs. For the case where the flexural buckling stress is low enough, wall local buckling will not occur. This is the case addressed in AISC *Specification* Section E7.1(a). For members where the flexural buckling stress is high enough, wall local buckling will occur. This is the case addressed in AISC *Specification* Section E7.1(b).

The HSS12×8×3⁄16 in this example is slender according to Table B4.1a. For effective length L_c = 24.0 ft, the flexural buckling critical stress was F_{cr} = 29.1 ksi. By Section E7.1, at F_{cr} = 29.1 ksi, the wide wall effective width must be determined but the narrow wall is fully effective. Thus, the axial strength is reduced because of local buckling of the wide wall. Table E.10 repeats the example analysis for two other column effective lengths and compares those results to the results for L_c = 24 ft calculated previously. For L_c = 18.0 ft, the flexural buckling critical stress, F_{cr} = 36.9 ksi, is high enough that both the wide and narrow walls must have their effective width determined according to Equation E7-3. For L_c = 40.0 ft the flexural buckling critical stress, F_{cr} = 12.2 ksi, is low enough that there will be no local buckling of either wall and the actual widths will be used according to Equation E7-2.

Table E.10. Analysis of HSS12×8×3⁄16 Column at Different Effective Lengths

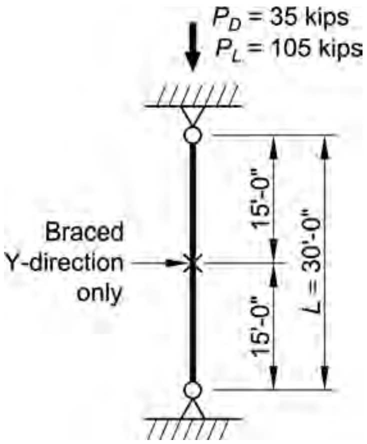
Effective length, L_c (ft)	18.0	24.0	40.0
Check Table B4.1 criterion (same as for L_c = 24.0 ft).			
λ_r	33.7	33.7	33.7
λ (narrow wall) = 43.0 > λ_r	Yes	Yes	Yes
λ (wide wall) = 66.0 > λ_r	Yes	Yes	Yes
F_{cr} (ksi)	36.9	29.1	12.2
Check AISC *Specification* Section E7.1 criteria.			
Narrow wall:			
$\lambda_r\sqrt{\dfrac{F_y}{F_{cr}}}$	$39.2 \le \lambda = 43.0$	$44.2 > \lambda = 43.0$	$68.2 > \lambda = 43.0$
Local buckling reduction per AISC *Specification* Section E7.1?	Yes	No	No
F_{el} (ksi)	58.5	–	–
b_e (in.)	7.05	–	–
Wide wall:			
$\lambda_r\sqrt{\dfrac{F_y}{F_{cr}}}$	$39.2 \le \lambda = 66.0$	$44.2 \le \lambda = 66.0$	$68.2 > \lambda = 66.0$
Local buckling reduction per AISC *Specification* Section E7.1?	Yes	Yes	No
F_{el} (ksi)	24.8	24.8	–
h_e (in.)	7.88	8.66	–
Effective area, A_e (in.^{2})	5.35	5.77	6.76
Compressive strength			
P_n (kips)	197	168	82.5
LRFD, $\phi_c P_n$ (kips)	177	151	74.2
ASD, P_n/Ω_c (kips)	118	101	49.4

EXAMPLE E.11 PIPE COMPRESSION MEMBER

Given:

Select an ASTM A53 Grade B Pipe compression member with a length of 30 ft to support a dead load of 35 kips and live load of 105 kips in axial compression. The column is pin-connected at the ends in both axes and braced at the midpoint in the *y-y* direction. The solution will be provided using:

(1) AISC *Manual* Tables
(2) Calculations using AISC *Specification* provisions



Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A53 Grade B
F_y = 35 ksi
F_u = 60 ksi

From ASCE/SEI 7, Chapter 2, the required compressive strength is:

LRFD	ASD
$P_u = 1.2(35\text{ kips}) + 1.6(105\text{ kips})$ $= 210\text{ kips}$	$P_a = 35\text{ kips} + 105\text{ kips}$ $= 140\text{ kips}$

(1) AISC *Manual* Table Solution

From AISC *Specification* Commentary Table C-A-7.1, for a pinned-pinned condition, K = 1.0. Therefore, $L_{cx} = K_xL_x$ = 1.0(30 ft) = 30.0 ft and $L_{cy} = K_yL_y$ = 1.0(15 ft) = 15.0 ft. Buckling about the *x-x* axis controls.

Enter AISC *Manual* Table 4-6 with L_c = 30.0 ft and select the lightest section with sufficient available strength to support the required strength.

Try a 10-in. Standard Pipe.

From AISC *Manual* Table 4-6, the available strength in axial compression is:

LRFD	ASD
$\phi_c P_n = 222\text{ kips} > 210\text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_c} = 148\text{ kips} > 140\text{ kips}$ **o.k.**

Available strength can also be determined by hand calculations, as demonstrated in the following.

(2) Calculation Using AISC *Specification* Provisions

From AISC *Manual* Table 1-14, the geometric properties are as follows:

Pipe 10 Std.
A_g = 11.5 in.^{2}
r = 3.68 in.

$$\lambda = \frac{D}{t} = 31.6$$

No Pipes shown in AISC *Manual* Table 4-6 are slender at 35 ksi, so no local buckling check is required; however, some round HSS are slender at higher steel strengths. The following calculations illustrate the required check.

Limiting Width-to-Thickness Ratio

Determine the wall limiting slenderness ratio, λ_r, from AISC *Specification* Table B4.1a, Case 9:

$$\begin{aligned}\lambda_r &= 0.11\frac{E}{F_y}\\ &= 0.11\left(\frac{29{,}000\text{ ksi}}{35\text{ ksi}}\right)\\ &= 91.1\end{aligned}$$

$\lambda < \lambda_r$; therefore, the pipe is not slender

Critical Stress, F_{cr}

$$\begin{aligned}\frac{L_c}{r} &= \frac{(30.0\text{ ft})(12\text{ in./ft})}{3.68\text{ in.}}\\ &= 97.8\end{aligned}$$

$$\begin{aligned}4.71\sqrt{\frac{E}{F_y}} &= 4.71\sqrt{\frac{29{,}000\text{ ksi}}{35\text{ ksi}}}\\ &= 136 > 97.8,\ \text{therefore, use AISC } \textit{Specification} \text{ Equation E3-2}\end{aligned}$$

$$\begin{aligned}F_e &= \frac{\pi^2 E}{\left(\frac{L_c}{r}\right)^2}\\ &= \frac{\pi^2(29{,}000\text{ ksi})}{(97.8)^2}\\ &= 29.9\text{ ksi}\end{aligned}$$

(*Spec*. Eq. E3-4)

$$\begin{aligned}F_{cr} &= \left(0.658^{\frac{F_y}{F_e}}\right)F_y\\ &= \left[0.658^{\left(\frac{35\text{ ksi}}{29.9\text{ ksi}}\right)}\right](35\text{ ksi})\\ &= 21.4\text{ ksi}\end{aligned}$$

(*Spec*. Eq. E3-2)

Available Compressive Strength

$$\begin{aligned}P_n &= F_{cr}A_g\\ &= (21.4\text{ ksi})(11.5\text{ in.}^2)\\ &= 246\text{ kips}\end{aligned}$$

(*Spec*. Eq. E3-1)

From AISC *Specification* Section E1, the available compressive strength is:

LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(246 \text{ kips})$ $= 221 \text{ kips} > 210 \text{ kips}$ **o.k.**	$\Omega_c = 1.67$ $\dfrac{P_n}{\Omega_c} = \dfrac{246 \text{ kips}}{1.67}$ $= 147 \text{ kips} > 140 \text{ kips}$ **o.k.**

Note that the design procedure would be similar for a round HSS column.

EXAMPLE E.12 BUILT-UP I-SHAPED MEMBER WITH DIFFERENT FLANGE SIZES

Given:

Compute the available strength of a built-up compression member with a length of 14 ft, as shown in Figure E.12-1. The ends are pinned. The outside flange is PL¾ in. × 5 in., the inside flange is PL¾ in. × 8 in., and the web is PL⅜ in. × 10½ in. The material is ASTM A572 Grade 50.

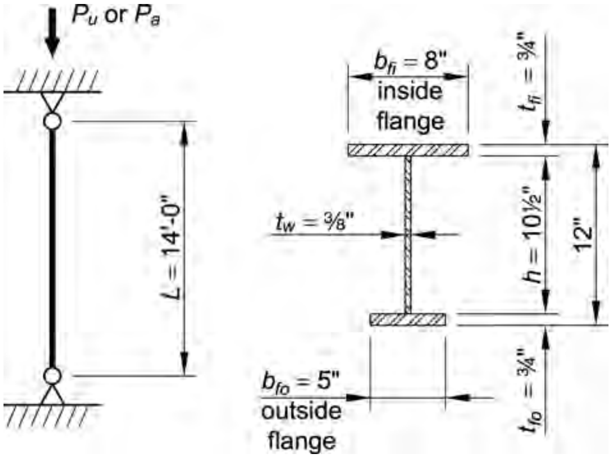


Fig. E.12-1. Column geometry for Example E.12.

Solution:

From AISC *Manual* Table 2-5, the material properties are as follows:

ASTM A572 Grade 50
F_y = 50 ksi
F_u = 65 ksi

There are no tables for special built-up shapes; therefore, the available strength is calculated as follows.

Slenderness Check

Check outside flange slenderness.

From AISC *Specification* Table B4.1a note [a], calculate k_c.

$$\begin{aligned}k_c &= \frac{4}{\sqrt{h/t_w}}\\ &= \frac{4}{\sqrt{\dfrac{10\tfrac{1}{2}\text{ in.}}{\tfrac{3}{8}\text{ in.}}}}\\ &= 0.756,\ \ 0.35 \le k_c \le 0.76 \quad \textbf{o.k.}\end{aligned}$$

For the outside flange, the slenderness ratio is:

$$\begin{aligned}\lambda &= \frac{b}{t}\\ &= \frac{2.50\text{ in.}}{\tfrac{3}{4}\text{ in.}}\\ &= 3.33\end{aligned}$$

Determine the limiting slenderness ratio, λ_r, from AISC *Specification* Table B4.1a, Case 2:

$$\begin{aligned}\lambda_r &= 0.64\sqrt{\frac{k_c E}{F_y}}\\ &= 0.64\sqrt{\frac{0.756(29{,}000\text{ ksi})}{50\text{ ksi}}}\\ &= 13.4\end{aligned}$$

$\lambda \le \lambda_r$; therefore, the outside flange is not slender

Check inside flange slenderness.

$$\begin{aligned}\lambda &= \frac{b}{t}\\ &= \frac{4.00\text{ in.}}{\tfrac{3}{4}\text{ in.}}\\ &= 5.33\end{aligned}$$

$\lambda \le \lambda_r$; therefore, the inside flange is not slender

Check web slenderness.

$$\begin{aligned}\lambda &= \frac{h}{t}\\ &= \frac{10\tfrac{1}{2}\text{ in.}}{\tfrac{3}{8}\text{ in.}}\\ &= 28.0\end{aligned}$$

Determine the limiting slenderness ratio, λ_r, for the web from AISC *Specification* Table B4.1a, Case 5:

$$\begin{aligned}\lambda_r &= 1.49\sqrt{\frac{E}{F_y}}\\ &= 1.49\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}\\ &= 35.9\end{aligned}$$

$\lambda \le \lambda_r$; therefore, the web is not slender

Section Properties (ignoring welds)

$$
\begin{aligned}
A_g &= b_{fi}t_{fi} + ht_w + b_{fo}t_{fo} \\
&= (8.00\text{ in.})(\tfrac{3}{4}\text{ in.}) + (10\tfrac{1}{2}\text{ in.})(\tfrac{3}{8}\text{ in.}) + (5.00\text{ in.})(\tfrac{3}{4}\text{ in.}) \\
&= 13.7\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
\bar{y} &= \frac{\Sigma A_i y_i}{\Sigma A_i} \\
&= \frac{(6.00\text{ in.}^2)(11.6\text{ in.}) + (3.94\text{ in.}^2)(6.00\text{ in.}) + (3.75\text{ in.}^2)(0.375\text{ in.})}{6.00\text{ in.}^2 + 3.94\text{ in.}^2 + 3.75\text{ in.}^2} \\
&= 6.91\text{ in.}
\end{aligned}
$$

Note that the center of gravity about the *x*-axis is measured from the bottom of the outside flange.

$$
\begin{aligned}
I_x &= \sum\left(\frac{bh^3}{12} + Ad^2\right) \\
&= \left[\frac{(8.00\text{ in.})(\tfrac{3}{4}\text{ in.})^3}{12} + (8.00\text{ in.})(\tfrac{3}{4}\text{ in.})(4.72\text{ in.})^2\right] + \left[\frac{(\tfrac{3}{8}\text{ in.})(10\tfrac{1}{2}\text{ in.})^3}{12} + (\tfrac{3}{8}\text{ in.})(10\tfrac{1}{2}\text{ in.})(0.910\text{ in.})^2\right] \\
&\quad + \left[\frac{(5.00\text{ in.})(\tfrac{3}{4}\text{ in.})^3}{12} + (5.00\text{ in.})(\tfrac{3}{4}\text{ in.})(6.54\text{ in.})^2\right] \\
&= 334\text{ in.}^4
\end{aligned}
$$

$$
\begin{aligned}
r_x &= \sqrt{\frac{I_x}{A}} \\
&= \sqrt{\frac{334\text{ in.}^4}{13.7\text{ in.}^2}} \\
&= 4.94\text{ in.}
\end{aligned}
$$

$$
\begin{aligned}
I_y &= \sum\frac{bh^3}{12} \\
&= \frac{(\tfrac{3}{4}\text{ in.})(8.00\text{ in.})^3}{12} + \frac{(10\tfrac{1}{2}\text{ in.})(\tfrac{3}{8}\text{ in.})^3}{12} + \frac{(\tfrac{3}{4}\text{ in.})(5.00\text{ in.})^3}{12} \\
&= 39.9\text{ in.}^4
\end{aligned}
$$

$$
\begin{aligned}
r_y &= \sqrt{\frac{I_y}{A}} \\
&= \sqrt{\frac{39.9\text{ in.}^4}{13.7\text{ in.}^2}} \\
&= 1.71\text{ in.}
\end{aligned}
$$

Elastic Buckling Stress about the x-x Axis

From AISC *Specification* Commentary Table C-A-7.1, for a pinned-pinned condition, K = 1.0. Therefore, $L_{cx} = L_{cy} = L_{cz} = KL$ = 1.0(14 ft) = 14.0 ft.

The effective slenderness ratio about the *x*-axis is:

$$\frac{L_{cx}}{r_x} = \frac{(14.0\text{ ft})(12\text{ in./ft})}{4.94\text{ in.}}$$
$$= 34.0$$

$$F_e = \frac{\pi^2 E}{\left(\dfrac{L_c}{r}\right)^2} \qquad (\textit{Spec}.\text{ Eq. E3-4})$$
$$= \frac{\pi^2 (29{,}000\text{ ksi})}{(34.0)^2}$$
$$= 248\text{ ksi} \quad \textbf{does not control}$$

Flexural-Torsional Elastic Buckling Stress

Calculate the torsional constant, J, using AISC Design Guide 9, Equation 3.4:

$$J = \sum \frac{bt^3}{3}$$
$$= \frac{(8.00\text{ in.})(3/4\text{ in.})^3}{3} + \frac{(10\tfrac{1}{2}\text{ in.})(3/8\text{ in.})^3}{3} + \frac{(5.00\text{ in.})(3/4\text{ in.})^3}{3}$$
$$= 2.01\text{ in.}^4$$

Distance between flange centroids:

$$h_o = d - \frac{t_{fi}}{2} - \frac{t_{fo}}{2}$$
$$= 12.0\text{ in.} - \frac{3/4\text{ in.}}{2} - \frac{3/4\text{ in.}}{2}$$
$$= 11.3\text{ in.}$$

Warping constant:

$$C_w = \frac{t_f h_o^2}{12}\left(\frac{b_{fi}^3 b_{fo}^3}{b_{fi}^3 + b_{fo}^3}\right)$$
$$= \frac{(3/4\text{ in.})(11.3\text{ in.})^2}{12}\left[\frac{(8.00\text{ in.})^3(5.00\text{ in.})^3}{(8.00\text{ in.})^3 + (5.00\text{ in.})^3}\right]$$
$$= 802\text{ in.}^6$$

Due to symmetry, both the centroid and the shear center lie on the y-axis. Therefore, $x_o = 0$. The distance from the center of the outside flange to the shear center is:

$$e = h_o\left(\frac{b_{fi}^3}{b_{fi}^3 + b_{fo}^3}\right)$$
$$= (11.3\text{ in.})\left[\frac{(8.00\text{ in.})^3}{(8.00\text{ in.})^3 + (5.00\text{ in.})^3}\right]$$
$$= 9.08\text{ in.}$$

Add one-half the flange thickness to determine the shear center location measured from the bottom of the outside flange.

$$\begin{aligned}e+\frac{t_f}{2} &= 9.08\text{ in.}+\frac{\tfrac{3}{4}\text{ in.}}{2}\\ &= 9.46\text{ in.}\end{aligned}$$

$$\begin{aligned}y_o &= \left(e+\frac{t_f}{2}\right)-\bar{y}\\ &= 9.46\text{ in.}-6.91\text{ in.}\\ &= 2.55\text{ in.}\end{aligned}$$

$$\begin{aligned}\bar{r}_o^{\,2} &= x_o^2+y_o^2+\frac{I_x+I_y}{A_g} \qquad (\textit{Spec}.\text{ Eq. E4-9})\\ &= (0)^2+(2.55\text{ in.})^2+\frac{334\text{ in.}^4+39.9\text{ in.}^4}{13.7\text{ in.}^2}\\ &= 33.8\text{ in.}^2\end{aligned}$$

$$\begin{aligned}H &= 1-\frac{x_o^2+y_o^2}{\bar{r}_o^{\,2}} \qquad (\textit{Spec}.\text{ Eq. E4-8})\\ &= 1-\frac{(0)^2+(2.55\text{ in.})^2}{33.8\text{ in.}^2}\\ &= 0.808\end{aligned}$$

The effective slenderness ratio about the *y*-axis is:

$$\begin{aligned}\frac{L_{cy}}{r_y} &= \frac{(14.0\text{ ft})(12\text{ in./ft})}{1.71\text{ in.}}\\ &= 98.2\end{aligned}$$

$$\begin{aligned}F_{ey} &= \frac{\pi^2 E}{\left(\dfrac{L_{cy}}{r_y}\right)^2} \qquad (\textit{Spec}.\text{ Eq. E4-6})\\ &= \frac{\pi^2(29{,}000\text{ ksi})}{(98.2)^2}\\ &= 29.7\text{ ksi}\end{aligned}$$

$$\begin{aligned}F_{ez} &= \left(\frac{\pi^2 EC_w}{L_{cz}^{\,2}}+GJ\right)\frac{1}{A_g\bar{r}_o^{\,2}} \qquad (\textit{Spec}.\text{ Eq. E4-7})\\ &= \left\{\frac{\pi^2(29{,}000\text{ ksi})(802\text{ in.}^6)}{\left[(14.0\text{ ft})(12\text{ in./ft})\right]^2}+(11{,}200\text{ ksi})(2.01\text{ in.}^4)\right\}\left[\frac{1}{(13.7\text{ in.}^2)(33.8\text{ in.}^2)}\right]\\ &= 66.2\text{ ksi}\end{aligned}$$

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{\left(F_{ey} + F_{ez}\right)^2}}\right] \qquad (\textit{Spec}.\text{ Eq. E4-3})$$

$$= \left[\frac{29.7\text{ ksi} + 66.2\text{ ksi}}{2(0.808)}\right]\left[1 - \sqrt{1 - \frac{4(29.7\text{ ksi})(66.2\text{ ksi})(0.808)}{(29.7\text{ ksi} + 66.2\text{ ksi})^2}}\right]$$

$$= 26.4\text{ ksi} \quad \textbf{controls}$$

Torsional and flexural-torsional buckling governs.

$$\frac{F_y}{F_e} = \frac{50\text{ ksi}}{26.4\text{ ksi}}$$

$$= 1.89$$

Because $\dfrac{F_y}{F_e} < 2.25$:

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y \qquad (\textit{Spec}.\text{ Eq. E3-2})$$

$$= \left(0.658^{1.89}\right)(50\text{ ksi})$$

$$= 22.7\text{ ksi}$$

Available Compressive Strength

$$P_n = F_{cr}A_g \qquad (\textit{Spec}.\text{ Eq. E3-1})$$

$$= (22.7\text{ ksi})\left(13.7\text{ in.}^2\right)$$

$$= 311\text{ kips}$$

From AISC *Specification* Section E1, the available compressive strength is:

LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(311\text{ kips})$ $= 280\text{ kips}$	$\Omega_c = 1.67$ $\dfrac{P_n}{\Omega_c} = \dfrac{311\text{ kips}}{1.67}$ $= 186\text{ kips}$

EXAMPLE E.13 DOUBLE-WT COMPRESSION MEMBER

Given:

Determine the available compressive strength for an ASTM A992 double-WT9×20 compression member, as shown in Figure E.13-1. Assume that ½-in.-thick connectors are welded in position at the ends and at equal intervals, "*a*", along the length. Use the minimum number of intermediate connectors needed to force the two WT-shapes to act as a single built-up compression member.

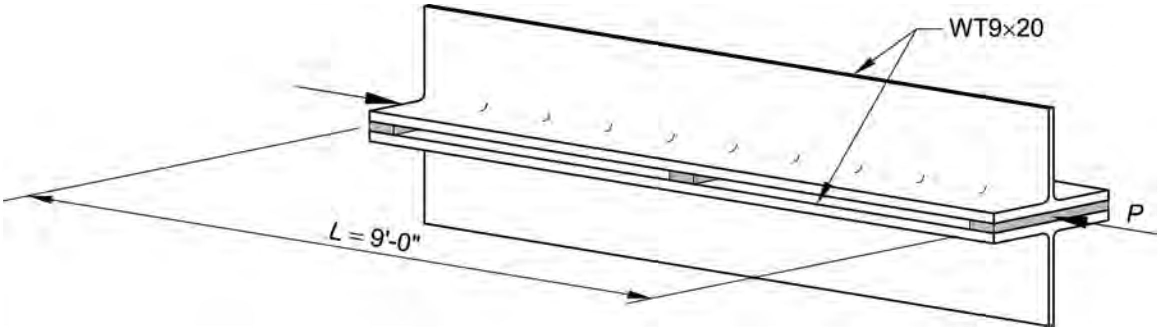


Fig. E.13-1. Double-WT compression member in Example E.13.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Tee
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 1-8 the geometric properties for a single WT9×20 are as follows:

A = 5.88 in.^{2}
d = 8.95 in.
t_w = 0.315 in.
d/t_w = 28.4
I_x = 44.8 in.^{4}
I_y = 9.55 in.^{4}
r_x = 2.76 in.
r_y = 1.27 in.
$\bar{y}$ = 2.29 in.
J = 0.404 in.^{4}
C_w = 0.788 in.^{6}

From mechanics of materials, the combined section properties for two WT9×20's, flange-to-flange, spaced ½-in. apart, are as follows:

$$\begin{aligned} A &= \Sigma A_{single\ tee} \\ &= 2\left(5.88\text{ in.}^2\right) \\ &= 11.8\text{ in.}^2 \end{aligned}$$

$$I_x = \Sigma\left(I_x + A\bar{y}^2\right)$$
$$= 2\left[44.8 \text{ in.}^4 + \left(5.88 \text{ in.}^2\right)\left(2.29 \text{ in.} + \tfrac{1}{4} \text{ in.}\right)^2\right]$$
$$= 165 \text{ in.}^4$$

$$r_x = \sqrt{\frac{I_x}{A}}$$
$$= \sqrt{\frac{165 \text{ in.}^4}{11.8 \text{ in.}^2}}$$
$$= 3.74 \text{ in.}$$

$$I_y = \Sigma I_{y \; single \; tee}$$
$$= 2\left(9.55 \text{ in.}^4\right)$$
$$= 19.1 \text{ in.}^4$$

$$r_y = \sqrt{\frac{I_y}{A}}$$
$$= \sqrt{\frac{19.1 \text{ in.}^4}{11.8 \text{ in.}^2}}$$
$$= 1.27 \text{ in.}$$

$$J = \Sigma J_{single \; tee}$$
$$= 2\left(0.404 \text{ in.}^4\right)$$
$$= 0.808 \text{ in.}^4$$

For the double-WT (cruciform) shape shown in Figure E.13-2 it is reasonable to take $C_w = 0$ and ignore any warping contribution to column strength.

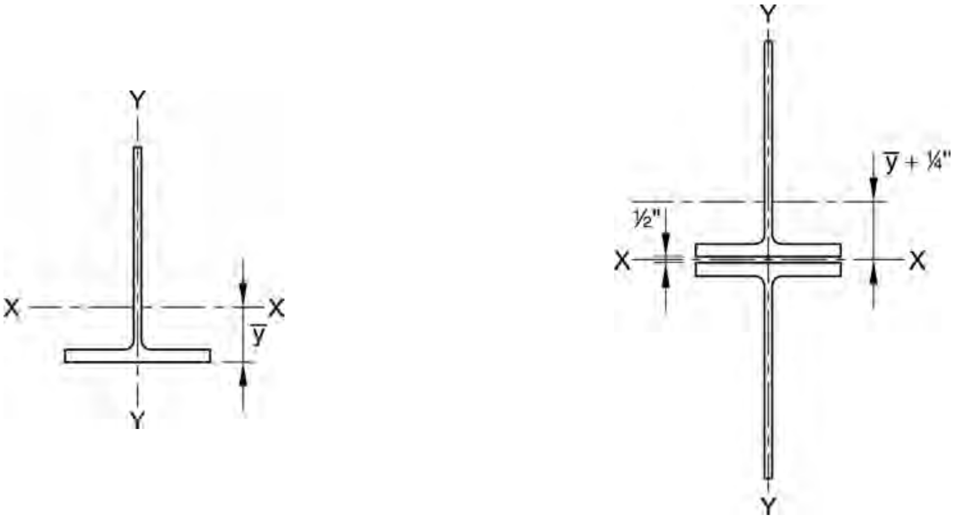


Fig. E.13-2. Double-WT shape cross section.

The y-axis of the combined section is the same as the y-axis of the single section. When buckling occurs about the y-axis, there is no relative slip between the two WTs. For buckling about the x-axis of the combined section, the WTs will slip relative to each other unless restrained by welded or slip-critical end connections.

Intermediate Connectors Dimensional Requirements

Determine the minimum number of intermediate connectors required.

From AISC *Specification* Section E6.2, the maximum slenderness ratio of each tee should not exceed three-fourths times the maximum slenderness ratio of the double-WT built-up section. For a WT9×20, the minimum radius of gyration is:

$$\begin{aligned} r_i &= r_y \\ &= 1.27 \text{ in.} \end{aligned}$$

Use K = 1.0 for both the single tee and the double tee; therefore, $L_{cy} = K_y L_y = 1.0(9 \text{ ft}) = 9.00 \text{ ft}$:

$$\left(\frac{a}{r_i}\right)_{single\ tee} \le \frac{3}{4}\left(\frac{L_{cy}}{r_{min}}\right)_{double\ tee}$$

$$\begin{aligned} a &\le \frac{3(r_y)_{single\ tee}}{4(r_y)_{double\ tee}}(L_{cy})_{double\ tee} \\ &= \frac{3}{4}\left(\frac{1.27 \text{ in.}}{1.27 \text{ in.}}\right)\left[(9.00 \text{ ft})(12 \text{ in./ft})\right] \\ &= 81.0 \text{ in.} \end{aligned}$$

Thus, one intermediate connector at mid-length [a = (4.5 ft)(12 in./ft) = 54.0 in.] satisfies AISC *Specification* Section E6.2 as shown in Figure E.13-3.

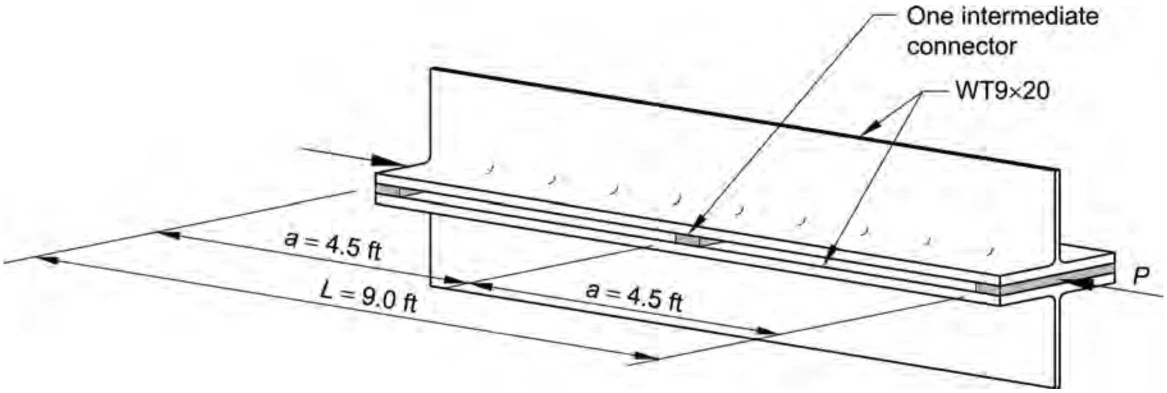


Figure E.13-3. Minimum connectors required for double-WT compression member.

Flexural Buckling and Torsional Buckling Strength

For the WT9×20, the stem is slender because d/t_w = 28.4 > 0.75 $\sqrt{29{,}000\text{ ksi}/50\text{ ksi}}$ = 18.1 (from AISC *Specification* Table B4.1a, Case 4). Therefore, the member is a slender element member and the provisions of Section E7 are followed. Determine the elastic buckling stress for flexural buckling about the *y*- and *x*-axes, and torsional buckling. Then, determine the effective area considering local buckling, the critical buckling stress, and the nominal strength.

Elastic Buckling Stress about the y-y Axis

$$\begin{aligned}\frac{L_{cy}}{r_y} &= \frac{(9.00\text{ ft})(12\text{ in./ft})}{1.27\text{ in.}}\\ &= 85.0\end{aligned}$$

$$\begin{aligned}F_{ey} &= \frac{\pi^2 E}{\left(\dfrac{L_{cy}}{r_y}\right)^2} \qquad \text{(}Spec.\text{ Eq. E4-6)}\\ &= \frac{\pi^2(29{,}000\text{ ksi})}{(85.0)^2}\\ &= 39.6\text{ ksi} \quad \textbf{controls}\end{aligned}$$

Elastic Buckling Stress about the x-x Axis

Flexural buckling about the *x*-axis is determined using the modified slenderness ratio to account for shear deformation of the intermediate connectors.

Note that the provisions of AISC *Specification* Section E6.1, which require that L_c/r be replaced with $(L_c/r)_m$, apply if “the buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes…”. Relative slip between the two sections occurs for buckling about the *x*-axis so the provisions of the section apply only to buckling about the *x*-axis.

The connectors are welded at the ends and the intermediate point. The modified slenderness is calculated using the spacing between intermediate connectors:

$$\begin{aligned}a &= (4.5\text{ ft})(12.0\text{ in./ft})\\ &= 54.0\text{ in.}\end{aligned}$$

$$\begin{aligned}r_i &= r_y\\ &= 1.27\text{ in.}\end{aligned}$$

$$\begin{aligned}\frac{a}{r_i} &= \frac{54.0\text{ in.}}{1.27\text{ in.}}\\ &= 42.5\end{aligned}$$

Because $a/r_i > 40$, use AISC *Specification* Equation E6-2b.

$$\left(\frac{L_c}{r}\right)_m = \sqrt{\left(\frac{L_c}{r}\right)_o^2 + \left(\frac{K_i a}{r_i}\right)^2} \qquad \text{(}Spec.\text{ Eq. E6-2b)}$$

where

$$\left(\frac{L_c}{r}\right)_o = \frac{L_{cx}}{r_x}$$
$$= \frac{(9.00\text{ ft})(12\text{ in./ft})}{3.74\text{ in.}}$$
$$= 28.9$$

$$\frac{K_i a}{r_i} = \frac{0.86(4.50\text{ ft})(12\text{ in./ft})}{1.27\text{ in.}}$$
$$= 36.6$$

Thus,

$$\left(\frac{L_c}{r}\right)_m = \sqrt{(28.9)^2 + (36.6)^2}$$
$$= 46.6$$

$$F_{ex} = \frac{\pi^2 E}{\left(\frac{L_{cx}}{r_x}\right)^2} \qquad \text{(\textit{Spec}. Eq. E4-5)}$$
$$= \frac{\pi^2(29{,}000\text{ ksi})}{(46.6)^2}$$
$$= 132\text{ ksi}$$

Torsional Buckling Elastic Stress

$$F_e = \left(\frac{\pi^2 E C_w}{L_{cz}^{\ 2}} + GJ\right)\frac{1}{I_x + I_y} \qquad \text{(\textit{Spec}. Eq. E4-2)}$$

The cruciform section made up of two back-to-back WT's has virtually no warping resistance, thus the warping contribution is ignored and *Specification* Equation E4-2 becomes:

$$F_e = \frac{GJ}{I_x + I_y}$$
$$= \frac{(11{,}200\text{ ksi})(0.808\text{ in.}^4)}{165\text{ in.}^4 + 19.1\text{ in.}^4}$$
$$= 49.2\text{ ksi}$$

Critical Stress

Use the smallest elastic buckling stress, F_e, from the limit states considered above to determine F_{cr} by AISC *Specification* Equation E3-2 or Equation E3-3, as follows:

$$F_e = 39.6\text{ ksi}$$

$$\begin{aligned}\frac{F_y}{F_e} &= \frac{50 \text{ ksi}}{39.6 \text{ ksi}}\\ &= 1.26\end{aligned}$$

Because $\dfrac{F_y}{F_e} < 2.25$,

$$\begin{aligned}F_{cr} &= \left(0.658^{\frac{F_y}{F_e}}\right)F_y \qquad \text{(\textit{Spec.} Eq. E3-2)}\\ &= \left(0.658^{1.26}\right)(50 \text{ ksi})\\ &= 29.5 \text{ ksi}\end{aligned}$$

Effective Area

Since the stem was previously shown to be slender, calculate the limits of AISC *Specification* Section E7.1 to determine if the stem is fully effective or if there is a reduction in effective area due to local buckling of the stem.

$$\lambda = 28.4$$

$$\begin{aligned}\lambda_r &= 0.75\sqrt{\frac{E}{F_y}}\\ &= 0.75\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}}\\ &= 18.1\end{aligned}$$

$$\begin{aligned}\lambda_r\sqrt{\frac{F_y}{F_{cr}}} &= 18.1\sqrt{\frac{50 \text{ ksi}}{29.5 \text{ ksi}}}\\ &= 23.6\end{aligned}$$

Because $\lambda > \lambda_r\sqrt{F_y/F_{cr}}$, the stem will not be fully effective and there will be a reduction in effective area due to local buckling of the stem. The effective width imperfection adjustment factors can be determined from AISC *Specification* Table E7.1, Case (c), as follows.

$$\begin{aligned}c_1 &= 0.22\\ c_2 &= 1.49\end{aligned}$$

Determine the elastic local buckling stress from AISC *Specification* Section E7.1.

$$\begin{aligned}F_{el} &= \left(c_2\frac{\lambda_r}{\lambda}\right)^2 F_y \qquad \text{(\textit{Spec.} Eq. E7-5)}\\ &= \left[1.49\left(\frac{18.1}{28.4}\right)\right]^2(50 \text{ ksi})\\ &= 45.1 \text{ ksi}\end{aligned}$$

Determine the effective width of the tee stem and the resulting effective area, where $b = d = 8.95$ in.

$$b_e = b\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} \qquad \text{(Spec. Eq. E7-3)}$$

$$= (8.95\text{ in.})\left(1 - 0.22\sqrt{\frac{45.1\text{ ksi}}{29.5\text{ ksi}}}\right)\sqrt{\frac{45.1\text{ ksi}}{29.5\text{ ksi}}}$$

$$= 8.06\text{ in.}$$

$$A_e = \sum A - \sum\left[t_w\left(b - b_e\right)\right]$$

$$= (2)\left(5.88\text{ in.}^2\right) - (2)(0.315\text{ in.})(8.95\text{ in.} - 8.06\text{ in.})$$

$$= 11.2\text{ in.}^2$$

Available Compressive Strength

$$P_n = F_{cr}A_e \qquad \text{(Spec. Eq. E7-1)}$$

$$= (29.5\text{ ksi})\left(11.2\text{ in.}^2\right)$$

$$= 330\text{ kips}$$

From AISC *Specification* Section E1, the available compressive strength is:

LRFD	ASD
$\phi_c = 0.90$	$\Omega_c = 1.67$
$\phi_c P_n = 0.90(330\text{ kips})$ $= 297\text{ kips}$	$\dfrac{P_n}{\Omega_c} = \dfrac{330\text{ kips}}{1.67}$ $= 198\text{ kips}$

EXAMPLE E.14 ECCENTRICALLY LOADED SINGLE-ANGLE COMPRESSION MEMBER (LONG LEG ATTACHED)

Given:

Determine the available strength of an eccentrically loaded ASTM A36 L8×4×½ single angle compression member, as shown in Figure E.14-1, with an effective length of 5 ft. The long leg of the angle is the attached leg, and the eccentric load is applied at 0.75t as shown. Use the provisions of the AISC *Specification* and compare the results to the available strength found in AISC *Manual* Table 4-12.

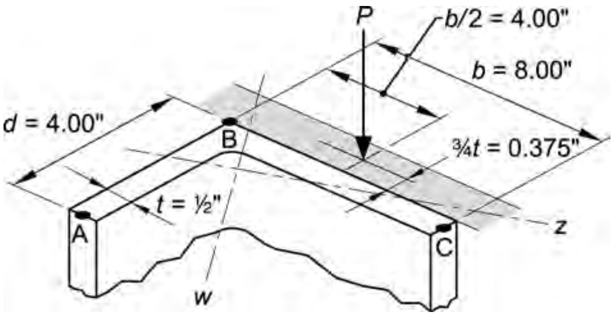


Fig. E.14-1. Eccentrically loaded single-angle compression member in Example E.14.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-7:

L8×4×½
$\bar{x}$ = 0.854 in.
$\bar{y}$ = 2.84 in.
A = 5.80 in.^{2}
I_x = 38.6 in.^{4}
I_y = 6.75 in.^{4}
I_z = 4.32 in.^{4}
r_z = 0.863 in.
tan α = 0.266

From AISC Shapes Database V15.0:

I_w = 41.0 in.^{4}
S_{wA} = 12.4 in.^{3}
S_{wB} = 16.3 in.^{3}
S_{wC} = 7.98 in.^{3}
S_{zA} = 1.82 in.^{3}
S_{zB} = 2.77 in.^{3}
S_{wC} = 5.81 in.^{3}

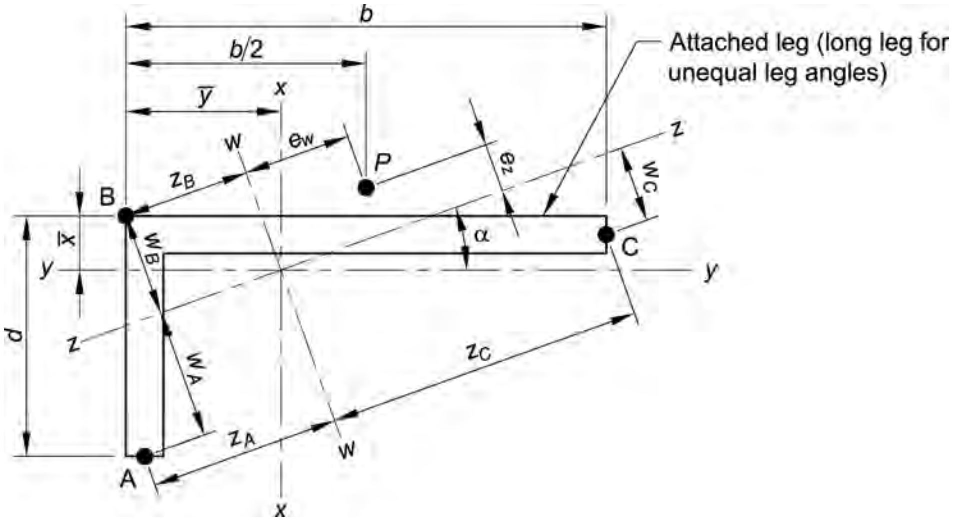


Fig. E.14-2. Geometry about principal axes.

The load is applied at the location shown in Figure E.14-2. Determine the eccentricities about the major (w-w axis) and minor (z-z axis) principal axes for the load, P. From AISC *Manual* Table 1-7, the angle of the principal axes is found to be $\alpha = \tan^{-1}(0.266) = 14.9°$.

Using the geometry shown in Figures E.14-2 and E.14-3:

$$
\begin{aligned}
e_w &= \left[\left(\bar{x}+0.75t\right)-\left(0.5b-\bar{y}\right)\tan\alpha\right]\sin\alpha+\left(\frac{0.5b-\bar{y}}{\cos\alpha}\right)\\
&=\left\{\left[0.854\text{ in.}+0.75\left(\tfrac{1}{2}\text{ in.}\right)\right]-\left[0.5\left(8.00\text{ in.}\right)-2.84\text{ in.}\right]\left(0.266\right)\right\}\left(\sin 14.9°\right)+\left[\frac{0.5\left(8.00\text{ in.}\right)-2.84\text{ in.}}{\left(\cos 14.9°\right)}\right]\\
&=1.44\text{ in.}
\end{aligned}
$$

$$
\begin{aligned}
e_z &= \left(\bar{x}+0.75t\right)\cos\alpha-\left(0.5b-\bar{y}\right)\sin\alpha\\
&=\left[0.854\text{ in.}+0.75\left(\tfrac{1}{2}\text{ in.}\right)\right]\left(\cos 14.9°\right)-\left[0.5\left(8.00\text{ in.}\right)-2.84\text{ in.}\right]\left(\sin 14.9°\right)\\
&=0.889\text{ in.}
\end{aligned}
$$

Because of these eccentricities, the moment resultant has components about both principal axes; therefore, the combined stress provisions of AISC *Specification* Section H2 must be followed.

$$\left|\frac{f_{ra}}{F_{ca}}+\frac{f_{rbw}}{F_{cbw}}+\frac{f_{rbz}}{F_{cbz}}\right|\le 1.0 \qquad \text{(}\textit{Spec}\text{. Eq. H2-1)}$$

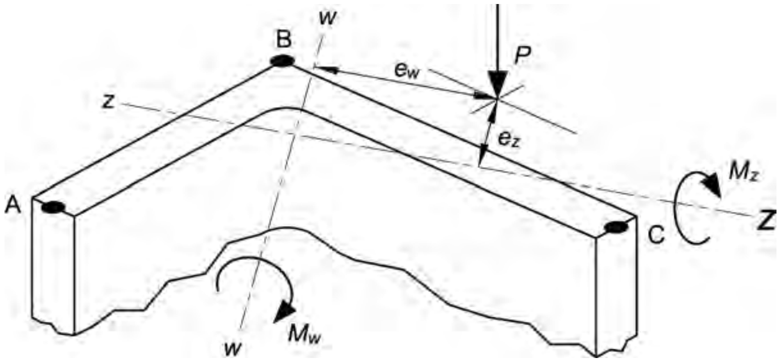


Fig. E.14-3. Applied moments and eccentric axial load.

Due to the load and the given eccentricities, moments about the *w*-*w* and *z*-*z* axes will have different effects on points A, B and C. The axial force will produce a compressive stress and the moments, where positive moments are in the direction shown in Figure E.14-3, will produce stresses with a sign indicated by the sense given in the following. In this example, compressive stresses will be taken as positive and tensile stresses will be taken as negative.

Point	Caused by M_w	Caused by M_z
A	tension	tension
B	tension	compression
C	compression	tension

Available Compressive Strength

Check the slenderness of the longest leg for uniform compression.

$$\begin{aligned}\lambda &= \frac{b}{t}\\ &= \frac{8.00\text{ in.}}{\tfrac{1}{2}\text{ in.}}\\ &= 16.0\end{aligned}$$

Check the slenderness of the shorter leg for uniform compression.

$$\begin{aligned}\lambda &= \frac{d}{t}\\ &= \frac{4.00\text{ in.}}{\tfrac{1}{2}\text{ in.}}\\ &= 8.00\end{aligned}$$

From *AISC Specification* Table B4.1a, Case 3, the limiting width-to-thickness ratio is:

$$\begin{aligned}\lambda_r &= 0.45\sqrt{\frac{E}{F_y}}\\ &= 0.45\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}}\\ &= 12.8\end{aligned}$$

Because $b/t = 16.0 > 12.8$, the longer leg is classified as a slender element for compression. Because $d/t = 8.00 < 12.8$, the shorter leg is classified as a nonslender element for compression.

Determine if torsional and flexural-torsional buckling is applicable, using the provisions of AISC *Specification* Section E4.

$$\lambda = 16.0$$

$$\begin{aligned} 0.71\sqrt{\frac{E}{F_y}} &= 0.71\sqrt{\frac{29{,}000 \text{ ksi}}{36 \text{ ksi}}} \\ &= 20.2 \end{aligned}$$

Because $\lambda < 0.71\sqrt{E / F_y}$, torsional and flexural-torsional buckling is not applicable.

Determine the critical stress, F_{cr}, with L_c = (5.00 ft)(12 in./ft) = 60.0 in. for buckling about the z-z axis.

$$\begin{aligned} \frac{L_{cz}}{r_z} &= \frac{60.0 \text{ in.}}{0.863 \text{ in.}} \\ &= 69.5 \end{aligned}$$

$$\begin{aligned} F_e &= \frac{\pi^2 E}{\left(\dfrac{L_{cz}}{r_z}\right)^2} \qquad \text{(\textit{Spec}. Eq. E3-4)} \\ &= \frac{\pi^2 (29{,}000 \text{ ksi})}{(69.5)^2} \\ &= 59.3 \text{ ksi} \end{aligned}$$

$$\begin{aligned} \frac{F_y}{F_e} &= \frac{36 \text{ ksi}}{59.3 \text{ ksi}} \\ &= 0.607 \end{aligned}$$

Because $\dfrac{F_y}{F_e} < 2.25$:

$$\begin{aligned} F_{cr} &= \left(0.658^{\frac{F_y}{F_e}}\right) F_y \qquad \text{(\textit{Spec}. Eq. E3-2)} \\ &= \left(0.658^{0.607}\right)(36 \text{ ksi}) \\ &= 27.9 \text{ ksi} \end{aligned}$$

Because the longer leg was found to be slender, the limits of AISC *Specification* Section E7.1 must be evaluated to determine if the leg is fully effective for compression or if a reduction in effective area must be taken to account for local buckling in the longer leg.

$$\lambda = 16.0$$

$$\lambda_r \sqrt{\frac{F_y}{F_{cr}}} = 12.8\sqrt{\frac{36 \text{ ksi}}{27.9 \text{ ksi}}}$$
$$= 14.5$$

Because $\lambda > 14.5,$ there will be a reduction in effective area due to local buckling in the longer leg. Determine the effective width imperfection adjustment factors per AISC *Specification* Table E7.1 as follows.

$$c_1 = 0.22$$
$$c_2 = 1.49$$

Determine the elastic local buckling stress from AISC *Specification* Section E7.1.

$$F_{el} = \left(c_2 \frac{\lambda_r}{\lambda}\right)^2 F_y \qquad \text{(}\textit{Spec}\text{. Eq. E7-5)}$$
$$= \left[1.49\left(\frac{12.8}{16.0}\right)\right]^2 (36 \text{ ksi})$$
$$= 51.2 \text{ ksi}$$

Determine the effective width of the angle leg and the resulting effective area.

$$b_e = b\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} \qquad \text{(}\textit{Spec}\text{. Eq. E7-3)}$$
$$= (8.00 \text{ in.})\left(1 - 0.22\sqrt{\frac{51.2 \text{ ksi}}{27.9 \text{ ksi}}}\right)\sqrt{\frac{51.2 \text{ ksi}}{27.9 \text{ ksi}}}$$
$$= 7.61 \text{ in.}$$

$$A_e = A_g - t\sum(b - b_e)$$
$$= 5.80 \text{ in.}^2 - (\tfrac{1}{2} \text{ in.})(8.00 \text{ in.} - 7.61 \text{ in.})$$
$$= 5.61 \text{ in.}^2$$

Available Compressive Strength

$$P_n = F_{cr} A_e \qquad \text{(}\textit{Spec}\text{. Eq. E7-1)}$$
$$= (27.9 \text{ ksi})(5.61 \text{ in.}^2)$$
$$= 157 \text{ kips}$$

From AISC *Specification* Section E1, the available compressive strength is:

LRFD	ASD
$\phi_c = 0.90$	$\Omega_c = 1.67$
$\phi_c P_n = 0.90(157 \text{ kips})$ $= 141 \text{ kips}$	$\dfrac{P_n}{\Omega_c} = \dfrac{157 \text{ kips}}{1.67}$ $= 94.0 \text{ kips}$

Determine the available flexural strengths, M_{cbw} and M_{cbz}, and the available flexural stresses at each point on the cross section.

Yielding

Consider the limit state of yielding for bending about the *w*-*w* and *z*-*z* axes at points A, B and C, according to AISC *Specification* Section F10.1.

w-*w* axis:

$$\begin{aligned} M_{ywA} &= F_y S_{wA} \\ &= (36\text{ ksi})(12.4\text{ in.}^3) \\ &= 446\text{ kip-in.} \end{aligned}$$

$$\begin{aligned} M_{nwA} &= 1.5M_{ywA} \\ &= 1.5(446\text{ kip-in.}) \\ &= 669\text{ kip-in.} \end{aligned}$$

(from *Spec.* Eq. F10-1)

$$\begin{aligned} M_{ywB} &= F_y S_{wB} \\ &= (36\text{ ksi})(16.3\text{ in.}^3) \\ &= 587\text{ kip-in.} \end{aligned}$$

$$\begin{aligned} M_{nwB} &= 1.5M_{ywB} \\ &= 1.5(587\text{ kip-in.}) \\ &= 881\text{ kip-in.} \end{aligned}$$

(from *Spec.* Eq. F10-1)

$$\begin{aligned} M_{ywC} &= F_y S_{wC} \\ &= (36\text{ ksi})(7.98\text{ in.}^3) \\ &= 287\text{ kip-in.} \end{aligned}$$

$$\begin{aligned} M_{nwC} &= 1.5M_{ywC} \\ &= 1.5(287\text{ kip-in.}) \\ &= 431\text{ kip-in.} \end{aligned}$$

(from *Spec.* Eq. F10-1)

z-*z* axis:

$$\begin{aligned} M_{yzA} &= F_y S_{zA} \\ &= (36\text{ ksi})(1.82\text{ in.}^3) \\ &= 65.5\text{ kip-in.} \end{aligned}$$

$$\begin{aligned} M_{nzA} &= 1.5M_{yzA} \\ &= 1.5(65.5\text{ kip-in.}) \\ &= 98.3\text{ kip-in.} \end{aligned}$$

(from *Spec.* Eq. F10-1)

$$\begin{aligned}M_{yzB} &= F_y S_{zB}\\ &= (36\text{ ksi})(2.77\text{ in.}^3)\\ &= 99.7\text{ kip-in.}\end{aligned}$$

$$\begin{aligned}M_{nzB} &= 1.5M_{yzB}\\ &= 1.5(99.7\text{ kip-in.})\\ &= 150\text{ kip-in.}\end{aligned}$$

(from *Spec*. Eq. F10-1)

$$\begin{aligned}M_{yzC} &= F_y S_{zC}\\ &= (36\text{ ksi})(5.81\text{ in.}^3)\\ &= 209\text{ kip-in.}\end{aligned}$$

$$\begin{aligned}M_{nzC} &= 1.5M_{yzC}\\ &= 1.5(209\text{ kip-in.})\\ &= 314\text{ kip-in.}\end{aligned}$$

(from *Spec*. Eq. F10-1)

Select the least M_n for each axis.

For the limit state of yielding about the *w*-*w* axis:

$$M_{nw} = 431\text{ kip-in. at point C}$$

For the limit state of yielding about the *z*-*z* axis:

$$M_{nz} = 98.3\text{ kip-in. at point A}$$

Lateral-Torsional Buckling

From AISC *Specification* Section F10.2, the limit state of lateral-torsional buckling of a single angle without continuous restraint along its length is a function of the elastic lateral-torsional buckling moment about the major principal axis. For bending about the major principal axis for a single angle:

$$M_{cr} = \frac{9EAr_z tC_b}{8L_b}\left[\sqrt{1+\left(4.4\frac{\beta_w r_z}{L_b t}\right)^2}+4.4\frac{\beta_w r_z}{L_b t}\right] \qquad (\textit{Spec}.\text{ Eq. F10-4})$$

From AISC *Specification* Section F1, for uniform moment along the member length, C_b = 1.0. From AISC *Specification* Commentary Table C-F10.1, an L8×4×½ has β_w = 5.48 in. From AISC *Specification* Commentary Figure C-F10.4b, with the tip of the long leg (point C) in compression for bending about the *w*-axis, β_w is taken as negative. Thus:

$$\begin{aligned}M_{cr} &= \frac{9(29{,}000\text{ ksi})(5.80\text{ in.}^2)(0.863\text{ in.})(\tfrac{1}{2}\text{ in.})(1.0)}{8(60.0\text{ in.})}\\ &\quad\times\left\{\sqrt{1+\left[4.4\frac{(-5.48\text{ in.})(0.863\text{ in.})}{(60.0\text{ in.})(\tfrac{1}{2}\text{ in.})}\right]^2}+4.4\frac{(-5.48\text{ in.})(0.863\text{ in.})}{(60.0\text{ in.})(\tfrac{1}{2}\text{ in.})}\right\}\\ &= 712\text{ kip-in.}\end{aligned}$$

$$\begin{aligned}\frac{M_{ywC}}{M_{cr}} &= \frac{287 \text{ kip-in.}}{712 \text{ kip-in.}}\\ &= 0.403\end{aligned}$$

Because $M_{ywC}/M_{cr} < 1.0$, determine M_n as follows:

$$M_{nwC} = \left(1.92 - 1.17\sqrt{\frac{M_{ywC}}{M_{cr}}}\right) M_{ywC} \leq 1.5 M_{ywC} \qquad \text{(from } \textit{Spec}\text{. Eq. F10-2)}$$

$$\begin{aligned} &= \left(1.92 - 1.17\sqrt{0.403}\right)(287 \text{ kip-in.}) < 1.5(287 \text{ kip-in.})\\ &= 338 \text{ kip-in.} < 431 \text{ kip-in.}\\ &= 338 \text{ kip-in.}\end{aligned}$$

Leg Local Buckling

From AISC *Specification* Section F10.3, the limit state of leg local buckling applies when the toe of the leg is in compression. As discussed previously and indicated in Table E.14-1, the only case in which a toe is in compression is point C for bending about the *w*-*w* axis. Thus, determine the slenderness of the long leg as a compression element subject to flexure. From AISC *Specification* Table B4.1b, Case 12:

$$\begin{aligned}\lambda_p &= 0.54\sqrt{\frac{E}{F_y}}\\ &= 0.54\sqrt{\frac{29{,}000 \text{ ksi}}{36 \text{ ksi}}}\\ &= 15.3\end{aligned}$$

$$\begin{aligned}\lambda_r &= 0.91\sqrt{\frac{E}{F_y}}\\ &= 0.91\sqrt{\frac{29{,}000 \text{ ksi}}{36 \text{ ksi}}}\\ &= 25.8\end{aligned}$$

$$\begin{aligned}\lambda &= \frac{b}{t}\\ &= \frac{8.0 \text{ in.}}{\tfrac{1}{2} \text{ in.}}\\ &= 16.0\end{aligned}$$

Because $\lambda_p < \lambda < \lambda_r$, the angle is noncompact for flexure for this loading. From AISC *Specification* Equation F10-6:

$$M_{nwC} = F_y S_{wC}\left(2.43 - 1.72\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}}\right) \qquad \text{(from } \textit{Spec}\text{. Eq. F10-6)}$$

$$\begin{aligned} &= (36 \text{ ksi})\left(7.98 \text{ in.}^3\right)\left[2.43 - 1.72(16.0)\sqrt{\frac{36 \text{ ksi}}{29{,}000 \text{ ksi}}}\right]\\ &= 420 \text{ kip-in.}\end{aligned}$$

Table E.14-1 provides a summary of nominal flexural strength at each point. T indicates the point is in tension and C indicates it is in compression.

Table E.14-1

Point	Yielding		Lateral-Torsional Buckling		Leg Local Buckling	
	M_{nw}, kip-in.	M_{nz}, kip-in.	M_{nw}, kip-in.	M_{nz}, kip-in.	M_{nw}, kip-in.	M_{nz}, kip-in.
A	669 T	98.3 T	–	–	–	–
B	881 T	150 C	–	–	–	–
C	431 C	314 T	338 C	–	420 C	–

Note: (–) indicates that the limit state is not applicable to this point.

Available Flexural Strength

Select the controlling nominal flexural strength for the *w-w* and *z-z* axes.

For the *w-w* axis:

$$M_{nw} = 338 \text{ kip-in.}$$

For the *z-z* axis:

$$M_{nz} = 98.3 \text{ kip-in.}$$

From AISC *Specification* Section F1, determine the available flexural strength for each axis, *w-w* and *z-z*, as follows:

LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
$\begin{aligned} M_{cbw} &= \phi_b M_{nw} \\ &= 0.90(338 \text{ kip-in.}) \\ &= 304 \text{ kip-in.} \end{aligned}$	$\begin{aligned} M_{cbw} &= \frac{M_{nw}}{\Omega_b} \\ &= \frac{338 \text{ kip-in.}}{1.67} \\ &= 202 \text{ kip-in.} \end{aligned}$
$\begin{aligned} M_{cbz} &= \phi_b M_{nz} \\ &= 0.90(98.3 \text{ kip-in.}) \\ &= 88.5 \text{ kip-in.} \end{aligned}$	$\begin{aligned} M_{cbz} &= \frac{M_{nz}}{\Omega_b} \\ &= \frac{98.3 \text{ kip-in.}}{1.67} \\ &= 58.9 \text{ kip-in.} \end{aligned}$

Required Flexural Strength

The load on the column is applied at eccentricities about the *w-w* and *z-z* axes resulting in the following moments:

$$\begin{aligned} M_w &= P_r e_w \\ &= P_r (1.44 \text{ in.}) \end{aligned}$$

and

$$
\begin{aligned}
M_z &= P_r e_z \\
&= P_r\left(0.889\text{ in.}\right)
\end{aligned}
$$

The combination of axial load and moment will produce second-order effects in the column which must be accounted for.

Using AISC *Specification* Appendix 8.2, an approximate second-order analysis can be performed. The required second-order flexural strengths will be $B_{1w}\,M_w$ and $B_{1z}\,M_z$, respectively, where

$$B_1 = \frac{C_m}{1-\dfrac{\alpha P_r}{P_{e1}}} \geq 1.0 \qquad \text{(}\textit{Spec}\text{. Eq. A-8-3)}$$

and

$\alpha = 1.0$ (LRFD)

$\alpha = 1.6$ (ASD)

C_m = 1.0 for a column with uniform moment along its length

For each axis, parameters P_{e1w} and P_{e1z}, as used in the moment magnification terms, B_{1w} and B_{1z}, are:

$$
\begin{aligned}
P_{e1w} &= \frac{\pi^2 EI_w}{\left(L_{c1}\right)^2} \qquad \text{(from }\textit{Spec}\text{. Eq. A-8-5)} \\
&= \frac{\pi^2\left(29{,}000\text{ ksi}\right)\left(41.0\text{ in.}^4\right)}{\left(60.0\text{ in.}\right)^2} \\
&= 3{,}260\text{ kips}
\end{aligned}
$$

$$
\begin{aligned}
P_{e1z} &= \frac{\pi^2 EI_z}{\left(L_{c1}\right)^2} \qquad \text{(from }\textit{Spec}\text{. Eq. A-8-5)} \\
&= \frac{\pi^2(29{,}000\text{ ksi})(4.32\text{ in.}^4)}{\left(60.0\text{ in.}\right)^2} \\
&= 343\text{ kips}
\end{aligned}
$$

and

$$
\begin{aligned}
B_{1w} &= \frac{C_m}{1-\dfrac{\alpha P_r}{P_{e1w}}} \qquad \text{(}\textit{Spec}\text{. Eq. A-8-3)} \\
&= \frac{1.0}{1-\dfrac{\alpha P_r}{3{,}260\text{ kips}}}
\end{aligned}
$$

$$
\begin{aligned}
B_{1z} &= \frac{C_m}{1-\dfrac{\alpha P_r}{P_{e1z}}} \qquad \text{(}\textit{Spec}\text{. Eq. A-8-3)} \\
&= \frac{1.0}{1-\dfrac{\alpha P_r}{343\text{ kips}}}
\end{aligned}
$$

Thus, the required second-order flexural strengths are:

$$M_{rw} = P_r\left(1.44\text{ in.}\right)\left(\frac{1.0}{1-\dfrac{\alpha P_r}{3{,}260\text{ kips}}}\right)$$

$$M_{rz} = P_r\left(0.889\text{ in.}\right)\left(\frac{1.0}{1-\dfrac{\alpha P_r}{343\text{ kips}}}\right)$$

Interaction of Axial and Flexural Strength

Evaluate the interaction of axial and flexural stresses according to the provisions of AISC *Specification* Section H2.

The interaction equation is given as:

$$\left|\frac{f_{ra}}{F_{ca}}+\frac{f_{rbw}}{F_{cbw}}+\frac{f_{rbz}}{F_{cbz}}\right| \le 1.0 \qquad \text{(}\textit{Spec}\text{. Eq. H2-1)}$$

where the stresses are to be considered at each point on the cross section with the appropriate sign representing the sense of the stress. Because the required stress and available stress at any point are both functions of the same section property, *A* or *S*, it is possible to convert Equation H2-1 from a stress based equation to a force based equation where the section properties will cancel.

Substituting the available strengths and the expressions for the required second-order flexural strengths into AISC *Specification* Equation H2-1 yields:

LRFD	ASD
$\left\lvert \dfrac{P_u}{141\text{ kips}}+\dfrac{P_u\left(1.44\text{ in.}\right)}{304\text{ kip-in.}}\left(\dfrac{1.0}{1-\dfrac{1.0P_u}{3{,}260\text{ kips}}}\right) + \left[\dfrac{P_u\left(0.889\text{ in.}\right)}{88.5\text{ kip-in.}}\right]\left(\dfrac{1}{1-\dfrac{1.0P_u}{343\text{ kips}}}\right)\right\rvert \le 1.0$	$\left\lvert \dfrac{P_a}{94.0\text{ kips}}+\dfrac{P_a\left(1.44\text{ in.}\right)}{202\text{ kip-in.}}\left(\dfrac{1.0}{1-\dfrac{1.6P_a}{3{,}260\text{ kips}}}\right) + \left[\dfrac{P_a\left(0.889\text{ in.}\right)}{58.9\text{ kip-in.}}\right]\left(\dfrac{1}{1-\dfrac{1.6P_a}{343\text{ kips}}}\right)\right\rvert \le 1.0$

These interaction equations must now be applied at each critical point on the section, points A, B and C using the appropriate sign for the sense of the resulting stress, with compression taken as positive.

For point A, the *w* term is negative and the *z* term is negative. Thus:

LRFD	ASD
$\left\lvert \dfrac{P_u}{141\text{ kips}}-\dfrac{P_u\left(1.44\text{ in.}\right)}{304\text{ kip-in.}}\left(\dfrac{1.0}{1-\dfrac{1.0P_u}{3{,}260\text{ kips}}}\right) - \left[\dfrac{P_u\left(0.889\text{ in.}\right)}{88.5\text{ kip-in.}}\right]\left(\dfrac{1}{1-\dfrac{1.0P_u}{343\text{ kips}}}\right)\right\rvert \le 1.0$ By iteration, $P_u = 88.4$ kips.	$\left\lvert \dfrac{P_a}{94.0\text{ kips}}-\dfrac{P_a\left(1.44\text{ in.}\right)}{202\text{ kip-in.}}\left(\dfrac{1.0}{1-\dfrac{1.6P_a}{3{,}260\text{ kips}}}\right) - \left[\dfrac{P_a\left(0.889\text{ in.}\right)}{58.9\text{ kip-in.}}\right]\left(\dfrac{1}{1-\dfrac{1.6P_a}{343\text{ kips}}}\right)\right\rvert \le 1.0$ By iteration, $P_a = 57.7$ kips.

For point B, the *w* term is negative and the *z* term is positive. Thus:

LRFD	ASD
$\left\lvert \frac{P_u}{141\text{ kips}} - \frac{P_u\left(1.44\text{ in.}\right)}{304\text{ kip-in.}}\left(\frac{1.0}{1-\frac{1.0P_u}{3{,}260\text{ kips}}}\right) + \left[\frac{P_u\left(0.889\text{ in.}\right)}{88.5\text{ kip-in.}}\right]\left(\frac{1}{1-\frac{1.0P_u}{343\text{ kips}}}\right) \right\rvert \le 1.0$ By iteration, P_u = 67.7 kips.	$\left\lvert \frac{P_a}{94.0\text{ kips}} - \frac{P_a\left(1.44\text{ in.}\right)}{202\text{ kip-in.}}\left(\frac{1.0}{1-\frac{1.6P_a}{3{,}260\text{ kips}}}\right) + \left[\frac{P_a\left(0.889\text{ in.}\right)}{58.9\text{ kip-in.}}\right]\left(\frac{1}{1-\frac{1.6P_a}{343\text{ kips}}}\right) \right\rvert \le 1.0$ By iteration, P_a = 44.6 kips.

For point C, the *w* term is positive and the *z* term is negative. Thus:

LRFD	ASD
$\left\lvert \frac{P_u}{141\text{ kips}} + \frac{P_u\left(1.44\text{ in.}\right)}{304\text{ kip-in.}}\left(\frac{1.0}{1-\frac{1.0P_u}{3{,}260\text{ kips}}}\right) - \left[\frac{P_u\left(0.889\text{ in.}\right)}{88.5\text{ kip-in.}}\right]\left(\frac{1}{1-\frac{1.0P_u}{343\text{ kips}}}\right) \right\rvert \le 1.0$ By iteration, P_u = 156 kips.	$\left\lvert \frac{P_a}{94.0\text{ kips}} + \frac{P_a\left(1.44\text{ in.}\right)}{202\text{ kip-in.}}\left(\frac{1.0}{1-\frac{1.6P_a}{3{,}260\text{ kips}}}\right) - \left[\frac{P_a\left(0.889\text{ in.}\right)}{58.9\text{ kip-in.}}\right]\left(\frac{1}{1-\frac{1.6P_a}{343\text{ kips}}}\right) \right\rvert \le 1.0$ By iteration, P_a = 99.5 kips.

Governing Available Strength

LRFD	ASD
From the above iterations, P_u = 67.7 kips From AISC *Manual* Table 4-12, $\phi P_n = 67.7\text{ kips}$	From the above iterations, P_a = 44.6 kips From AISC *Manual* Table 4-12, $\frac{P_n}{\Omega} = 44.6\text{ kips}$

Thus, the calculations demonstrate how the values for this member in AISC *Manual* Table 4-12 can be confirmed.

Chapter F
Design of Members for Flexure

INTRODUCTION

This *Specification* chapter contains provisions for calculating the flexural strength of members subject to simple bending about one principal axis. Included are specific provisions for I-shaped members, channels, HSS, box sections, tees, double angles, single angles, rectangular bars, rounds and unsymmetrical shapes. Also included is a section with proportioning requirements for beams and girders.

There are selection tables in the AISC *Manual* for standard beams in the commonly available yield strengths. The section property tables for most cross sections provide information that can be used to conveniently identify noncompact and slender element sections. LRFD and ASD information is presented side-by-side.

Most of the formulas from this chapter are illustrated by the following examples. The design and selection techniques illustrated in the examples for both LRFD and ASD will result in similar designs.

F1. GENERAL PROVISIONS

Selection and evaluation of all members is based on deflection requirements and strength, which is determined as the design flexural strength, $\phi_b M_n$, or the allowable flexural strength, M_n/Ω_b,

where

M_n = the lowest nominal flexural strength based on the limit states of yielding, lateral torsional-buckling, and local buckling, where applicable

ϕ_b = 0.90 (LRFD)

Ω_b = 1.67 (ASD)

This design approach is followed in all examples.

The term L_b is used throughout this chapter to describe the length between points which are either braced against lateral displacement of the compression flange or braced against twist of the cross section. Requirements for bracing systems and the required strength and stiffness at brace points are given in AISC *Specification* Appendix 6.

The use of C_b is illustrated in several of the following examples. AISC *Manual* Table 3-1 provides tabulated C_b values for some common situations.

F2. DOUBLY SYMMETRIC COMPACT I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MAJOR AXIS

AISC *Specification* Section F2 applies to the design of compact beams and channels. As indicated in the User Note in Section F2 of the AISC *Specification*, the vast majority of rolled I-shaped beams and channels fall into this category. The curve presented as a solid line in Figure F-1 is a generic plot of the nominal flexural strength, M_n, as a function of the unbraced length, L_b. The horizontal segment of the curve at the far left, between L_b = 0 ft and L_p, is the range where the strength is limited by flexural yielding. In this region, the nominal strength is taken as the full plastic moment strength of the section as given by AISC *Specification* Equation F2-1. In the range of the curve at the far right, starting at L_r, the strength is limited by elastic buckling. The strength in this region is given by AISC *Specification* Equation F2-3. Between these regions, within the linear region of the curve between $M_n = M_p$ at L_p on the left, and $M_n = 0.7M_y = 0.7F_yS_x$ at L_r on the right, the strength is limited by inelastic buckling. The strength in this region is provided in AISC *Specification* Equation F2-2.

The curve plotted as a heavy solid line represents the case where C_b = 1.0, while the heavy dashed line represents the case where C_b exceeds 1.0. The nominal strengths calculated in both AISC *Specification* Equations F2-2 and F2-3 are linearly proportional to C_b, but are limited to M_p as shown in the figure.

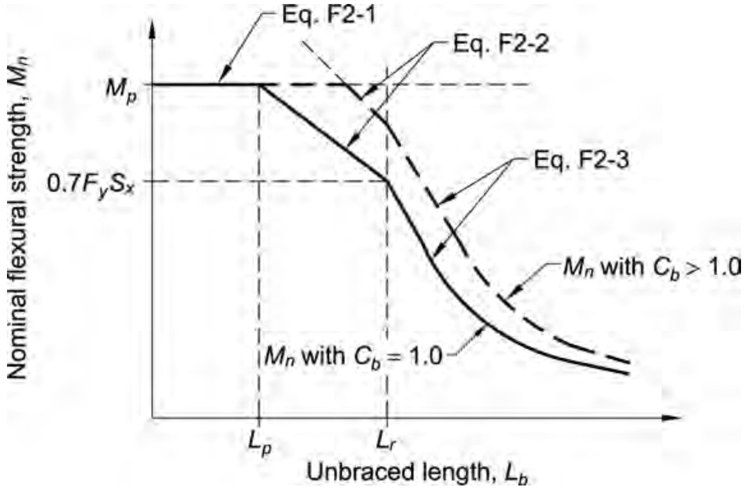


Fig. F-1. Nominal flexural strength versus unbraced length.

$$M_n = M_p = F_yZ_x \qquad (\textit{Spec}.\ \text{Eq. F2-1})$$

$$M_n = C_b\left[M_p - \left(M_p - 0.7F_yS_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \qquad (\textit{Spec}.\ \text{Eq. F2-2})$$

$$M_n = F_{cr}S_x \le M_p \qquad (\textit{Spec}.\ \text{Eq. F2-3})$$

where

$$F_{cr} = \frac{C_b\pi^2E}{\left(\dfrac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_xh_o}\left(\frac{L_b}{r_{ts}}\right)^2} \qquad (\textit{Spec}.\ \text{Eq. F2-4})$$

The provisions of this section are illustrated in Example F.1 (W-shape beam) and Example F.2 (channel).

Inelastic design provisions are given in AISC *Specification* Appendix 1. L_{pd}, the maximum unbraced length for prismatic member segments containing plastic hinges is less than L_p.

F3. DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH COMPACT WEBS AND NONCOMPACT OR SLENDER FLANGES BENT ABOUT THEIR MAJOR AXIS

The strength of shapes designed according to this section is limited by local buckling of the compression flange. Only a few standard wide-flange shapes have noncompact flanges. For these sections, the strength reduction for F_y = 50 ksi steel varies. The approximate percentages of M_p about the strong axis that can be developed by noncompact members when braced such that $L_b \le L_p$ are shown as follows:

W21×48 = 99%
W10×12 = 99%
W6×8.5 = 97%
W14×99 = 99%
W8×31 = 99%
W14×90 = 97%
W8×10 = 99%
W12×65 = 98%
W6×15 = 94%

The strength curve for the flange local buckling limit state, shown in Figure F-2, is similar in nature to that of the lateral-torsional buckling curve. The horizontal axis parameter is $\lambda = b_f/2t_f$. The flat portion of the curve to the left of λ_{pf} is the plastic yielding strength, M_p. The curved portion to the right of λ_{rf} is the strength limited by elastic buckling of the flange. The linear transition between these two regions is the strength limited by inelastic flange buckling.

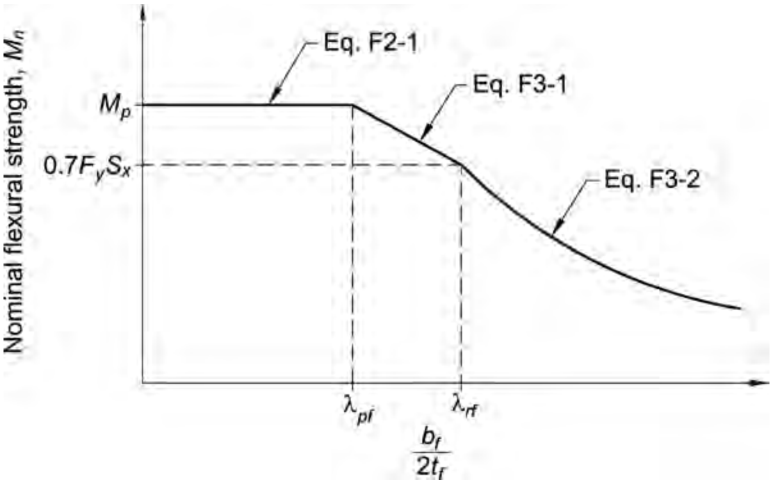


Fig. F-2. Flange local buckling strength.

$$M_n = M_p = F_yZ_x \quad \text{(\textit{Spec}. Eq. F2-1)}$$

$$M_n = M_p - (M_p - 0.7F_yS_x)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \quad \text{(\textit{Spec}. Eq. F3-1)}$$

$$M_n = \frac{0.9Ek_cS_x}{\lambda^2} \quad \text{(\textit{Spec}. Eq. F3-2)}$$

where

$k_c = \frac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.

The strength reductions due to flange local buckling of the few standard rolled shapes with noncompact flanges are incorporated into the design tables in Part 3 and Part 6 of the AISC *Manual*.

There are no standard I-shaped members with slender flanges. The noncompact flange provisions of this section are illustrated in Example F.3.

F4. OTHER I-SHAPED MEMBERS WITH COMPACT OR NONCOMPACT WEBS BENT ABOUT THEIR MAJOR AXIS

This section of the AISC *Specification* applies to doubly symmetric I-shaped members with noncompact webs and singly symmetric I-shaped members (those having different flanges) with compact or noncompact webs.

F5. DOUBLY SYMMETRIC AND SINGLY SYMMETRIC I-SHAPED MEMBERS WITH SLENDER WEBS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric and singly symmetric I-shaped members with slender webs, formerly designated as “plate girders”.

F6. I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MINOR AXIS

I-shaped members and channels bent about their minor axis are not subject to lateral-torsional buckling. Rolled or built-up shapes with noncompact or slender flanges, as determined by AISC *Specification* Table B4.1b, must be checked for strength based on the limit state of flange local buckling using Equations F6-2 or F6-3 as applicable.

The vast majority of W, M, C and MC shapes have compact flanges, and can therefore develop the full plastic moment, M_p, about the minor axis. The provisions of this section are illustrated in Example F.5.

F7. SQUARE AND RECTANGULAR HSS AND BOX SECTIONS

Square and rectangular HSS need to be checked for the limit states of yielding, and flange and web local buckling. Lateral-torsional buckling is also possible for rectangular HSS or box sections bent about the strong axis; however, as indicated in the User Note in AISC *Specification* Section F7, deflection will usually control the design before there is a significant reduction in flexural strength due to lateral-torsional buckling.

The design and section property tables in the AISC *Manual* were calculated using a design wall thickness of 93% of the nominal wall thickness (see AISC *Specification* Section B4.2). Strength reductions due to local buckling have been accounted for in the AISC *Manual* design tables. The selection of a square HSS with compact flanges is illustrated in Example F.6. The provisions for a rectangular HSS with noncompact flanges is illustrated in Example F.7. The provisions for a square HSS with slender flanges are illustrated in Example F.8. Available flexural strengths of rectangular and square HSS are listed in Tables 3-12 and 3-13, respectively. If HSS members are specified using ASTM A1065 or ASTM A1085 material, the design wall thickness may be taken equal to the nominal wall thickness.

F8. ROUND HSS

The definition of HSS encompasses both tube and pipe products. The lateral-torsional buckling limit state does not apply, but round HSS are subject to strength reductions from local buckling. Available strengths of round HSS and Pipes are listed in AISC *Manual* Tables 3-14 and 3-15, respectively. The tabulated properties and available flexural strengths of these shapes in the AISC *Manual* are calculated using a design wall thickness of 93% of the nominal wall thickness. The design of a Pipe is illustrated in Example F.9. If round HSS members are specified using ASTM A1085 material, the design wall thickness may be taken equal to the nominal wall thickness.

F9. TEES AND DOUBLE ANGLES LOADED IN THE PLANE OF SYMMETRY

The AISC *Specification* provides a check for flange local buckling, which applies only when a noncompact or slender flange is in compression due to flexure. This limit state will seldom govern. A check for local buckling of the tee stem in flexural compression was added in the 2010 edition of the *Specification*. The provisions were expanded to include local buckling of double-angle web legs in flexural compression in the 2016 edition. Attention should be given to end conditions of tees to avoid inadvertent fixed end moments that induce compression in the web unless this limit state is checked. The design of a WT-shape in bending is illustrated in Example F.10.

F10. SINGLE ANGLES

Section F10 of the AISC *Specification* permits the flexural design of single angles using either the principal axes or geometric axes (x- and y-axes). When designing single angles without continuous bracing using the geometric axis design provisions, M_y must be multiplied by 0.80 for use in Equations F10-1, F10-2 and F10-3. The design of a single angle in bending is illustrated in Example F.11.

F11. RECTANGULAR BARS AND ROUNDS

The AISC *Manual* does not include design tables for these shapes. The local buckling limit state does not apply to any bars. With the exception of rectangular bars bent about the strong axis, solid square, rectangular and round bars are not subject to lateral-torsional buckling and are governed by the yielding limit state only. Rectangular bars bent about the strong axis are subject to lateral-torsional buckling and are checked for this limit state with Equations F11-2 and F11-3, as applicable.

These provisions can be used to check plates and webs of tees in connections. A design example of a rectangular bar in bending is illustrated in Example F.12. A design example of a round bar in bending is illustrated in Example F.13.

F12. UNSYMMETRICAL SHAPES

Due to the wide range of possible unsymmetrical cross sections, specific lateral-torsional and local buckling provisions are not provided in this *Specification* section. A general template is provided, but appropriate literature investigation and engineering judgment are required for the application of this section. A design example of a Z-shaped section in bending is illustrated in Example F.14.

F13. PROPORTIONS OF BEAMS AND GIRDERS

This section of the *Specification* includes a limit state check for tensile rupture due to holes in the tension flange of beams, proportioning limits for I-shaped members, detail requirements for cover plates and connection requirements for built-up beams connected side-to-side. Also included are unbraced length requirements for beams designed using the moment redistribution provisions of AISC *Specification* Section B3.3.

EXAMPLE F.1-1A W-SHAPE FLEXURAL MEMBER DESIGN IN MAJOR AXIS BENDING, CONTINUOUSLY BRACED

Given:

Select a W-shape beam for span and uniform dead and live loads as shown in Figure F.1-1A. Limit the member to a maximum nominal depth of 18 in. Limit the live load deflection to *L*/360. The beam is simply supported and continuously braced. The beam is ASTM A992 material.

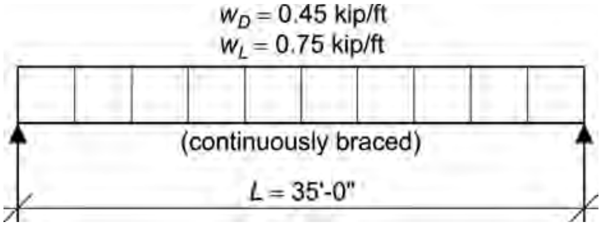


Fig. F.1-1A. Beam loading and bracing diagram.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From Chapter 2 of ASCE/SEI 7, the required flexural strength is:

LRFD	ASD
$w_u = 1.2(0.45\text{ kip/ft}) + 1.6(0.75\text{ kip/ft})$ $= 1.74\text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_u = \dfrac{w_u L^2}{8}$ $= \dfrac{(1.74\text{ kip/ft})(35\text{ ft})^2}{8}$ $= 266\text{ kip-ft}$	$w_a = 0.45\text{ kip/ft} + 0.75\text{ kip/ft}$ $= 1.20\text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_a = \dfrac{w_a L^2}{8}$ $= \dfrac{(1.20\text{ kip/ft})(35\text{ ft})^2}{8}$ $= 184\text{ kip-ft}$

Required Moment of Inertia for Live-Load Deflection Criterion of L/360

$$\Delta_{max} = \frac{L}{360}$$
$$= \frac{(35\text{ ft})(12\text{ in./ft})}{360}$$
$$= 1.17\text{ in.}$$

$$I_{x(reqd)} = \frac{5w_L L^4}{384E\Delta_{max}}$$ (from AISC *Manual* Table 3-23, Case 1)

$$= \frac{5(0.75\text{ kip/ft})(35\text{ ft})^4(12\text{ in./ft})^3}{384(29{,}000\text{ ksi})(1.17\text{ in.})}$$

$$= 746\text{ in.}^4$$

Beam Selection

Select a W18×50 from AISC *Manual* Table 3-3.

$I_x = 800\text{ in.}^4 > 746\text{ in.}^4$ **o.k.**

Per the User Note in AISC *Specification* Section F2, the section is compact. Because the beam is continuously braced and compact, only the yielding limit state applies.

From AISC *Manual* Table 3-2, the available flexural strength is:

LRFD	ASD
$\phi_b M_n = \phi_b M_{px}$ $= 379\text{ kip-ft} > 266\text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = \dfrac{M_{px}}{\Omega_b}$ $= 252\text{ kip-ft} > 184\text{ kip-ft}$ **o.k.**

EXAMPLE F.1-1B W-SHAPE FLEXURAL MEMBER DESIGN IN MAJOR AXIS BENDING, CONTINUOUSLY BRACED

Given:

Verify the available flexural strength of the ASTM A992 W18×50 beam selected in Example F.1-1A by directly applying the requirements of the AISC *Specification*.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W18×50
Z_x = 101 in.^{3}

The required flexural strength from Example F.1-1A is:

LRFD	ASD
$M_u = 266$ kip-ft	$M_a = 184$ kip-ft

Nominal Flexural Strength

Per the User Note in AISC *Specification* Section F2, the section is compact. Because the beam is continuously braced and compact, only the yielding limit state applies.

$$\begin{aligned} M_n &= M_p = F_y Z_x \\ &= (50 \text{ ksi})(101 \text{ in.}^3) \\ &= 5{,}050 \text{ kip-in. or } 421 \text{ kip-ft} \end{aligned}$$ (*Spec*. Eq. F2-1)

Available Flexural Strength

From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(421 \text{ kip-ft})$ $= 379 \text{ kip-ft} > 266 \text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{421 \text{ kip-ft}}{1.67}$ $= 252 \text{ kip-ft} > 184 \text{ kip-ft}$ **o.k.**

EXAMPLE F.1-2A W-SHAPE FLEXURAL MEMBER DESIGN IN MAJOR AXIS BENDING, BRACED AT THIRD POINTS

Given:

Use the AISC *Manual* tables to verify the available flexural strength of the W18×50 beam size selected in Example F.1-1A for span and uniform dead and live loads as shown in Figure F.1-2A. The beam is simply supported and braced at the ends and third points. The beam is ASTM A992 material.

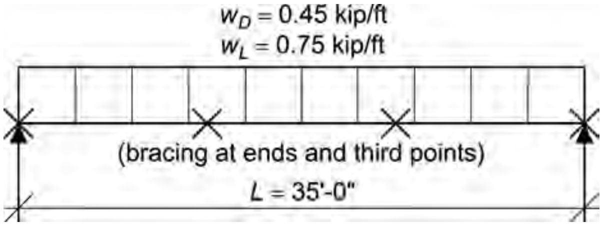


Fig. F.1-2A. Beam loading and bracing diagram.

Solution:

The required flexural strength at midspan from Example F.1-1A is:

LRFD	ASD
$M_u = 266$ kip-ft	$M_a = 184$ kip-ft

Unbraced Length

$$L_b = \frac{35\text{ ft}}{3}$$
$$= 11.7\text{ ft}$$

By inspection, the middle segment will govern. From AISC *Manual* Table 3-1, for a uniformly loaded beam braced at the ends and third points, C_b = 1.01 in the middle segment. Conservatively neglect this small adjustment in this case.

Available Flexural Strength

Enter AISC *Manual* Table 3-10 and find the intersection of the curve for the W18×50 with an unbraced length of 11.7 ft. Obtain the available strength from the appropriate vertical scale to the left.

From AISC *Manual* Table 3-10, the available flexural strength is:

LRFD	ASD
$\phi_b M_n \approx 302\text{ kip-ft} > 266\text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} \approx 201\text{ kip-ft} > 184\text{ kip-ft}$ **o.k.**

EXAMPLE F.1-2B W-SHAPE FLEXURAL MEMBER DESIGN IN MAJOR AXIS BENDING, BRACED AT THIRD POINTS

Given:

Verify the available flexural strength of the W18×50 beam selected in Example F.1-1A with the beam braced at the ends and third points by directly applying the requirements of the AISC *Specification*. The beam is ASTM A992 material.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W18×50
r_y = 1.65 in.
S_x = 88.9 in.^{3}
J = 1.24 in.^{4}
r_{ts} = 1.98 in.
h_o = 17.4 in.

The required flexural strength from Example F.1-1A is:

LRFD	ASD
$M_u = 266$ kip-ft	$M_a = 184$ kip-ft

Nominal Flexural Strength

Calculate C_b. For the lateral-torsional buckling limit state, the nonuniform moment modification factor can be calculated using AISC *Specification* Equation F1-1. For the center segment of the beam, the required moments for AISC *Specification* Equation F1-1 can be calculated as a percentage of the maximum midspan moment as: M_{max} = 1.00, M_A = 0.972, M_B = 1.00, and M_C = 0.972.

$$\begin{aligned} C_b &= \frac{12.5M_{max}}{2.5M_{max}+3M_A+4M_B+3M_C} \\ &= \frac{12.5(1.00)}{2.5(1.00)+3(0.972)+4(1.00)+3(0.972)} \\ &= 1.01 \end{aligned}$$

(*Spec*. Eq. F1-1)

For the end-span beam segments, the required moments for AISC *Specification* Equation F1-1 can be calculated as a percentage of the maximum midspan moment as: M_{max} = 0.889, M_A = 0.306, M_B = 0.556, and M_C = 0.750.

$$\begin{aligned} C_b &= \frac{12.5M_{max}}{2.5M_{max}+3M_A+4M_B+3M_C} \\ &= \frac{12.5(0.889)}{2.5(0.889)+3(0.306)+4(0.556)+3(0.750)} \\ &= 1.46 \end{aligned}$$

(*Spec*. Eq. F1-1)

Thus, the center span, with the higher required strength and lower C_b, will govern.

The limiting laterally unbraced length for the limit state of yielding is:

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \qquad (\textit{Spec}.\text{ Eq. F2-5})$$

$$\begin{aligned} &= 1.76\left(1.65\text{ in.}\right)\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} \\ &= 69.9\text{ in. or }5.83\text{ ft} \end{aligned}$$

The limiting unbraced length for the limit state of inelastic lateral-torsional buckling, with c = 1 from AISC *Specification* Equation F2-8a for doubly symmetric I-shaped members, is:

$$L_r = 1.95 r_{ts}\frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_x h_o}+\sqrt{\left(\frac{Jc}{S_x h_o}\right)^2+6.76\left(\frac{0.7F_y}{E}\right)^2}} \qquad (\textit{Spec}.\text{ Eq. F2-6})$$

$$\begin{aligned} &= 1.95\left(1.98\text{ in.}\right)\left[\frac{29{,}000\text{ ksi}}{0.7\left(50\text{ ksi}\right)}\right]\sqrt{\frac{\left(1.24\text{ in.}^4\right)\left(1.0\right)}{\left(88.9\text{ in.}^3\right)\left(17.4\text{ in.}\right)}+\sqrt{\left[\frac{\left(1.24\text{ in.}^4\right)\left(1.0\right)}{\left(88.9\text{ in.}^3\right)\left(17.4\text{ in.}\right)}\right]^2+6.76\left[\frac{0.7\left(50\text{ ksi}\right)}{29{,}000\text{ ksi}}\right]^2}} \\ &= 203\text{ in. or }16.9\text{ ft} \end{aligned}$$

For a compact beam with an unbraced length of $L_p < L_b \le L_r$, the lesser of either the flexural yielding limit state or the inelastic lateral-torsional buckling limit state controls the nominal strength.

M_p = 5,050 kip-in. (from Example F.1-1B)

$$M_n = C_b\left[M_p - (M_p - 0.7F_y S_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \qquad (\textit{Spec}.\text{ Eq. F2-2})$$

$$\begin{aligned} &= 1.01\left\{5{,}050\text{ kip-in.} - \left[5{,}050\text{ kip-in.} - 0.7\left(50\text{ ksi}\right)\left(88.9\text{ in.}^3\right)\right]\left(\frac{11.7\text{ ft} - 5.83\text{ ft}}{16.9\text{ ft} - 5.83\text{ ft}}\right)\right\} \le 5{,}050\text{ kip-in.} \\ &= 4{,}060\text{ kip-in.} < 5{,}050\text{ kip-in.} \\ &= 4{,}060\text{ kip-in. or }339\text{ kip-ft} \end{aligned}$$

Available Flexural Strength

From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
$\phi_b M_n = 0.90\left(339\text{ kip-ft}\right)$ $= 305\text{ kip-ft} > 266\text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = \dfrac{339\text{ kip-ft}}{1.67}$ $= 203\text{ kip-ft} > 184\text{ kip-ft}$ **o.k.**

EXAMPLE F.1-3A W-SHAPE FLEXURAL MEMBER DESIGN IN MAJOR AXIS BENDING, BRACED AT MIDSPAN

Given:

Use the AISC *Manual* tables to verify the available flexural strength of the W18×50 beam size selected in Example F.1-1A for span and uniform dead and live loads as shown in Figure F.1-3A. The beam is simply supported and braced at the ends and midpoint. The beam is ASTM A992 material.

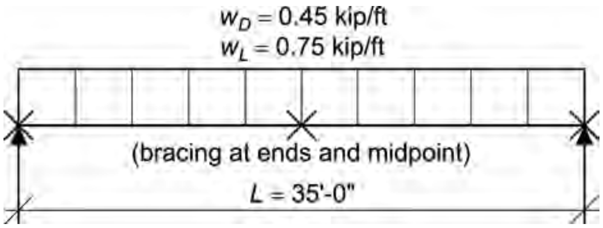


Fig. F.1-3A. Beam loading and bracing diagram.

Solution:

The required flexural strength at midspan from Example F.1-1A is:

LRFD	ASD
$M_u = 266$ kip-ft	$M_a = 184$ kip-ft

Unbraced Length

$$L_b = \frac{35\text{ ft}}{2} = 17.5\text{ ft}$$

From AISC *Manual* Table 3-1, for a uniformly loaded beam braced at the ends and at the center point, C_b = 1.30. There are several ways to make adjustments to AISC *Manual* Table 3-10 to account for C_b greater than 1.0.

Procedure A

Available moments from the sloped and curved portions of the plots from AISC *Manual* Table 3-10 may be multiplied by C_b, but may not exceed the value of the horizontal portion (ϕM_p for LRFD, M_p/Ω for ASD).

Obtain the available strength of a W18×50 with an unbraced length of 17.5 ft from AISC *Manual* Table 3-10.

Enter AISC *Manual* Table 3-10 and find the intersection of the curve for the W18×50 with an unbraced length of 17.5 ft. Obtain the available strength from the appropriate vertical scale to the left.

LRFD	ASD
$\phi_b M_n \approx 222$ kip-ft	$\dfrac{M_n}{\Omega_b} \approx 148$ kip-ft
From AISC *Manual* Table 3-2:	From AISC *Manual* Table 3-2:
$\phi_b M_p = 379$ kip-ft (upper limit on $C_b \phi_b M_n$)	$\dfrac{M_p}{\Omega_b} = 252$ kip-ft (upper limit on $C_b \dfrac{M_n}{\Omega_b}$)

LRFD	ASD
Adjust for C_b. $1.30(222\text{ kip-ft}) = 289\text{ kip-ft}$ Check limit. $289\text{ kip-ft} < \phi_b M_p = 379\text{ kip-ft}$ **o.k.** Check available versus required strength. $289\text{ kip-ft} > 266\text{ kip-ft}$ **o.k.**	Adjust for C_b. $1.30(148\text{ kip-ft}) = 192\text{ kip-ft}$ Check limit. $192\text{ kip-ft} < \dfrac{M_p}{\Omega_b} = 252\text{ kip-ft}$ **o.k.** Check available versus required strength. $192\text{ kip-ft} > 184\text{ kip-ft}$ **o.k.**

Procedure B

For preliminary selection, the required strength can be divided by C_b and directly compared to the strengths in AISC *Manual* Table 3-10. Members selected in this way must be checked to ensure that the required strength does not exceed the available plastic moment strength of the section.

Calculate the adjusted required strength.

LRFD	ASD
$M'_u = \dfrac{266\text{ kip-ft}}{1.30}$ $= 205\text{ kip-ft}$	$M'_a = \dfrac{184\text{ kip-ft}}{1.30}$ $= 142\text{ kip-ft}$

Obtain the available strength for a W18×50 with an unbraced length of 17.5 ft from AISC *Manual* Table 3-10.

LRFD	ASD
$\phi_b M_n \approx 222\text{ kip-ft} > 205\text{ kip-ft}$ **o.k.** $\phi_b M_p = 379\text{ kip-ft} > 266\text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} \approx 148\text{ kip-ft} > 142\text{ kip-ft}$ **o.k.** $\dfrac{M_p}{\Omega_b} = 252\text{ kip-ft} > 184\text{ kip-ft}$ **o.k.**

EXAMPLE F.1-3B W-SHAPE FLEXURAL MEMBER DESIGN IN MAJOR-AXIS BENDING, BRACED AT MIDSPAN

Given:

Verify the available flexural strength of the W18×50 beam selected in Example F.1-1A with the beam braced at the ends and center point by directly applying the requirements of the AISC *Specification*. The beam is ASTM A992 material.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W18×50
r_{ts} = 1.98 in.
S_x = 88.9 in.^{3}
J = 1.24 in.^{4}
h_o = 17.4 in.

The required flexural strength from Example F.1-1A is:

LRFD	ASD
$M_u = 266$ kip-ft	$M_a = 184$ kip-ft

Nominal Flexural Strength

Calculate C_b. The required moments for AISC *Specification* Equation F1-1 can be calculated as a percentage of the maximum midspan moment as: M_{max} = 1.00, M_A = 0.438, M_B = 0.750, and M_C = 0.938.

$$\begin{aligned} C_b &= \frac{12.5M_{max}}{2.5M_{max}+3M_A+4M_B+3M_C} \qquad \text{(}\textit{Spec}\text{. Eq. F1-1)} \\ &= \frac{12.5(1.00)}{2.5(1.00)+3(0.438)+4(0.750)+3(0.938)} \\ &= 1.30 \end{aligned}$$

From AISC *Manual* Table 3-2:

L_p = 5.83 ft
L_r = 16.9 ft

From Example F.1-3A:

L_b = 17.5 ft

For a compact beam with an unbraced length $L_b > L_r$, the limit state of elastic lateral-torsional buckling applies.

Calculate F_{cr}, where c = 1.0 for doubly symmetric I-shapes.

$$F_{cr}=\frac{C_b\pi^2E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1+0.078\frac{Jc}{S_xh_o}\left(\frac{L_b}{r_{ts}}\right)^2} \qquad \text{(}\textit{Spec}\text{. Eq. F2-4)}$$

$$=\frac{1.30\pi^2\left(29{,}000\text{ ksi}\right)}{\left[\frac{(17.5\text{ ft})(12\text{ in./ft})}{1.98\text{ in.}}\right]^2}\sqrt{1+0.078\frac{\left(1.24\text{ in.}^4\right)(1.0)}{\left(88.9\text{ in.}^3\right)(17.4\text{ in.})}\left[\frac{(17.5\text{ ft})(12\text{ in./ft})}{1.98\text{ in.}}\right]^2}$$

$$=43.2\text{ ksi}$$

$$M_p=5{,}050\text{ kip-in. (from Example F.1-1B)}$$

$$M_n=F_{cr}S_x\le M_p \qquad \text{(}\textit{Spec}\text{. Eq. F2-3)}$$

$$=\left(43.2\text{ ksi}\right)\left(88.9\text{ in.}^3\right)\le 5{,}050\text{ kip-in.}$$

$$=3{,}840\text{ kip-in.}<5{,}050\text{ kip-in.}$$

$$=3{,}840\text{ kip-in. or 320 kip-ft}$$

Available Flexural Strength

From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b=0.90$ $\phi_bM_n=0.90\left(320\text{ kip-ft}\right)$ $=288\text{ kip-ft}>266\text{ kip-ft}$ **o.k.**	$\Omega_b=1.67$ $\frac{M_n}{\Omega_b}=\frac{320\text{ kip-ft}}{1.67}$ $=192\text{ kip-ft}>184\text{ kip-ft}$ **o.k.**

EXAMPLE F.2-1A COMPACT CHANNEL FLEXURAL MEMBER, CONTINUOUSLY BRACED

Given:

Using the AISC *Manual* tables, select a channel to serve as a roof edge beam for span and uniform dead and live loads as shown in Figure F.2-1A. The beam is simply supported and continuously braced. Limit the live load deflection to *L*/360. The channel is ASTM A36 material.

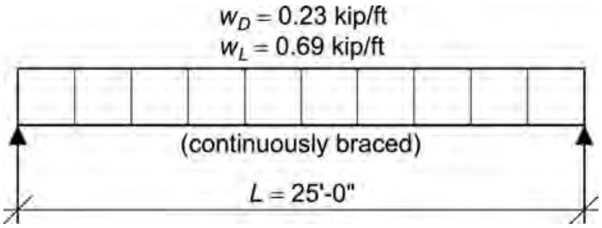


Fig. F.2-1A. Beam loading and bracing diagram.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From Chapter 2 of ASCE/SEI 7, the required flexural strength is:

LRFD	ASD
$w_u = 1.2(0.23\text{ kip/ft}) + 1.6(0.69\text{ kip/ft})$ $= 1.38\text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_u = \dfrac{w_u L^2}{8}$ $= \dfrac{(1.38\text{ kip/ft})(25\text{ ft})^2}{8}$ $= 108\text{ kip-ft}$	$w_a = 0.23\text{ kip/ft} + 0.69\text{ kip/ft}$ $= 0.920\text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_a = \dfrac{w_a L^2}{8}$ $= \dfrac{(0.920\text{ kip/ft})(25\text{ ft})^2}{8}$ $= 71.9\text{ kip-ft}$

Beam Selection

Per the User Note in AISC *Specification* Section F2, all ASTM A36 channels are compact. Because the beam is compact and continuously braced, the yielding limit state governs and $M_n = M_p$. Try C15×33.9 from AISC *Manual* Table 3-8.

LRFD	ASD
$\phi_b M_n = \phi_b M_p$ $= 137\text{ kip-ft} > 108\text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = \dfrac{M_p}{\Omega_b}$ $= 91.3\text{ kip-ft} > 71.9\text{ kip-ft}$ **o.k.**

Live Load Deflection

Limit the live load deflection at the center of the beam to *L*/360.

$$\begin{aligned}\Delta_{max} &= \frac{L}{360}\\ &= \frac{(25\text{ ft})(12\text{ in./ft})}{360}\\ &= 0.833\text{ in.}\end{aligned}$$

For C15×33.9, I_x = 315 in.^{4} from AISC *Manual* Table 1-5.

The maximum calculated deflection is:

$$\begin{aligned}\Delta_{max} &= \frac{5w_L L^4}{384EI} \qquad \text{(from AISC } \textit{Manual} \text{ Table 3-23, Case 1)}\\ &= \frac{5(0.69\text{ kip/ft})(25\text{ ft})^4(12\text{ in./ft})^3}{384(29{,}000\text{ ksi})(315\text{ in.}^4)}\\ &= 0.664\text{ in.} < 0.833\text{ in.} \quad \textbf{o.k.}\end{aligned}$$

EXAMPLE F.2-1B COMPACT CHANNEL FLEXURAL MEMBER, CONTINUOUSLY BRACED

Given:

Verify the available flexural strength of the C15×33.9 beam selected in Example F.2-1A by directly applying the requirements of the AISC *Specification*. The channel is ASTM A36 material.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-5, the geometric properties are as follows:

C15×33.9
$Z_x = 50.8 \text{ in.}^3$

The required flexural strength from Example F.2-1A is:

LRFD	ASD
$M_u = 108 \text{ kip-ft}$	$M_a = 71.9 \text{ kip-ft}$

Nominal Flexural Strength

Per the User Note in AISC *Specification* Section F2, all ASTM A36 C- and MC-shapes are compact.

A channel that is continuously braced and compact is governed by the yielding limit state.

$$\begin{aligned} M_n &= M_p = F_y Z_x \\ &= (36 \text{ ksi})(50.8 \text{ in.}^3) \\ &= 1{,}830 \text{ kip-in. or } 152 \text{ kip-ft} \end{aligned}$$ (*Spec*. Eq. F2-1)

Available Flexural Strength

From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(152 \text{ kip-ft})$ $= 137 \text{ kip-ft} > 108 \text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{152 \text{ kip-ft}}{1.67}$ $= 91.0 \text{ kip-ft} > 71.9 \text{ kip-ft}$ **o.k.**

EXAMPLE F.2-2A COMPACT CHANNEL FLEXURAL MEMBER WITH BRACING AT ENDS AND FIFTH POINTS

Given:

Use the AISC *Manual* tables to verify the available flexural strength of the C15×33.9 beam selected in Example F.2-1A for span and uniform dead and live loads as shown in Figure F.2-2A. The beam is simply supported and braced at the ends and fifth points. The channel is ASTM A36 material.

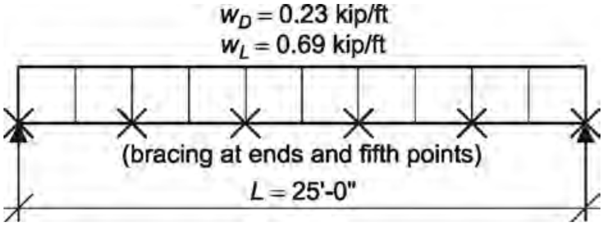


Fig. F.2-2A. Beam loading and bracing diagram.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

The center segment will govern by inspection.

The required flexural strength at midspan from Example F.2-1A is:

LRFD	ASD
$M_u = 108$ kip-ft	$M_a = 71.9$ kip-ft

From AISC *Manual* Table 3-1, with an almost uniform moment across the center segment, C_b = 1.00; therefore, no adjustment is required.

Unbraced Length

$$L_b = \frac{25\text{ ft}}{5}$$
$$= 5.00\text{ ft}$$

Obtain the strength of the C15×33.9 with an unbraced length of 5.00 ft from AISC *Manual* Table 3-11.

Enter AISC *Manual* Table 3-11 and find the intersection of the curve for the C15×33.9 with an unbraced length of 5.00 ft. Obtain the available strength from the appropriate vertical scale to the left.

LRFD	ASD
$\phi_b M_n \approx 130\text{ kip-ft} > 108\text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} \approx 87.0\text{ kip-ft} > 71.9\text{ kip-ft}$ **o.k.**

EXAMPLE F.2-2B COMPACT CHANNEL FLEXURAL MEMBER WITH BRACING AT ENDS AND FIFTH POINTS

Given:

Verify the results from Example F.2-2A by directly applying the requirements of the AISC *Specification*.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-5, the geometric properties are as follows:

C15×33.9
$S_x = 42.0$ in.^{3}

The required flexural strength from Example F.2-1A is:

LRFD	ASD
$M_u = 108$ kip-ft	$M_a = 71.9$ kip-ft

Available Flexural Strength

Per the User Note in AISC *Specification* Section F2, all ASTM A36 C- and MC-shapes are compact.

From AISC *Manual* Table 3-1, for the center segment of a uniformly loaded beam braced at the ends and the fifth points:

$C_b = 1.00$

From AISC *Manual* Table 3-8, for a C15×33.9:

$L_p = 3.75$ ft
$L_r = 14.5$ ft

From Example F2.2A:

$L_b = 5.00$ ft

For a compact channel with $L_p < L_b \leq L_r$, the lesser of the flexural yielding limit state or the inelastic lateral-torsional buckling limit state controls the available flexural strength.

The nominal flexural strength based on the flexural yielding limit state, from Example F.2-1B, is:

$$\begin{aligned} M_n &= M_p \\ &= 1{,}830 \text{ kip-in.} \end{aligned}$$

The nominal flexural strength based on the lateral-torsional buckling limit state is:

$$M_n = C_b\left[M_p - \left(M_p - 0.7F_yS_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \qquad \text{(}\textit{Spec}\text{. Eq. F2-2)}$$

$$\begin{aligned}
&= 1.00\left\{1{,}830 \text{ kip-in.} - \left[1{,}830 \text{ kip-in.} - 0.7\left(36 \text{ ksi}\right)\left(42.0 \text{ in.}^3\right)\right]\left(\frac{5.00 \text{ ft} - 3.75 \text{ ft}}{14.5 \text{ ft} - 3.75 \text{ ft}}\right)\right\} \le 1{,}830 \text{ kip-in.}\\
&= 1{,}740 \text{ kip-in.} < 1{,}830 \text{ kip-in.}\\
&= 1{,}740 \text{ kip-in. or } 145 \text{ kip-ft}
\end{aligned}$$

Available Flexural Strength

From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90\left(145 \text{ kip-ft}\right)$ $= 131 \text{ kip-ft} > 108 \text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{145 \text{ kip-ft}}{1.67}$ $= 86.8 \text{ kip-ft} > 71.9 \text{ kip-ft}$ **o.k.**

EXAMPLE F.3A W-SHAPE FLEXURAL MEMBER WITH NONCOMPACT FLANGES IN MAJOR AXIS BENDING

Given:

Using the AISC *Manual* tables, select a W-shape beam for span, uniform dead load, and concentrated live loads as shown in Figure F.3A. The beam is simply supported and continuously braced. Also calculate the deflection. The beam is ASTM A992 material.

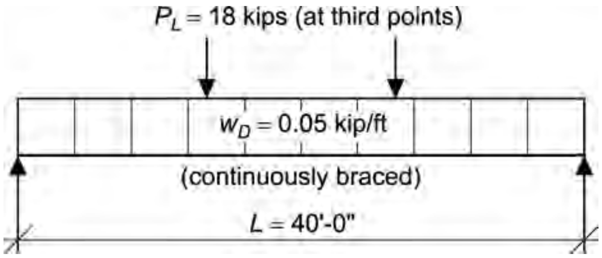


Fig. F.3A. Beam loading and bracing diagram.

Note: A beam with noncompact flanges will be selected to demonstrate that the tabulated values of the AISC *Manual* account for flange compactness.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From Chapter 2 of ASCE/SEI 7, the required flexural strength at midspan is:

LRFD	ASD
$w_u = 1.2(0.05\text{ kip/ft})$ $= 0.0600\text{ kip/ft}$	$w_a = 0.05\text{ kip/ft}$
$P_u = 1.6(18\text{ kips})$ $= 28.8\text{ kips}$	$P_a = 18\text{ kips}$
From AISC *Manual* Table 3-23, Cases 1 and 9:	From AISC *Manual* Table 3-23, Cases 1 and 9:
$M_u = \dfrac{w_u L^2}{8} + P_u a$ $= \dfrac{(0.0600\text{ kip/ft})(40\text{ ft})^2}{8} + (28.8\text{ kips})\left(\dfrac{40\text{ ft}}{3}\right)$ $= 396\text{ kip-ft}$	$M_a = \dfrac{w_a L^2}{8} + P_a a$ $= \dfrac{(0.05\text{ kip/ft})(40\text{ ft})^2}{8} + (18\text{ kips})\left(\dfrac{40\text{ ft}}{3}\right)$ $= 250\text{ kip-ft}$

Beam Selection

For a continuously braced W-shape, the available flexural strength equals the available plastic flexural strength.

Select the lightest section providing the required strength from the bold entries in AISC *Manual* Table 3-2.

Try a W21×48.

This beam has a noncompact compression flange at F_y = 50 ksi as indicated by footnote "f" in AISC *Manual* Table 3-2. This shape is also footnoted in AISC *Manual* Table 1-1.

From AISC *Manual* Table 3-2, the available flexural strength is:

LRFD	ASD
$\phi_b M_n = \phi_b M_{px}$ $= 398 \text{ kip-ft} > 396 \text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = \dfrac{M_{px}}{\Omega_b}$ $= 265 \text{ kip-ft} > 250 \text{ kip-ft}$ **o.k.**

Note: The value M_{px} in AISC *Manual* Table 3-2 includes the strength reductions due to the shape being noncompact.

Deflection

From AISC *Manual* Table 1-1:

I_x = 959 in.^{4}

The maximum deflection occurs at the center of the beam.

$$\Delta_{max} = \frac{5w_D L^4}{384EI} + \frac{23P_L L^3}{648EI} \qquad \text{(AISC \textit{Manual} Table 3-23, Cases 1 and 9)}$$

$$= \frac{5(0.05\text{ kip/ft})(40\text{ ft})^4(12\text{ in./ft})^3}{384(29{,}000\text{ ksi})\left(959\text{ in.}^4\right)} + \frac{23(18\text{ kips})(40\text{ ft})^3(12\text{ in./ft})^3}{648(29{,}000\text{ ksi})\left(959\text{ in.}^4\right)}$$

$$= 2.64\text{ in.}$$

This deflection can be compared with the appropriate deflection limit for the application. Deflection will often be more critical than strength in beam design.

EXAMPLE F.3B W-SHAPE FLEXURAL MEMBER WITH NONCOMPACT FLANGES IN MAJOR AXIS BENDING

Given:

Verify the results from Example F.3A by directly applying the requirements of the AISC *Specification*.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W21×48
$S_x = 93.0 \text{ in.}^3$
$Z_x = 107 \text{ in.}^3$
$\dfrac{b_f}{2t_f} = 9.47$

The required flexural strength from Example F.3A is:

LRFD	ASD
$M_u = 396$ kip-ft	$M_a = 250$ kip-ft

Flange Slenderness

$$\lambda = \frac{b_f}{2t_f} = 9.47$$

The limiting width-to-thickness ratios for the compression flange are:

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} = 9.15$$

(*Spec*. Table B4.1b, Case 10)

$$\lambda_{rf} = 1.0\sqrt{\frac{E}{F_y}} = 1.0\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} = 24.1$$

(*Spec*. Table B4.1b, Case 10)

$\lambda_{pf} < \lambda < \lambda_{rf}$, therefore, the compression flange is noncompact. This could also be determined from the footnote “f” in AISC *Manual* Table 1-1.

Nominal Flexural Strength

Because the beam is continuously braced, and therefore not subject to lateral-torsional buckling, the available strength is based on the limit state of compression flange local buckling. From AISC *Specification* Section F3.2:

$$\begin{aligned} M_p &= F_y Z_x \\ &= (50 \text{ ksi})(107 \text{ in.}^3) \\ &= 5{,}350 \text{ kip-in. or } 446 \text{ kip-ft} \end{aligned}$$

(*Spec*. Eq. F2-1)

$$\begin{aligned} M_n &= \left[M_p - (M_p - 0.7F_y S_x)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \\ &= \left\{ 5{,}350 \text{ kip-in.} - \left[5{,}350 \text{ kip-in.} - 0.7(50 \text{ ksi})(93.0 \text{ in.}^3) \right]\left(\frac{9.47 - 9.15}{24.1 - 9.15} \right) \right\} \\ &= 5{,}310 \text{ kip-in. or } 442 \text{ kip-ft} \end{aligned}$$

(*Spec*. Eq. F3-1)

Available Flexural Strength

From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(442 \text{ kip-ft})$ $= 398 \text{ kip-ft} > 396 \text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{442 \text{ kip-ft}}{1.67}$ $= 265 \text{ kip-ft} > 250 \text{ kip-ft}$ **o.k.**

Note that these available strengths are identical to the tabulated values in AISC *Manual* Table 3-2, as shown in Example F.3A, which account for the noncompact flange.

EXAMPLE F.4 W-SHAPE FLEXURAL MEMBER, SELECTION BY MOMENT OF INERTIA FOR MAJOR AXIS BENDING

Given:

Using the AISC *Manual* tables, select a W-shape using the moment of inertia required to limit the live load deflection to 1.00 in. for span and uniform dead and live loads as shown in Figure F.4. The beam is simply supported and continuously braced. The beam is ASTM A992 material.

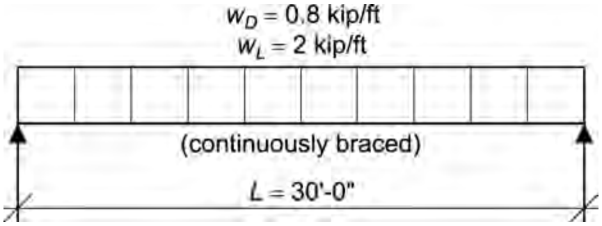


Fig. F.4. Beam loading and bracing diagram.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From Chapter 2 of ASCE/SEI 7, the required flexural strength is:

LRFD	ASD
$w_u = 1.2(0.8\text{ kip/ft}) + 1.6(2\text{ kip/ft})$ $= 4.16\text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_u = \dfrac{w_u L^2}{8}$ $= \dfrac{(4.16\text{ kip/ft})(30\text{ ft})^2}{8}$ $= 468\text{ kip-ft}$	$w_a = 0.8\text{ kip/ft} + 2\text{ kip/ft}$ $= 2.80\text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_a = \dfrac{w_a L^2}{8}$ $= \dfrac{(2.80\text{ kip/ft})(30\text{ ft})^2}{8}$ $= 315\text{ kip-ft}$

Minimum Required Moment of Inertia

The maximum live load deflection, Δ_{max}, occurs at midspan and is calculated as:

$$\Delta_{max} = \frac{5 w_L L^4}{384 EI} \qquad \text{(AISC } \textit{Manual} \text{ Table 3-23, Case 1)}$$

Rearranging and substituting Δ_{max} = 1.00 in.,

$$
\begin{aligned}
I_{min} &= \frac{5w_L L^4}{384E\Delta_{max}} \\
&= \frac{5\left(2\text{ kip/ft}\right)\left(30\text{ ft}\right)^4\left(12\text{ in./ft}\right)^3}{384\left(29{,}000\text{ ksi}\right)\left(1.00\text{ in.}\right)} \\
&= 1{,}260\text{ in.}^4
\end{aligned}
$$

Beam Selection

Select the lightest section with the required moment of inertia from the bold entries in AISC *Manual* Table 3-3.

Try a W24×55.

$I_x = 1{,}350\text{ in.}^4 > 1{,}260\text{ in.}^4$ **o.k.**

Because the W24×55 is continuously braced and compact, its strength is governed by the yielding limit state and AISC *Specification* Section F2.1.

From AISC *Manual* Table 3-2, the available flexural strength is:

LRFD	ASD
$\phi_b M_n = \phi_b M_{px}$ $= 503\text{ kip-ft} > 468\text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = \dfrac{M_{px}}{\Omega_b}$ $= 334\text{ kip-ft} > 315\text{ kip-ft}$ **o.k.**

EXAMPLE F.5 I-SHAPED FLEXURAL MEMBER IN MINOR AXIS BENDING

Given:

Using the AISC *Manual* tables, select a W-shape beam loaded on its minor axis for span and uniform dead and live loads as shown in Figure F.5. Limit the live load deflection to $L/240$. The beam is simply supported and braced only at the ends. The beam is ASTM A992 material.

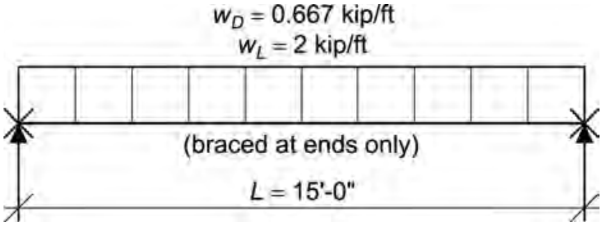


Fig. F.5. Beam loading and bracing diagram.

Note: Although not a common design case, this example is being used to illustrate AISC *Specification* Section F6 (I-shaped members and channels bent about their minor axis).

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From Chapter 2 of ASCE/SEI 7, the required flexural strength is:

LRFD	ASD
$w_u = 1.2(0.667 \text{ kip/ft}) + 1.6(2 \text{ kip/ft})$ $= 4.00 \text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_u = \dfrac{w_u L^2}{8}$ $= \dfrac{(4.00 \text{ kip/ft})(15 \text{ ft})^2}{8}$ $= 113 \text{ kip-ft}$	$w_a = 0.667 \text{ kip/ft} + 2 \text{ kip/ft}$ $= 2.67 \text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_a = \dfrac{w_a L^2}{8}$ $= \dfrac{(2.67 \text{ kip/ft})(15 \text{ ft})^2}{8}$ $= 75.1 \text{ kip-ft}$

Minimum Required Moment of Inertia

The maximum live load deflection permitted is:

$$\Delta_{max} = \frac{L}{240}$$
$$= \frac{(15 \text{ ft})(12 \text{ in./ft})}{240}$$
$$= 0.750 \text{ in.}$$

$$I_{y,reqd} = \frac{5w_L L^4}{384E\Delta_{max}}$$ (modified AISC *Manual* Table 3-23, Case 1)

$$= \frac{5(2\text{ kip/ft})(15\text{ ft})^4(12\text{ in./ft})^3}{384(29{,}000\text{ ksi})(0.750\text{ in.})}$$

$$= 105\text{ in.}^4$$

Beam Selection

Select the lightest section from the bold entries in AISC *Manual* Table 3-5.

Try a W12×58.

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W12×58
S_y = 21.4 in.^{3}
Z_y = 32.5 in.^{3}
I_y = 107 in.^{4} > 105 in.^{4} **o.k. (**for deflection requirement)

Nominal Flexural Strength

AISC *Specification* Section F6 applies. Because the W12×58 has compact flanges per the User Note in this Section, the yielding limit state governs the design.

$$M_n = M_p = F_y Z_y \le 1.6F_y S_y$$ (*Spec*. Eq. F6-1)

$$= (50\text{ ksi})(32.5\text{ in.}^3) \le 1.6(50\text{ ksi})(21.4\text{ in.}^3)$$

$$= 1{,}630\text{ kip-in.} < 1{,}710\text{ kip-in.}$$

$$= 1{,}630\text{ kip-in or }136\text{ kip-ft}$$

Available Flexural Strength

From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(136\text{ kip-ft})$ $= 122\text{ kip-ft} > 113\text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $\frac{M_n}{\Omega_b} = \frac{136\text{ kip-ft}}{1.67}$ $= 81.4\text{ kip-ft} > 75.1\text{ kip-ft}$ **o.k.**

EXAMPLE F.6 SQUARE HSS FLEXURAL MEMBER WITH COMPACT FLANGES

Given:

Using the AISC *Manual* tables, select a square HSS beam for span and uniform dead and live loads as shown in Figure F.6. Limit the live load deflection to *L*/240. The beam is simply supported and continuously braced. The HSS is ASTM A500 Grade C material.

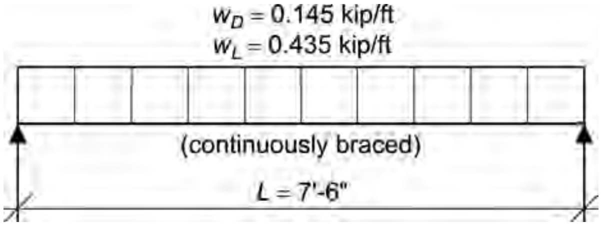


Fig. F.6. Beam loading and bracing diagram.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500 Grade C, rectangular HSS
F_y = 50 ksi
F_u = 62 ksi

From Chapter 2 of ASCE/SEI 7, the required flexural strength is:

LRFD	ASD
$$w_u = 1.2(0.145 \text{ kip/ft}) + 1.6(0.435 \text{ kip/ft}) = 0.870 \text{ kip/ft}$$ From AISC *Manual* Table 3-23, Case 1: $$M_u = \frac{w_u L^2}{8} = \frac{(0.870 \text{ kip/ft})(7.5 \text{ ft})^2}{8} = 6.12 \text{ kip-ft}$$	$$w_a = 0.145 \text{ kip/ft} + 0.435 \text{ kip/ft} = 0.580 \text{ kip/ft}$$ From AISC *Manual* Table 3-23, Case 1: $$M_a = \frac{w_a L^2}{8} = \frac{(0.580 \text{ kip/ft})(7.5 \text{ ft})^2}{8} = 4.08 \text{ kip-ft}$$

Minimum Required Moment of Inertia

The maximum live load deflection permitted is:

$$\Delta_{max} = \frac{L}{240} = \frac{(7.5 \text{ ft})(12 \text{ in./ft})}{240} = 0.375 \text{ in.}$$

Determine the minimum required moment of inertia as follows.

$$I_{req} = \frac{5w_L L^4}{384E\Delta_{max}}$$ (from AISC *Manual* Table 3-23, Case 1)

$$= \frac{5(0.435\text{ kip/ft})(7.5\text{ ft})^4(12\text{ in./ft})^3}{384(29{,}000\text{ ksi})(0.375\text{ in.})}$$

$$= 2.85\text{ in.}^4$$

Beam Selection

Select an HSS with a minimum I_x of 2.85 in.^{4}, using AISC *Manual* Table 1-12, and having adequate available strength, using AISC *Manual* Table 3-13.

Try an HSS3½×3½×⅛.

From AISC *Manual* Table 1-12,

$I_x = 2.90\text{ in.}^4 > 2.85\text{ in.}^4$ **o.k.**

From AISC *Manual* Table 3-13, the available flexural strength is:

LRFD	ASD
$\phi_b M_n = 7.21\text{ kip-ft} > 6.12\text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = 4.79\text{ kip-ft} > 4.08\text{ kip-ft}$ **o.k.**

EXAMPLE F.7A RECTANGULAR HSS FLEXURAL MEMBER WITH NONCOMPACT FLANGES

Given:

Using the AISC *Manual* tables, select a rectangular HSS beam for span and uniform dead and live loads as shown in Figure F.7A. Limit the live load deflection to *L*/240. The beam is simply supported and braced at the end points only. A noncompact member was selected here to illustrate the relative ease of selecting noncompact shapes from the AISC *Manual*, as compared to designing a similar shape by applying the AISC *Specification* requirements directly, as shown in Example F.7B. The HSS is ASTM A500 Grade C material.

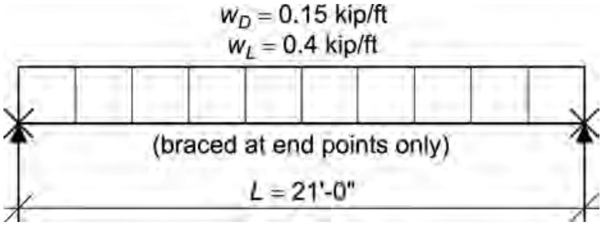


Fig. F.7A. Beam loading and bracing diagram.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500 Grade C, rectangular HSS
F_y = 50 ksi
F_u = 62 ksi

From Chapter 2 of ASCE/SEI 7, the required flexural strength is:

LRFD	ASD
$w_u = 1.2(0.15\text{ kip/ft}) + 1.6(0.4\text{ kip/ft})$ $= 0.820\text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_u = \dfrac{w_u L^2}{8}$ $= \dfrac{(0.820\text{ kip/ft})(21\text{ ft})^2}{8}$ $= 45.2\text{ kip-ft}$	$w_a = 0.15\text{ kip/ft} + 0.4\text{ kip/ft}$ $= 0.550\text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_a = \dfrac{w_a L^2}{8}$ $= \dfrac{(0.550\text{ kip/ft})(21\text{ ft})^2}{8}$ $= 30.3\text{ kip-ft}$

Minimum Required Moment of Inertia

The maximum live load deflection permitted is:

$$\begin{aligned}\Delta_{max} &= \frac{L}{240}\\ &= \frac{(21\text{ ft})(12\text{ in./ft})}{240}\\ &= 1.05\text{ in.}\end{aligned}$$

Determine the minimum required moment of inertia as follows:

$$I_{min} = \frac{5w_L L^4}{384E\Delta_{max}}$$ (from AISC *Manual* Table 3-23, Case 1)

$$= \frac{5\left(0.4 \text{ kip/ft}\right)\left(21 \text{ ft}\right)^4 \left(12 \text{ in./ft}\right)^3}{384\left(29{,}000 \text{ ksi}\right)\left(1.05 \text{ in.}\right)}$$

$$= 57.5 \text{ in.}^4$$

Beam Selection

Select a rectangular HSS with a minimum I_x of 57.5 in.^{4}, using AISC *Manual* Table 1-11, and having adequate available strength, using AISC *Manual* Table 3-12.

Try an HSS10×6×3⁄16 oriented in the strong direction. This rectangular HSS section was purposely selected for illustration purposes because it has a noncompact flange. See AISC *Manual* Table 1-12A for compactness criteria.

$I_x = 74.6 \text{ in.}^4 > 57.5 \text{ in.}^4$ **o.k.**

From AISC *Manual* Table 3-12, the available flexural strength is:

LRFD	ASD
$\phi_b M_n = 59.7 \text{ kip-ft} > 45.2 \text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = 39.7 \text{ kip-ft} > 30.3 \text{ kip-ft}$ **o.k.**

Note: Because AISC *Manual* Table 3-12 does not account for lateral-torsional buckling, it needs to be checked using AISC *Specification* Section F7.4.

As discussed in the User Note to AISC *Specification* Section F7.4, lateral-torsional buckling will not occur in square sections or sections bending about their minor axis. In HSS sizes, deflection will often occur before there is a significant reduction in flexural strength due to lateral-torsional buckling. See Example F.7B for the calculation accounting for lateral-torsional buckling for the HSS10×6×3⁄16.

EXAMPLE F.7B RECTANGULAR HSS FLEXURAL MEMBER WITH NONCOMPACT FLANGES

Given:

In Example F.7A the required information was easily determined by consulting the tables of the AISC *Manual*. The purpose of the following calculation is to demonstrate the use of the AISC *Specification* to calculate the flexural strength of an HSS member with a noncompact compression flange. The HSS is ASTM A500 Grade C material.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500 Grade C, rectangular HSS
F_y = 50 ksi
F_u = 62 ksi

From AISC *Manual* Table 1-11, the geometric properties are as follows:

HSS10×6×3⁄16
A_g = 5.37 in.^{2}
Z_x = 18.0 in.^{3}
S_x = 14.9 in.^{3}
r_y = 2.52 in.
J = 73.8 in.^{4}
b/t = 31.5
h/t = 54.5

Flange Compactness

$$\begin{aligned}\lambda &= \frac{b}{t_f}\\ &= \frac{b}{t}\\ &= 31.5\end{aligned}$$

From AISC *Specification* Table B4.1b, Case 17, the limiting width-to-thickness ratios for the flange are:

$$\begin{aligned}\lambda_p &= 1.12\sqrt{\frac{E}{F_y}}\\ &= 1.12\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}\\ &= 27.0\end{aligned}$$

$$\begin{aligned}\lambda_r &= 1.40\sqrt{\frac{E}{F_y}}\\ &= 1.40\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}\\ &= 33.7\end{aligned}$$

$\lambda_p < \lambda < \lambda_r$; therefore, the flange is noncompact and AISC *Specification* Equation F7-2 applies.

Web Compactness

$$\begin{aligned}\lambda &= \frac{h}{t}\\ &= 54.5\end{aligned}$$

From AISC *Specification* Table B4.1b, Case 19, the limiting width-to-thickness ratio for the web is:

$$\begin{aligned}\lambda_p &= 2.42\sqrt{\frac{E}{F_y}}\\ &= 2.42\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}\\ &= 58.3\end{aligned}$$

$\lambda < \lambda_p$; therefore, the web is compact and the limit state of web local buckling does not apply.

Nominal Flexural Strength

Flange Local Buckling

From AISC *Specification* Section F7.2(b), the limit state of flange local buckling applies for HSS with noncompact flanges and compact webs.

$$\begin{aligned}M_p &= F_yZ_x \qquad \text{(from } \textit{Spec}\text{. Eq. F7-1)}\\ &= (50\text{ ksi})(18.0\text{ in.}^3)\\ &= 900\text{ kip-in.}\end{aligned}$$

$$\begin{aligned}M_n &= M_p - (M_p - F_yS)\left(3.57\frac{b}{t_f}\sqrt{\frac{F_y}{E}} - 4.0\right) \le M_p \qquad (\textit{Spec}\text{. Eq. F7-2})\\ &= 900\text{ kip-in.} - \left[900\text{ kip-in.} - (50\text{ ksi})(14.9\text{ in.}^3)\right]\left[3.57(31.5)\sqrt{\frac{50\text{ ksi}}{29{,}000\text{ ksi}}} - 4.0\right] \le 900\text{ kip-in.}\\ &= 796\text{ kip-in.} < 900\text{ kip-in.}\\ &= 796\text{ kip-in. or } 66.4\text{ kip-ft}\end{aligned}$$

Yielding and Lateral-Torsional Buckling

Determine the limiting laterally unbraced lengths for the limit state of yielding and the limit state of inelastic lateral-torsional buckling using AISC *Specification* Section F7.4.

$$\begin{aligned}L_b &= (21\text{ ft})(12\text{ in./ft})\\ &= 252\text{ in.}\end{aligned}$$

$$\begin{aligned}L_p &= 0.13Er_y\frac{\sqrt{JA_g}}{M_p} \qquad (\textit{Spec}\text{. Eq. F7-12})\\ &= 0.13(29{,}000\text{ ksi})(2.52\text{ in.})\frac{\sqrt{(73.8\text{ in.}^4)(5.37\text{ in.}^2)}}{900\text{ kip-in.}}\\ &= 210\text{ in.}\end{aligned}$$

$$L_r = 2Er_y \frac{\sqrt{JA_g}}{0.7F_yS_x} \qquad \text{(}\textit{Spec}\text{. Eq. F7-13)}$$

$$= 2(29{,}000\text{ ksi})(2.52\text{ in.})\frac{\sqrt{(73.8\text{ in.}^4)(5.37\text{ in.}^2)}}{0.7(50\text{ ksi})(14.9\text{ in.}^3)}$$

$$= 5{,}580\text{ in.}$$

For the lateral-torsional buckling limit state, the lateral-torsional buckling modification factor can be calculated using AISC *Specification* Equation F1-1. For the beam, the required moments for AISC *Specification* Equation F1-1 can be calculated as a percentage of the maximum midspan moment as: M_{max} = 1.00, M_A = 0.750, M_B = 1.00, and M_C = 0.750.

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \qquad \text{(}\textit{Spec}\text{. Eq. F1-1)}$$

$$= \frac{12.5(1.00)}{2.5(1.00) + 3(0.750) + 4(1.00) + 3(0.750)}$$

$$= 1.14$$

Since $L_p < L_b < L_r$, the nominal moment strength considering lateral-torsional buckling is given by:

$$M_n = C_b\left[M_p - (M_p - 0.7F_yS_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \qquad \text{(}\textit{Spec}\text{. Eq. F7-10)}$$

$$= 1.14\left\{900\text{ kip-in.} - \left[900\text{ kip-in.} - 0.7(50\text{ ksi})(14.9\text{ in.}^3)\right]\left(\frac{252\text{ in.} - 210\text{ in.}}{5{,}580\text{ in.} - 210\text{ in.}}\right)\right\} \le 900\text{ kip-in.}$$

$$= 1{,}020\text{ kip-in.} > 900\text{ kip-in.}$$

$$= 900\text{ kip-in. or } 75.0\text{ kip-ft}$$

Available Flexural Strength

The nominal strength is controlled by flange local buckling and therefore:

$$M_n = 66.4\text{ kip-ft}$$

From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(66.4\text{ kip-ft})$ $= 59.8\text{ kip-ft}$	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{66.4\text{ kip-ft}}{1.67}$ $= 39.8\text{ kip-ft}$

EXAMPLE F.8A SQUARE HSS FLEXURAL MEMBER WITH SLENDER FLANGES

Given:

Using AISC *Manual* tables, verify the strength of an HSS8×8×3⁄16 beam for span and uniform dead and live loads as shown in Figure F.8A. Limit the live load deflection to *L*/240. The beam is simply supported and continuously braced. The HSS is ASTM A500 Grade C material.

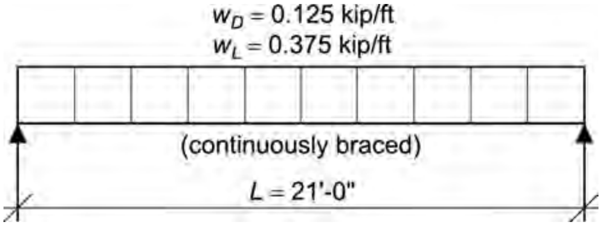


Fig. F.8A. Beam loading and bracing diagram.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500 Grade C
F_y = 50 ksi
F_u = 62 ksi

From AISC *Manual* Table 1-12, the geometric properties are as follows:

HSS8×8×3⁄16
$I_x = I_y = 54.4 \text{ in.}^4$

From Chapter 2 of ASCE/SEI 7, the required flexural strength is:

LRFD	ASD
$w_u = 1.2(0.125\text{ kip/ft}) + 1.6(0.375\text{ kip/ft})$ $= 0.750\text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_u = \dfrac{w_u L^2}{8}$ $= \dfrac{(0.750\text{ kip/ft})(21.0\text{ ft})^2}{8}$ $= 41.3\text{ kip-ft}$	$w_a = 0.125\text{ kip/ft} + 0.375\text{ kip/ft}$ $= 0.500\text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_a = \dfrac{w_a L^2}{8}$ $= \dfrac{(0.500\text{ kip/ft})(21.0\text{ ft})^2}{8}$ $= 27.6\text{ kip-ft}$

From AISC *Manual* Table 3-13, the available flexural strength is:

LRFD	ASD
$\phi_b M_n = 46.3\text{ kip-ft} > 41.3\text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = 30.8\text{ kip-ft} > 27.6\text{ kip-ft}$ **o.k.**

Note that the strengths given in AISC *Manual* Table 3-13 incorporate the effects of noncompact and slender elements.

Deflection

The maximum live load deflection permitted is:

$$
\begin{aligned}
\Delta_{max} &= \frac{L}{240} \\
&= \frac{(21.0\text{ ft})(12\text{ in./ft})}{240} \\
&= 1.05\text{ in.}
\end{aligned}
$$

The calculated deflection is:

$$
\Delta = \frac{5w_L L^4}{384EI}
$$

(modified AISC *Manual* Table 3-23 Case 1)

$$
\begin{aligned}
&= \frac{5(0.375\text{ kip/ft})(21.0\text{ ft})^4(12\text{ in./ft})^3}{384(29{,}000\text{ ksi})(54.4\text{ in.}^4)} \\
&= 1.04\text{ in.} < 1.05\text{ in.} \quad \textbf{o.k.}
\end{aligned}
$$

EXAMPLE F.8B SQUARE HSS FLEXURAL MEMBER WITH SLENDER FLANGES

Given:

In Example F.8A the available strengths were easily determined from the tables of the AISC *Manual*. The purpose of the following calculation is to demonstrate the use of the AISC *Specification* to calculate the flexural strength of the HSS beam given in Example F.8A. The HSS is ASTM A500 Grade C material.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500 Grade C, rectangular HSS
F_y = 50 ksi
F_u = 62 ksi

From AISC *Manual* Table 1-12, the geometric properties are as follows:

HSS8×8×3⁄16
I = 54.4 in.^{4}
Z = 15.7 in.^{3}
S = 13.6 in.^{3}
B = 8.00 in.
H = 8.00 in.
t = 0.174 in.
b/t = 43.0
h/t = 43.0

The required flexural strength from Example F.8A is:

LRFD	ASD
$M_u = 41.3$ kip-ft	$M_a = 27.6$ kip-ft

Flange Slenderness

The outside corner radii of HSS shapes are taken as 1.5*t* and the design thickness is used in accordance with AISC *Specification* Section B4.1b to check compactness.

Determine the limiting ratio for a slender HSS flange in flexure from AISC *Specification* Table B4.1b, Case 17.

$$\begin{aligned}\lambda_r &= 1.40\sqrt{\frac{E}{F_y}} \\ &= 1.40\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} \\ &= 33.7\end{aligned}$$

$$\begin{aligned}\lambda &= \frac{b}{t} \\ &= \frac{b}{t_f} \\ &= 43.0 > \lambda_r;\ \text{therefore, the flange is slender}\end{aligned}$$

Web Slenderness

Determine the limiting ratio for a compact web in flexure from AISC *Specification* Table B4.1b, Case 19.

$$\begin{aligned}\lambda_p &= 2.42\sqrt{\frac{E}{F_y}}\\ &= 2.42\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}\\ &= 58.3\end{aligned}$$

$$\begin{aligned}\lambda &= \frac{h}{t}\\ &= 43.0 < \lambda_p;\text{ therefore, the web is compact and the limit state of web local buckling does not apply}\end{aligned}$$

Nominal Flexural Strength

Flange Local Buckling

For HSS sections with slender flanges and compact webs, AISC *Specification* Section F7.2(c) applies.

$$M_n = F_y S_e \qquad \text{(}\textit{Spec}\text{. Eq. F7-3)}$$

From AISC *Specification* Section B4.1b(d), the width of the compression flange is determined as follows:

$$\begin{aligned}b &= 8.00\text{ in.} - 3(0.174\text{ in.})\\ &= 7.48\text{ in.}\end{aligned}$$

Where the effective section modulus, S_e, is determined using the effective width of the compression flange as follows:

$$b_e = 1.92t_f\sqrt{\frac{E}{F_y}}\left(1-\frac{0.38}{b/t_f}\sqrt{\frac{E}{F_y}}\right) \le b \qquad \text{(}\textit{Spec}\text{. Eq. F7-4)}$$

$$\begin{aligned}&= 1.92(0.174\text{ in.})\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}\left[1-\left(\frac{0.38}{43.0}\right)\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}\right] \le 7.48\text{ in.}\\ &= 6.33\text{ in.}\end{aligned}$$

The ineffective width of the compression flange is:

$$\begin{aligned}b - b_e &= 7.48\text{ in.} - 6.33\text{ in.}\\ &= 1.15\text{ in.}\end{aligned}$$

An exact calculation of the effective moment of inertia and section modulus could be performed taking into account the ineffective width of the compression flange and the resulting neutral axis shift. Alternatively, a simpler but slightly conservative calculation can be performed by removing the ineffective width symmetrically from both the top and bottom flanges.

$$
\begin{aligned}
I_{eff} &\approx I_x - \left(\sum \frac{bt^3}{12} + \sum ad^2 \right) \\
&= 54.4 \text{ in.}^4 - 2\left[\frac{(1.15 \text{ in.})(0.174 \text{ in.})^3}{12} + (1.15 \text{ in.})(0.174 \text{ in.})\left(\frac{8.00 \text{ in.} - 0.174 \text{ in.}}{2} \right)^2 \right] \\
&= 48.3 \text{ in.}^4
\end{aligned}
$$

The effective section modulus is calculated as follows:

$$
\begin{aligned}
S_e &= \frac{I_{eff}}{\left(\dfrac{H}{2} \right)} \\
&= \frac{48.3 \text{ in.}^4}{\left(\dfrac{8.00 \text{ in.}}{2} \right)} \\
&= 12.1 \text{ in.}^3
\end{aligned}
$$

$$
\begin{aligned}
M_n &= F_y S_e \qquad \text{(}\textit{Spec}\text{. Eq. F7-3)} \\
&= (50 \text{ ksi})(12.1 \text{ in.}^3) \\
&= 605 \text{ kip-in. or } 50.4 \text{ kip-ft}
\end{aligned}
$$

Available Flexural Strength

From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(50.4 \text{ kip-ft})$ $= 45.4 \text{ kip-ft} > 41.3 \text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{50.4 \text{ kip-ft}}{1.67}$ $= 30.2 \text{ kip-ft} > 27.6 \text{ kip-ft}$ **o.k.**

Note that the calculated available strengths are somewhat lower than those in AISC *Manual* Table 3-13 due to the use of the conservative calculation of the effective section modulus. Also, note that per the User Note in AISC *Specification* Section F7.4, lateral-torsional buckling is not applicable to square HSS.

EXAMPLE F.9A PIPE FLEXURAL MEMBER

Given:

Using AISC *Manual* tables, select a Pipe shape with an 8-in. nominal depth for span and uniform dead and live loads as shown in Figure F.9A. There is no deflection limit for this beam. The beam is simply supported and braced at end points only. The Pipe is ASTM A53 Grade B material.

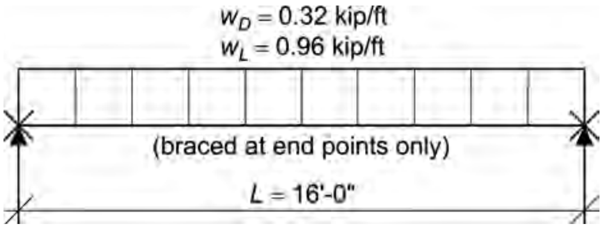


Fig. F.9A. Beam loading and bracing diagram.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A53 Grade B
$F_y = 35$ ksi
$F_u = 60$ ksi

From Chapter 2 of ASCE/SEI 7, the required flexural strength is:

LRFD	ASD
$w_u = 1.2(0.32 \text{ kip/ft}) + 1.6(0.96 \text{ kip/ft})$ $= 1.92 \text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_u = \dfrac{w_u L^2}{8}$ $= \dfrac{(1.92 \text{ kip/ft})(16 \text{ ft})^2}{8}$ $= 61.4 \text{ kip-ft}$	$w_a = 0.32 \text{ kip/ft} + 0.96 \text{ kip/ft}$ $= 1.28 \text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_a = \dfrac{w_a L^2}{8}$ $= \dfrac{(1.28 \text{ kip/ft})(16 \text{ ft})^2}{8}$ $= 41.0 \text{ kip-ft}$

Pipe Selection

Select a member from AISC *Manual* Table 3-15 having the required strength.

Select Pipe 8 x-Strong.

From AISC *Manual* Table 3-15, the available flexural strength is:

LRFD	ASD
$\phi_b M_n = 81.4 \text{ kip-ft} > 61.4 \text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = 54.1 \text{ kip-ft} > 41.0 \text{ kip-ft}$ **o.k.**

EXAMPLE F.9B PIPE FLEXURAL MEMBER

Given:

The available strength in Example F.9A was easily determined using AISC *Manual* Table 3-15. The following example demonstrates the calculation of the available strength by directly applying the AISC *Specification*.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A53 Grade B
$F_y = 35$ ksi
$F_u = 60$ ksi

From AISC *Manual* Table 1-14, the geometric properties are as follows:

Pipe 8 x-Strong
$Z = 31.0 \text{ in.}^3$
$D/t = 18.5$

The required flexural strength from Example F.9A is:

LRFD	ASD
$M_u = 61.4$ kip-ft	$M_a = 41.0$ kip-ft

Slenderness Check

Determine the limiting diameter-to-thickness ratio for a compact section from AISC *Specification* Table B4.1b Case 20.

$$\begin{aligned}\lambda_p &= 0.07\frac{E}{F_y}\\ &= 0.07\left(\frac{29{,}000 \text{ ksi}}{35 \text{ ksi}}\right)\\ &= 58.0\end{aligned}$$

$$\begin{aligned}\lambda &= \frac{D}{t}\\ &= 18.5 < \lambda_p;\ \text{therefore, the section is compact and the limit state of flange local buckling does not apply}\end{aligned}$$

$$\begin{aligned}\frac{0.45E}{F_y} &= \frac{0.45(29{,}000 \text{ ksi})}{35 \text{ ksi}}\\ &= 373 > 18.5;\ \text{therefore, AISC } \textit{Specification} \text{ Section F8 applies}\end{aligned}$$

Nominal Flexural Strength

Based on the limit state of yielding given in AISC *Specification* Section F8.1:

$$M_n = M_p = F_y Z \qquad \text{(}\textit{Spec}\text{. Eq. F8-1)}$$
$$= (35\text{ ksi})(31.0\text{ in.}^3)$$
$$= 1{,}090\text{ kip-in. or }90.4\text{ kip-ft}$$

Available Flexural Strength

From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(90.4\text{ kip-ft})$ $= 81.4\text{ kip-ft} > 61.4\text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{90.4\text{ kip-ft}}{1.67}$ $= 54.1\text{ kip-ft} > 41.0\text{ kip-ft}$ **o.k.**

EXAMPLE F.10 WT-SHAPE FLEXURAL MEMBER

Given:

Directly applying the requirements of the AISC *Specification*, select a WT beam with a 5-in. nominal depth for span and uniform dead and live loads as shown in Figure F.10. The toe of the stem of the WT is in tension. There is no deflection limit for this member. The beam is simply supported and continuously braced. The WT is ASTM A992 material.

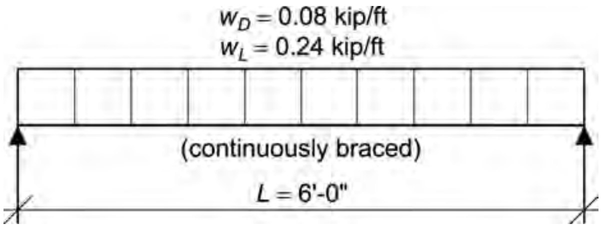


Fig. F.10. Beam loading and bracing diagram.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From Chapter 2 of ASCE/SEI 7, the required flexural strength is:

LRFD	ASD
$w_u = 1.2(0.08 \text{ kip/ft}) + 1.6(0.24 \text{ kip/ft})$ $= 0.480 \text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_u = \dfrac{w_u L^2}{8}$ $= \dfrac{(0.480 \text{ kip/ft})(6 \text{ ft})^2}{8}$ $= 2.16 \text{ kip-ft}$	$w_a = 0.08 \text{ kip/ft} + 0.24 \text{ kip/ft}$ $= 0.320 \text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_a = \dfrac{w_a L^2}{8}$ $= \dfrac{(0.320 \text{ kip/ft})(6 \text{ ft})^2}{8}$ $= 1.44 \text{ kip-ft}$

Try a WT5×6.

From AISC *Manual* Table 1-8, the geometric properties are as follows:

WT5×6
d = 4.94 in.
I_x = 4.35 in.^{4}
Z_x = 2.20 in.^{3}
S_x = 1.22 in.^{3}
b_f = 3.96 in.
t_f = 0.210 in.
$\bar{y}$ = 1.36 in.

$b_f/2t_f = 9.43$

$$S_{xc} = \frac{I_x}{\bar{y}}$$
$$= \frac{4.35\text{ in.}^4}{1.36\text{ in.}}$$
$$= 3.20\text{ in.}^3$$

Nominal Flexural Strength

Yielding

From AISC *Specification* Section F9.1, for the limit state of yielding:

$$M_n = M_p \qquad \text{(}\textit{Spec}\text{. Eq. F9-1)}$$

$$M_y = F_y S_x \qquad \text{(}\textit{Spec}\text{. Eq. F9-3)}$$
$$= (50\text{ ksi})(1.22\text{ in.}^3)$$
$$= 61.0\text{ kip-in.}$$

$$M_p = F_y Z_x \le 1.6M_y \text{ (for stems in tension)} \qquad \text{(}\textit{Spec}\text{. Eq. F9-2)}$$
$$= (50\text{ ksi})(2.20\text{ in.}^3) \le 1.6(61.0\text{ kip-in.})$$
$$= 110\text{ kip-in.} > 97.6\text{ kip-in.}$$
$$= 97.6\text{ kip-in. or } 8.13\text{ kip-ft}$$

Lateral-Torsional Buckling

From AISC *Specification* Section F9.2, because the WT is continuously braced, the limit state of lateral-torsional buckling does not apply.

Flange Local Buckling

The limit state of flange local buckling is checked using AISC *Specification* Section F9.3.

Flange Slenderness

$$\lambda = \frac{b_f}{2t_f}$$
$$= 9.43$$

From AISC *Specification* Table B4.1b, Case 10, the limiting width-to-thickness ratio for the flange is:

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}}$$
$$= 0.38\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}$$
$$= 9.15$$

$$\begin{aligned}\lambda_{rf} &= 1.0\sqrt{\frac{E}{F_y}}\\ &= 1.0\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}\\ &= 24.1\end{aligned}$$

Because $\lambda_{pf} < \lambda < \lambda_{rf}$, the flange is noncompact and the limit state of flange local buckling will apply.

From AISC *Specification* Section F9.3, the nominal flexural strength of a tee with a noncompact flange is:

$$M_n = \left[M_p - \left(M_p - 0.7F_yS_{xc}\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \le 1.6M_y \qquad (\textit{Spec}.\text{ Eq. F9-14})$$

$$\begin{aligned}&= \left\{110\text{ kip-in.} - \left[110\text{ kip-in.} - 0.7\left(50\text{ ksi}\right)\left(3.20\text{ in.}^3\right)\right]\left(\frac{9.43-9.15}{24.1-9.15}\right)\right\} \le 97.6\text{ kip-in.}\\ &= 110\text{ kip-in.} > 97.6\text{ kip-in.}\\ &= 97.6\text{ kip-in.}\end{aligned}$$

Flexural yielding controls:

$$M_n = 97.6\text{ kip-in. or }8.13\text{ kip-ft}$$

Available Flexural Strength

From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90\left(8.13\text{ kip-ft}\right)$ $= 7.32\text{ kip-ft} > 2.16\text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{8.13\text{ kip-ft}}{1.67}$ $= 4.87\text{ kip-ft} > 1.44\text{ kip-ft}$ **o.k.**

EXAMPLE F.11A SINGLE-ANGLE FLEXURAL MEMBER WITH BRACING AT ENDS ONLY

Given:

Directly applying the requirements of the AISC *Specification*, select a single angle for span and uniform dead and live loads as shown in Figure F.11A. The vertical leg of the single angle is up and the toe is in compression. There are no horizontal loads. There is no deflection limit for this angle. The beam is simply supported and braced at the end points only. Assume bending about the geometric *x*-*x* axis and that there is no lateral-torsional restraint. The angle is ASTM A36 material.

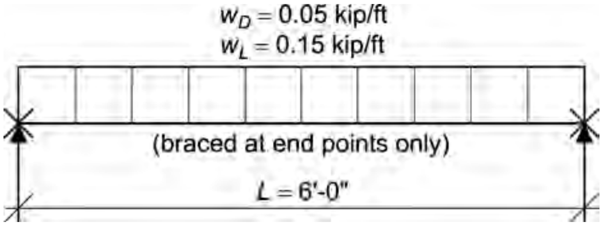


Fig. F.11A. Beam loading and bracing diagram.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From Chapter 2 of ASCE/SEI 7, the required flexural strength is:

LRFD	ASD
$w_{ux} = 1.2(0.05\text{ kip/ft}) + 1.6(0.15\text{ kip/ft})$ $= 0.300\text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_{ux} = \dfrac{w_{ux}L^2}{8}$ $= \dfrac{(0.300\text{ kip/ft})(6\text{ ft})^2}{8}$ $= 1.35\text{ kip-ft}$	$w_{ax} = 0.05\text{ kip/ft} + 0.15\text{ kip/ft}$ $= 0.200\text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_{ax} = \dfrac{w_{ax}L^2}{8}$ $= \dfrac{(0.200\text{ kip/ft})(6\text{ ft})^2}{8}$ $= 0.900\text{ kip-ft}$

Try a L4×4×¼.

From AISC *Manual* Table 1-7, the geometric properties are as follows:

L4×4×¼
S_x = 1.03 in.^{3}

Nominal Flexural Strength

Yielding

From AISC *Specification* Section F10.1, the nominal flexural strength due to the limit state of flexural yielding is:

$$\begin{aligned}M_n &= 1.5M_y \qquad \text{(}\textit{Spec}\text{. Eq. F10-1)}\\ &= 1.5F_yS_x\\ &= 1.5(36\text{ ksi})(1.03\text{ in.}^3)\\ &= 55.6\text{ kip-in.}\end{aligned}$$

Lateral-Torsional Buckling

From AISC *Specification* Section F10.2, for single angles bending about a geometric axis with no lateral-torsional restraint, M_y is taken as 0.80 times the yield moment calculated using the geometric section modulus.

$$\begin{aligned}M_y &= 0.80F_yS_x\\ &= 0.80(36\text{ ksi})(1.03\text{ in.}^3)\\ &= 29.7\text{ kip-in.}\end{aligned}$$

Determine M_{cr}.

For bending moment about one of the geometric axes of an equal-leg angle with no axial compression, with no lateral-torsional restraint, and with maximum compression at the toe, use AISC *Specification* Equation F10-5a.

C_b = 1.14 from AISC *Manual* Table 3-1

$$M_{cr} = \frac{0.58Eb^4tC_b}{L_b^2}\left(\sqrt{1+0.88\left(\frac{L_bt}{b^2}\right)^2}-1\right) \qquad \text{(}\textit{Spec}\text{. Eq. F10-5a)}$$

$$\begin{aligned}&= \frac{0.58(29{,}000\text{ ksi})(4.00\text{ in.})^4(¼\text{ in.})(1.14)}{\left[(6\text{ ft})(12\text{ in./ft})\right]^2}\left\{\sqrt{1+0.88\left[\frac{(6\text{ ft})(12\text{ in./ft})(¼\text{ in.})}{(4.00\text{ in.})^2}\right]^2}-1\right\}\\ &= 107\text{ kip-in.}\end{aligned}$$

$$\begin{aligned}\frac{M_y}{M_{cr}} &= \frac{29.7\text{ kip-in.}}{107\text{ kip-in.}};\\ &= 0.278 < 1.0;\text{ therefore, AISC }\textit{Specification}\text{ Equation F10-2 is applicable}\end{aligned}$$

$$M_n = \left(1.92-1.17\sqrt{\frac{M_y}{M_{cr}}}\right)M_y \le 1.5M_y \qquad \text{(}\textit{Spec}\text{. Eq. F10-2)}$$

$$\begin{aligned}&= \left(1.92-1.17\sqrt{\frac{29.7\text{ kip-in.}}{107\text{ kip-in.}}}\right)(29.7\text{ kip-in.}) \le 1.5(29.7\text{ kip-in.})\\ &= 38.7\text{ kip-in.} < 44.6\text{ kip-in.}\\ &= 38.7\text{ kip-in.}\end{aligned}$$

Leg Local Buckling

AISC *Specification* Section F10.3 applies when the toe of the leg is in compression.

Check slenderness of the leg in compression.

$$\begin{aligned}\lambda &= \frac{b}{t} \\ &= \frac{4.00\text{ in.}}{\text{¼ in.}} \\ &= 16.0\end{aligned}$$

Determine the limiting compact slenderness ratios from AISC *Specification* Table B4.1b, Case 12.

$$\begin{aligned}\lambda_p &= 0.54\sqrt{\frac{E}{F_y}} \\ &= 0.54\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}} \\ &= 15.3\end{aligned}$$

Determine the limiting noncompact slenderness ratios from AISC *Specification* Table B4.1b, Case 12.

$$\begin{aligned}\lambda_r &= 0.91\sqrt{\frac{E}{F_y}} \\ &= 0.91\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}} \\ &= 25.8\end{aligned}$$

$\lambda_p < \lambda < \lambda_r$, therefore, the leg is noncompact in flexure

$$\begin{aligned}S_c &= 0.80S_x \\ &= 0.80\left(1.03\text{in.}^3\right) \\ &= 0.824\text{ in.}^3\end{aligned}$$

$$M_n = F_y S_c\left[2.43 - 1.72\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}}\right] \qquad (\textit{Spec}.\text{ Eq. F10-6})$$

$$\begin{aligned}&= \left(36\text{ ksi}\right)\left(0.824\text{ in.}^3\right)\left[2.43 - 1.72\left(16.0\right)\sqrt{\frac{36\text{ ksi}}{29{,}000\text{ ksi}}}\right] \\ &= 43.3\text{ kip-in.}\end{aligned}$$

The lateral-torsional buckling limit state controls.

M_n = 38.7 kip-in. or 3.23 kip-ft

Available Flexural Strength

From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(3.23\text{ kip-ft})$ $= 2.91\text{ kip-ft} > 1.35\text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{3.23\text{ kip-ft}}{1.67}$ $= 1.93\text{ kip-ft} > 0.900\text{ kip-ft}$ **o.k.**

EXAMPLE F.11B SINGLE-ANGLE FLEXURAL MEMBER WITH BRACING AT ENDS AND MIDSPAN

Given:

Directly applying the requirements of the AISC *Specification*, select a single angle for span and uniform dead and live loads as shown in Figure F.11B. The vertical leg of the single angle is up and the toe is in compression. There are no horizontal loads. There is no deflection limit for this angle. The beam is simply supported and braced at the end points and midspan. Assume bending about the geometric *x*-*x* axis and that there is lateral-torsional restraint at the midspan and ends only. The angle is ASTM A36 material.

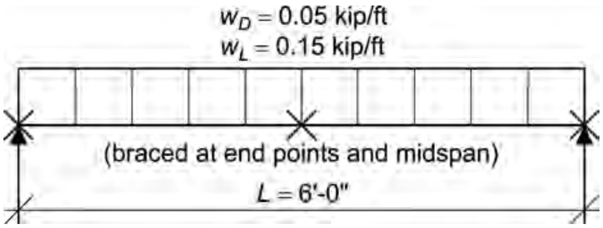


Fig. F.11B. Beam loading and bracing diagram.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From Chapter 2 of ASCE/SEI 7, the required flexural strength is:

LRFD	ASD
$w_{ux} = 1.2(0.05\text{ kip/ft}) + 1.6(0.15\text{ kip/ft})$ $= 0.300\text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_{ux} = \dfrac{w_{ux}L^2}{8}$ $= \dfrac{(0.300\text{ kip/ft})(6\text{ ft})^2}{8}$ $= 1.35\text{ kip-ft}$	$w_{ax} = 0.05\text{ kip/ft} + 0.15\text{ kip/ft}$ $= 0.200\text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_{ax} = \dfrac{w_{ax}L^2}{8}$ $= \dfrac{(0.200\text{ kip/ft})(6\text{ ft})^2}{8}$ $= 0.900\text{ kip-ft}$

Try a L4×4×¼.

From AISC *Manual* Table 1-7, the geometric properties are as follows:

L4×4×¼
S_x = 1.03 in.^{3}

Nominal Flexural Strength

Flexural Yielding

From AISC *Specification* Section F10.1, the nominal flexural strength due to the limit state of flexural yielding is:

$$\begin{aligned} M_n &= 1.5M_y \\ &= 1.5F_yS_x \\ &= 1.5(36\text{ ksi})(1.03\text{ in.}^3) \\ &= 55.6\text{ kip-in.} \end{aligned}$$

(*Spec*. Eq. F10-1)

Lateral-Torsional Buckling

From AISC *Specification* Section F10.2(b)(2)(ii), for single angles with lateral-torsional restraint at the point of maximum moment, M_y is taken as the yield moment calculated using the geometric section modulus.

$$\begin{aligned} M_y &= F_yS_x \\ &= (36\text{ ksi})(1.03\text{ in.}^3) \\ &= 37.1\text{ kip-in.} \end{aligned}$$

Determine M_{cr}.

For bending moment about one of the geometric axes of an equal-leg angle with no axial compression, with lateral-torsional restraint at the point of maximum moment only (at midspan in this case), and with maximum compression at the toe, M_{cr} shall be taken as 1.25 times M_{cr} computed using AISC *Specification* Equation F10-5a.

C_b = 1.30 from AISC *Manual* Table 3-1

$$M_{cr} = 1.25\left(\frac{0.58Eb^4tC_b}{L_b^{\,2}}\right)\left(\sqrt{1+0.88\left(\frac{L_bt}{b^2}\right)^2}-1\right)$$

(from *Spec*. Eq. F10-5a)

$$\begin{aligned} &= 1.25\left[\frac{0.58(29{,}000\text{ ksi})(4.00\text{ in.})^4(\tfrac{1}{4}\text{ in.})(1.30)}{\left[(3\text{ ft})(12\text{ in./ft})\right]^2}\right]\left\{\sqrt{1+0.88\left[\frac{(3\text{ ft})(12\text{ in./ft})(\tfrac{1}{4}\text{ in.})}{(4.00\text{ in.})^2}\right]^2}-1\right\} \\ &= 176\text{ kip-in.} \end{aligned}$$

$$\begin{aligned} \frac{M_y}{M_{cr}} &= \frac{37.1\text{ kip-in.}}{176\text{ kip-in.}} \\ &= 0.211 < 1.0;\ \text{therefore, AISC } \textit{Specification} \text{ Equation F10-2 is applicable} \end{aligned}$$

$$M_n = \left(1.92-1.17\sqrt{\frac{M_y}{M_{cr}}}\right)M_y \le 1.5M_y$$

(*Spec*. Eq. F10-2)

$$\begin{aligned} &= \left(1.92-1.17\sqrt{\frac{37.1\text{ kip-in.}}{176\text{ kip-in.}}}\right)(37.1\text{ kip-in.}) \le 1.5(37.1\text{ kip-in.}) \\ &= 51.3\text{ kip-in.} < 55.7\text{ kip-in.} \\ &= 51.3\text{ kip-in.} \end{aligned}$$

Leg Local Buckling

M_n = 43.3 kip-in. from Example F.11A.

The leg local buckling limit state controls.

M_n = 43.3 kip-in. or 3.61 kip-ft

Available Flexural Strength

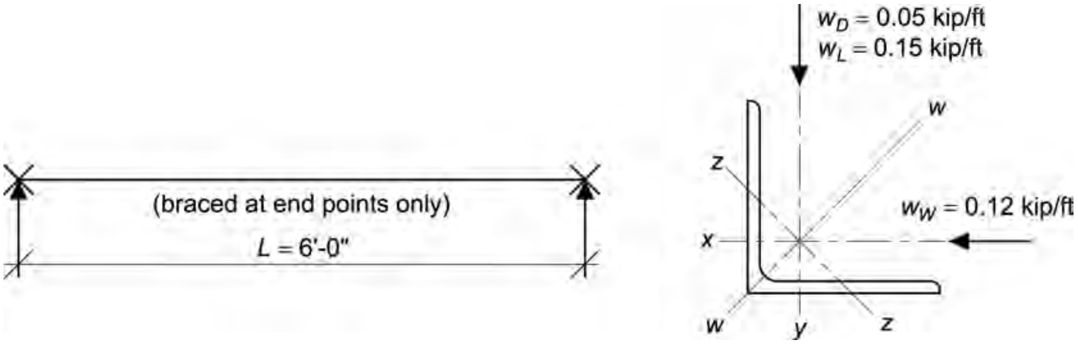
From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(3.61\text{ kip-ft})$ $= 3.25\text{ kip-ft} > 1.35\text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{3.61\text{ kip-ft}}{1.67}$ $= 2.16\text{ kip-ft} > 0.900\text{ kip-ft}$ **o.k.**

EXAMPLE F.11C SINGLE-ANGLE FLEXURAL MEMBER WITH VERTICAL AND HORIZONTAL LOADING

Given:

Directly applying the requirements of the AISC *Specification*, select a single angle for span and uniform vertical dead and live loads as shown in Figure F.11C-1. The horizontal load is a uniform wind load. There is no deflection limit for this angle. The angle is simply supported and braced at the end points only and there is no lateral-torsional restraint. Use load combination 4 from Section 2.3.1 of ASCE/SEI 7 for LRFD and load combination 6 from Section 2.4.1 of ASCE/SEI 7 for ASD. The angle is ASTM A36 material.



(a) Beam bracing diagram *(b) Beam loading*

Fig. F.11C-1. Beam loading and bracing diagram.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From Chapter 2 of ASCE/SEI 7, the required flexural strength is:

LRFD	ASD
$w_{ux} = 1.2(0.05\text{ kip/ft}) + 0.15\text{ kip/ft}$ $= 0.210\text{ kip/ft}$	$w_{ax} = 0.05\text{ kip/ft} + 0.75(0.15\text{ kip/ft})$ $= 0.163\text{ kip/ft}$
$w_{uy} = 1.0(0.12\text{ kip/ft})$ $= 0.120\text{ kip/ft}$	$w_{ay} = 0.75\left[0.6(0.12\text{ kip/ft})\right]$ $= 0.0540\text{ kip/ft}$
$M_{ux} = \dfrac{w_{ux}L^2}{8}$ $= \dfrac{(0.210\text{ kip/ft})(6\text{ ft})^2}{8}$ $= 0.945\text{ kip-ft}$	$M_{ax} = \dfrac{w_{ax}L^2}{8}$ $= \dfrac{(0.163\text{ kip/ft})(6\text{ ft})^2}{8}$ $= 0.734\text{ kip-ft}$

LRFD	ASD
$M_{uy} = \dfrac{w_{uy}L^2}{8}$ $= \dfrac{(0.120\text{ kip/ft})(6\text{ ft})^2}{8}$ $= 0.540\text{ kip-ft}$	$M_{ay} = \dfrac{w_{ay}L^2}{8}$ $= \dfrac{(0.0540\text{ kip/ft})(6\text{ ft})^2}{8}$ $= 0.243\text{ kip-ft}$

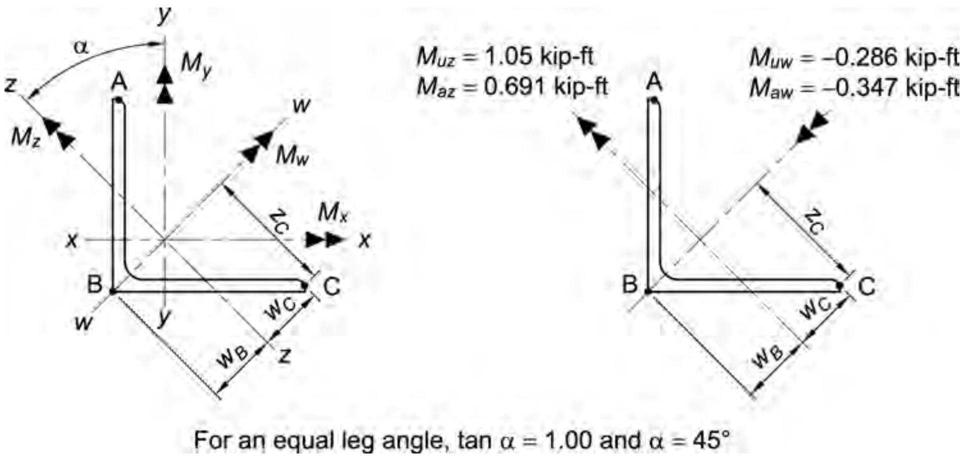
Try a L4×4×¼.

Sign convention for geometric axes moments are:

LRFD	ASD
$M_{ux} = -0.945\text{ kip-ft}$ $M_{uy} = 0.540\text{ kip-ft}$	$M_{ax} = -0.734\text{ kip-ft}$ $M_{ay} = 0.243\text{ kip-ft}$

As shown in Figure F.11C-2, the principal axes moments are:

LRFD	ASD
$M_{uw} = M_{ux}\cos\alpha + M_{uy}\sin\alpha$ $= (-0.945\text{ kip-ft})(\cos 45°)$ $+ (0.540\text{ kip-ft})(\sin 45°)$ $= -0.286\text{ kip-ft}$ $M_{uz} = -M_{ux}\sin\alpha + M_{uy}\cos\alpha$ $= -(-0.945\text{ kip-ft})(\sin 45°)$ $+ (0.540\text{ kip-ft})(\cos 45°)$ $= 1.05\text{ kip-ft}$	$M_{aw} = M_{ax}\cos\alpha + M_{ay}\sin\alpha$ $= (-0.734\text{ kip-ft})(\cos 45°)$ $+ (0.243\text{ kip-ft})(\sin 45°)$ $= -0.347\text{ kip-ft}$ $M_{az} = -M_{ax}\sin\alpha + M_{ay}\cos\alpha$ $= -(-0.734\text{ kip-ft})(\sin 45°)$ $+ (0.243\text{ kip-ft})(\cos 45°)$ $= 0.691\text{ kip-ft}$



For an equal leg angle, tan α = 1.00 and α = 45°

(a) Positive geometric and principal axes *(b) Principal axis moments*

Fig. F.11C-2. Example F.11C single angle geometric and principal axes moments.

From AISC *Manual* Table 1-7, the geometric properties are as follows:

L4×4×¼
$A = 1.93\text{ in.}^2$
$S_x = S_y = 1.03\text{ in.}^3$
$I_x = I_y = 3.00\text{ in.}^4$
$I_z = 1.19\text{ in.}^4$
$r_z = 0.783\text{ in.}$

Additional principal axes properties from the AISC *Shapes Database* are as follows:

$w_B = 1.53\text{ in.}$
$w_C = 1.39\text{ in.}$
$z_C = 2.74\text{ in.}$
$I_w = 4.82\text{ in.}^4$
$S_{zB} = 0.778\text{ in.}^3$
$S_{zC} = 0.856\text{ in.}^3$
$S_{wC} = 1.76\text{ in.}^3$

Z-Axis Nominal Flexural Strength

Note that M_{uz} and M_{az} are positive; therefore, the toes of the angle are in compression.

Flexural Yielding

From AISC *Specification* Section F10.1, the nominal flexural strength due to the limit state of flexural yielding is:

$$\begin{aligned} M_{nz} &= 1.5M_y \\ &= 1.5F_yS_{zB} \\ &= 1.5(36\text{ ksi})\left(0.778\text{ in.}^3\right) \\ &= 42.0\text{ kip-in.} \end{aligned}$$

(from *Spec*. Eq. F10-1)

Lateral-Torsional Buckling

From the User Note in AISC *Specification* Section F10, the limit state of lateral-torsional buckling does not apply for bending about the minor axis.

Leg Local Buckling

Check slenderness of outstanding leg in compression.

$$\begin{aligned} \lambda &= \frac{b}{t} \\ &= \frac{4.00\text{ in.}}{¼\text{ in.}} \\ &= 16.0 \end{aligned}$$

From AISC *Specification* Table B4.1b, Case 12, the limiting width-to-thickness ratios are:

$$\lambda_p = 0.54\sqrt{\frac{E}{F_y}}$$
$$= 0.54\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}}$$
$$= 15.3$$

$$\lambda_r = 0.91\sqrt{\frac{E}{F_y}}$$
$$= 0.91\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}}$$
$$= 25.8$$

Because $\lambda_p < \lambda < \lambda_r$, the leg is noncompact in flexure.

$S_c = S_{zC}$ (to toe in compression)
$= 0.856\text{ in.}^3$

$$M_{nz} = F_y S_c\left[2.43 - 1.72\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}}\right] \qquad (\textit{Spec}.\text{ Eq. F10-6})$$
$$= (36\text{ ksi})(0.856\text{ in.}^3)\left[2.43 - 1.72(16.0)\sqrt{\frac{36\text{ ksi}}{29{,}000\text{ ksi}}}\right]$$
$$= 45.0\text{ kip-in.}$$

The flexural yielding limit state controls.

$M_{nz} = 42.0$ kip-in. or 3.50 kip-ft

Z-Axis Available Flexural Strength

From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_{nz} = 0.90(3.50\text{ kip-ft})$ $= 3.15\text{ kip-ft}$	$\Omega_b = 1.67$ $\frac{M_{nz}}{\Omega_b} = \frac{3.50\text{ kip-ft}}{1.67}$ $= 2.10\text{ kip-ft}$

W-Axis Nominal Flexural Strength

Flexural Yielding

$$M_{nw} = 1.5M_y \qquad (\text{from }\textit{Spec}.\text{ Eq. F10-1})$$
$$= 1.5F_y S_{wC}$$
$$= 1.5(36\text{ ksi})(1.76\text{ in.}^3)$$
$$= 95.0\text{ kip-in.}$$

Lateral-Torsional Buckling

Determine M_{cr}.

For bending about the major principal axis of an equal-leg angle without continuous lateral-torsional restraint, use AISC *Specification* Equation F10-4.

C_b = 1.14 from *Manual* Table 3-1

From AISC *Specification* Section F10.2(b)(1), $\beta_w = 0$ for equal leg angles.

$$M_{cr} = \frac{9EAr_z tC_b}{8L_b}\left[\sqrt{1+\left(4.4\frac{\beta_w r_z}{L_b t}\right)^2}+4.4\frac{\beta_w r_z}{L_b t}\right] \qquad (\textit{Spec}.\text{ Eq. F10-4})$$

$$\begin{aligned}
&= \frac{9(29{,}000\text{ ksi})(1.93\text{ in.}^2)(0.783\text{ in.})(\tfrac{1}{4}\text{ in.})(1.14)}{8(6\text{ ft})(12\text{ in./ft})} \\
&\quad\times\left\{\sqrt{1+\left[4.4\frac{0(0.783\text{ in.})}{(6\text{ ft})(12\text{ in./ft})(\tfrac{1}{4}\text{ in.})}\right]^2}+4.4\left[\frac{0(0.783\text{ in.})}{(6\text{ ft})(12\text{ in./ft})(\tfrac{1}{4}\text{ in.})}\right]\right\} \\
&= 195\text{ kip-in.}
\end{aligned}$$

$$\begin{aligned}
M_y &= F_y S_{wC} \\
&= (36\text{ ksi})(1.76\text{ in.}^3) \\
&= 63.4\text{ kip-in.}
\end{aligned}$$

$$\begin{aligned}
\frac{M_y}{M_{cr}} &= \frac{63.4\text{ kip-in.}}{195\text{ kip-in.}} \\
&= 0.325 < 1.0, \text{ therefore, AISC } \textit{Specification} \text{ Equation F10-2 is applicable}
\end{aligned}$$

$$M_{nw} = \left(1.92-1.17\sqrt{\frac{M_y}{M_{cr}}}\right)M_y \le 1.5M_y \qquad (\textit{Spec}.\text{ Eq. F10-2})$$

$$\begin{aligned}
&= \left(1.92-1.17\sqrt{\frac{63.4\text{ kip-in.}}{195\text{ kip-in.}}}\right)(63.4\text{ kip-in.}) \le 1.5(63.4\text{ kip-in.}) \\
&= 79.4\text{ kip-in.} < 95.1\text{ kip-in.} \\
&= 79.4\text{ kip-in.}
\end{aligned}$$

Leg Local Buckling

From the preceding calculations, the leg is noncompact in flexure.

$$\begin{aligned}
S_c &= S_{wC} \text{ (to toe in compression)} \\
&= 1.76\text{ in.}^3
\end{aligned}$$

$$M_{nw} = F_y S_c \left[2.43 - 1.72 \left(\frac{b}{t} \right) \sqrt{\frac{F_y}{E}} \right] \qquad (Spec.\ \text{Eq. F10-6})$$

$$= (36 \text{ ksi})(1.76 \text{ in.}^3) \left[2.43 - 1.72(16.0) \sqrt{\frac{36 \text{ ksi}}{29{,}000 \text{ ksi}}} \right]$$

$$= 92.5 \text{ kip-in.}$$

The lateral-torsional buckling limit state controls.

M_{nw} = 79.4 kip-in. or 6.62 kip-ft

W-Axis Available Flexural Strength

From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_{nw} = 0.90(6.62 \text{ kip-ft})$ $= 5.96 \text{ kip-ft}$	$\Omega_b = 1.67$ $\frac{M_{nw}}{\Omega_b} = \frac{6.62 \text{ kip-ft}}{1.67}$ $= 3.96 \text{ kip-ft}$

Combined Loading

The moment resultant has components about both principal axes; therefore, the combined stress ratio must be checked using the provisions of AISC *Specification* Section H2.

$$\left| \frac{f_{ra}}{F_{ca}} + \frac{f_{rbw}}{F_{cbw}} + \frac{f_{rbz}}{F_{cbz}} \right| \le 1.0 \qquad (Spec.\ \text{Eq. H2-1})$$

Note: Rather than convert moments into stresses, it is acceptable to simply use the moments in the interaction equation because the section properties that would be used to convert the moments to stresses are the same in the numerator and denominator of each term. It is also important for the designer to keep track of the signs of the stresses at each point so that the proper sign is applied when the terms are combined. The sign of the moments used to convert geometric axis moments to principal axis moments will indicate which points are in tension and which are in compression but those signs will not be used in the interaction equations directly.

Based on Figure F.11C-2, the required flexural strength and available flexural strength for this beam can be summarized as:

LRFD	ASD
$M_{uw} = 0.286 \text{ kip-ft}$	$M_{aw} = 0.347 \text{ kip-ft}$
$\phi_b M_{nw} = 5.96 \text{ kip-ft}$	$\frac{M_{nw}}{\Omega_b} = 3.96 \text{ kip-ft}$
$M_{uz} = 1.05 \text{ kip-ft}$	$M_{az} = 0.691 \text{ kip-ft}$
$\phi_b M_{nz} = 3.15 \text{ kip-ft}$	$\frac{M_{nz}}{\Omega_b} = 2.10 \text{ kip-ft}$

At point B:

M_w causes no stress at point B; therefore, the stress ratio is set to zero. M_z causes tension at point B; therefore it will be taken as negative.

LRFD	ASD
$\left\|0-\dfrac{1.05\text{ kip-ft}}{3.15\text{ kip-ft}}\right\|=0.333\le 1.0$ **o.k.**	$\left\|0-\dfrac{0.691\text{ kip-ft}}{2.10\text{ kip-ft}}\right\|=0.329\le 1.0$ **o.k.**

At point C:

M_w causes tension at point C; therefore, it will be taken as negative. M_z causes compression at point C; therefore, it will be taken as positive.

LRFD	ASD
$\left\|-\dfrac{0.286\text{ kip-ft}}{5.96\text{ kip-ft}}+\dfrac{1.05\text{ kip-ft}}{3.15\text{ kip-ft}}\right\|=0.285\le 1.0$ **o.k.**	$\left\|-\dfrac{0.347\text{ kip-ft}}{3.96\text{ kip-ft}}+\dfrac{0.691\text{ kip-ft}}{2.10\text{ kip-ft}}\right\|=0.241\le 1.0$ **o.k.**

At point A:

M_w and M_z cause compression at point A; therefore, both will be taken as positive.

LRFD	ASD
$\left\|\dfrac{0.286\text{ kip-ft}}{5.96\text{ kip-ft}}+\dfrac{1.05\text{ kip-ft}}{3.15\text{ kip-ft}}\right\|=0.381\le 1.0$ **o.k.**	$\left\|\dfrac{0.347\text{ kip-ft}}{3.96\text{ kip-ft}}+\dfrac{0.691\text{ kip-ft}}{2.10\text{ kip-ft}}\right\|=0.417\le 1.0$ **o.k.**

Thus, the interaction of stresses at each point is seen to be less than 1.0 and this member is adequate to carry the required load. Although all three points were checked, it was expected that point A would be the controlling point because compressive stresses add at this point.

EXAMPLE F.12 RECTANGULAR BAR IN MAJOR AXIS BENDING

Given:

Directly applying the requirements of the AISC *Specification*, select a rectangular bar for span and uniform vertical dead and live loads as shown in Figure F.12. The beam is simply supported and braced at the end points and midspan. Conservatively use C_b = 1.0. Limit the depth of the member to 5 in. The bar is ASTM A36 material.

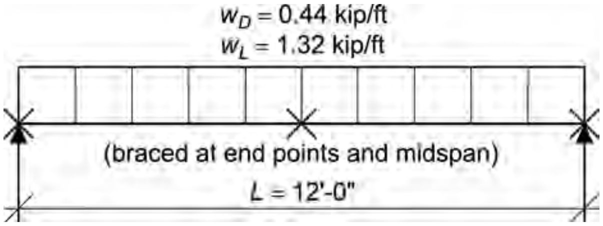


Fig. F.12. Beam loading and bracing diagram.

Solution:

From AISC *Manual* Table 2-5, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From Chapter 2 of ASCE/SEI 7, the required flexural strength is:

LRFD	ASD
$w_u = 1.2(0.44 \text{ kip/ft}) + 1.6(1.32 \text{ kip/ft})$ $= 2.64 \text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_u = \dfrac{w_u L^2}{8}$ $= \dfrac{(2.64 \text{ kip/ft})(12 \text{ ft})^2}{8}$ $= 47.5 \text{ kip-ft}$	$w_a = 0.44 \text{ kip/ft} + 1.32 \text{ kip/ft}$ $= 1.76 \text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_a = \dfrac{w_a L^2}{8}$ $= \dfrac{(1.76 \text{ kip/ft})(12 \text{ ft})^2}{8}$ $= 31.7 \text{ kip-ft}$

Try a BAR 5 in. × 3 in.

From AISC *Manual* Table 17-27, the geometric properties are as follows:

$$\begin{aligned}S_x &= \frac{bd^2}{6}\\ &= \frac{(3.00 \text{ in.})(5.00 \text{ in.})^2}{6}\\ &= 12.5 \text{ in.}^3\end{aligned}$$

$$
\begin{aligned}
Z_x &= \frac{bd^2}{4} \\
&= \frac{(3.00\text{ in.})(5.00\text{ in.})^2}{4} \\
&= 18.8\text{ in.}^3
\end{aligned}
$$

Nominal Flexural Strength

Flexural Yielding

Check limit from AISC *Specification* Section F11.1.

$$
\begin{aligned}
\frac{L_b d}{t^2} &= \frac{(6\text{ ft})(12\text{ in./ft})(5.00\text{ in.})}{(3.00\text{ in.})^2} \\
&= 40.0
\end{aligned}
$$

$$
\begin{aligned}
\frac{0.08E}{F_y} &= \frac{0.08(29{,}000\text{ ksi})}{36\text{ ksi}} \\
&= 64.4 > 40.0;\ \text{therefore, the yielding limit state applies}
\end{aligned}
$$

$$M_n = M_p = F_y Z \le 1.6 F_y S$$ (*Spec*. Eq. F11-1)

$$
\begin{aligned}
1.6F_y S &= 1.6F_y S_x \\
&= 1.6(36\text{ ksi})\left(12.5\text{ in.}^3\right) \\
&= 720\text{ kip-in.}
\end{aligned}
$$

$$
\begin{aligned}
F_y Z &= F_y Z_x \\
&= (36\text{ ksi})\left(18.8\text{ in.}^3\right) \\
&= 677\text{ kip-in.} < 720\text{ kip-in.}
\end{aligned}
$$

Use M_n = 677 kip-in. or 56.4 kip-ft.

Lateral-Torsional Buckling

From AISC *Specification* Section F11.2(a), because $\frac{L_b d}{t^2} \le \frac{0.08E}{F_y}$, the lateral-torsional buckling limit state does not apply.

Available Flexural Strength

From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(56.4 \text{ kip-ft})$ $= 50.8 \text{ kip-ft} > 47.5 \text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{56.4 \text{ kip-ft}}{1.67}$ $= 33.8 \text{ kip-ft} > 31.7 \text{ kip-ft}$ **o.k.**

EXAMPLE F.13 ROUND BAR IN BENDING

Given:

Select a round bar for span and concentrated dead and live loads, at midspan, as shown in Figure F.13. The beam is simply supported and braced at the end points only. Conservatively use C_b = 1.0. Limit the diameter of the member to 2 in. The weight of the bar is negligible. The bar is ASTM A36 material.

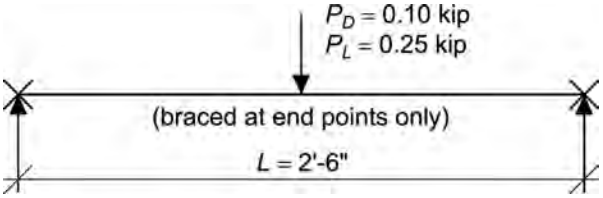


Fig. F.13. Beam loading and bracing diagram.

Solution:

From AISC *Manual* Table 2-5, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From Chapter 2 of ASCE/SEI 7 the required flexural strength is:

LRFD	ASD
$P_u = 1.2(0.10\text{ kip}) + 1.6(0.25\text{ kip})$ $= 0.520\text{ kip}$ From AISC *Manual* Table 3-23, Case 7: $M_u = \dfrac{P_u L}{4}$ $= \dfrac{(0.520\text{ kip})(2.5\text{ ft})}{4}$ $= 0.325\text{ kip-ft}$	$P_a = 0.10\text{ kip} + 0.25\text{ kip}$ $= 0.350\text{ kip}$ From AISC *Manual* Table 3-23, Case 7: $M_a = \dfrac{P_a L}{4}$ $= \dfrac{(0.350\text{ kip})(2.5\text{ ft})}{4}$ $= 0.219\text{ kip-ft}$

Try a BAR 1-in.-diameter.

From AISC *Manual* Table 17-27, the geometric properties are as follows:

$$S = \frac{\pi d^3}{32}$$
$$= \frac{\pi(1.00\text{ in.})^3}{32}$$
$$= 0.0982\text{ in.}^3$$

$$
\begin{aligned}
Z &= \frac{d^3}{6} \\
&= \frac{(1.00\text{ in.})^3}{6} \\
&= 0.167\text{ in.}^3
\end{aligned}
$$

Nominal Flexural Strength

Flexural Yielding

From AISC *Specification* Section F11.1, the nominal flexural strength based on the limit state of flexural yielding is:

$$M_n = M_p = F_y Z \leq 1.6 F_y S_x \qquad \text{(}\textit{Spec}\text{. Eq. F11-1)}$$

$$
\begin{aligned}
1.6F_y S &= 1.6(36\text{ ksi})\left(0.0982\text{ in.}^3\right) \\
&= 5.66\text{ kip-in.}
\end{aligned}
$$

$$
\begin{aligned}
F_y Z &= (36\text{ ksi})\left(0.167\text{ in.}^3\right) \\
&= 6.01\text{ kip-in.} > 5.66\text{ kip-in, therefore, } M_n = 5.66\text{ kip-in.}
\end{aligned}
$$

From *AISC Specification* Section F11.2, the limit state lateral-torsional buckling need not be considered for rounds.

The flexural yielding limit state controls.

M_n = 5.66 kip-in. or 0.472 kip-ft

Available Flexural Strength

From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(0.472\text{ kip-ft})$ $= 0.425\text{ kip-ft} > 0.325\text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{0.472\text{ kip-ft}}{1.67}$ $= 0.283\text{ kip-ft} > 0.219\text{ kip-ft}$ **o.k.**

EXAMPLE F.14 POINT-SYMMETRICAL Z-SHAPE IN MAJOR AXIS BENDING

Given:

Directly applying the requirements of the AISC *Specification*, determine the available flexural strength of a Z-shaped flexural member for the span and loading shown in Figure F.14-1. The beam is simply supported and braced at the third and end points. Assume C_b = 1.0. Assume the beam is loaded through the shear center. The geometry for the member is shown in Figure F.14-2. The member is ASTM A36 material.

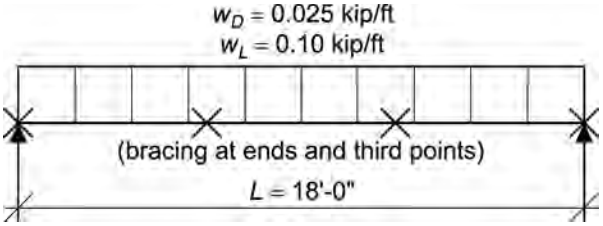


Fig. F.14-1. Beam loading and bracing diagram.

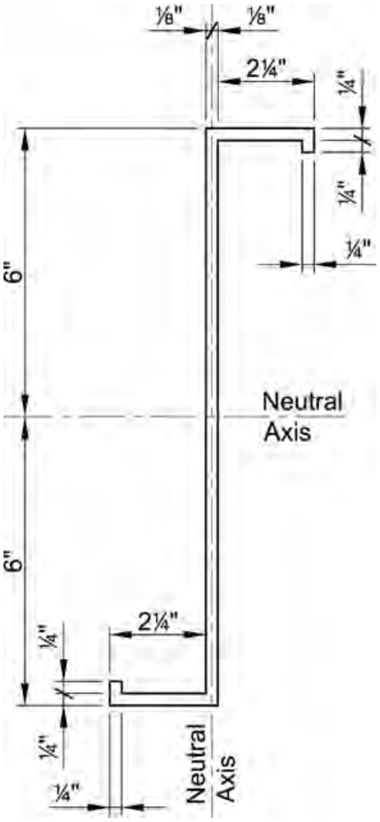


Fig. F.14-2. Beam geometry for Example F.14.

Solution:

From AISC *Manual* Table 2-5, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

The geometric properties are as follows:

$$\begin{aligned} t_w &= t_f \\ &= \tfrac{1}{4}\text{ in.} \end{aligned}$$

$$\begin{aligned} A &= 2(2.50\text{ in.})(\tfrac{1}{4}\text{ in.}) + 2(\tfrac{1}{4}\text{ in.})(\tfrac{1}{4}\text{ in.}) + (11.5\text{ in.})(\tfrac{1}{4}\text{ in.}) \\ &= 4.25\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} I_x &= 2\left[\frac{(\tfrac{1}{4}\text{ in.})(\tfrac{1}{4}\text{ in.})^3}{12} + (\tfrac{1}{4}\text{ in.})^2(5.63\text{ in.})^2\right] + 2\left[\frac{(2.50\text{ in.})(\tfrac{1}{4}\text{ in.})^3}{12} + (2.50\text{ in.})(\tfrac{1}{4}\text{ in.})(5.88\text{ in.})^2\right] \\ &\quad + \frac{(\tfrac{1}{4}\text{ in.})(11.5\text{ in.})^3}{12} \\ &= 78.9\text{ in.}^4 \end{aligned}$$

$$\bar{y} = 6.00\text{ in.}$$

$$\begin{aligned} S_x &= \frac{I_x}{\bar{y}} \\ &= \frac{78.9\text{ in.}^4}{6.00\text{ in.}} \\ &= 13.2\text{ in.}^3 \end{aligned}$$

$$\begin{aligned} I_y &= 2\left[\frac{(\tfrac{1}{4}\text{ in.})(\tfrac{1}{4}\text{ in.})^3}{12} + (\tfrac{1}{4}\text{ in.})^2(2.25\text{ in.})^2\right] + 2\left[\frac{(\tfrac{1}{4}\text{ in.})(2.50\text{ in.})^3}{12} + (2.50\text{ in.})(\tfrac{1}{4}\text{ in.})(1.13\text{ in.})^2\right] \\ &\quad + \frac{(11.5\text{ in.})(\tfrac{1}{4}\text{ in.})^3}{12} \\ &= 2.90\text{ in.}^4 \end{aligned}$$

$$\begin{aligned} r_y &= \sqrt{\frac{I_y}{A}} \\ &= \sqrt{\frac{2.90\text{ in.}^4}{4.25\text{ in.}^2}} \\ &= 0.826\text{ in.} \end{aligned}$$

The effective radius of gyration, r_{ts}, may be conservatively approximated from the User Note in AISC *Specification* Section F2.2. A more exact method may be derived as discussed in AISC Design Guide 9, *Torsional Analysis of Structural Steel Members* (Seaburg and Carter, 1997), for a Z-shape that excludes lips.

From AISC *Specification* Section F2.2 User Note:

$$r_{ts} \approx \frac{b_f}{\sqrt{12\left(1+\frac{1}{6}\frac{ht_w}{b_f t_f}\right)}}$$

$$= \frac{2.50\text{ in.}}{\sqrt{12\left\{1+\left(\frac{1}{6}\right)\left[\frac{(11.5\text{ in.})(\text{¼ in.})}{(2.50\text{ in.})(\text{¼ in.})}\right]\right\}}}$$

$$= 0.543\text{ in.}$$

From Chapter 2 of ASCE/SEI 7, the required flexural strength is:

LRFD	ASD
$w_u = 1.2(0.025\text{ kip/ft}) + 1.6(0.10\text{ kip/ft})$ $= 0.190\text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_u = \frac{w_u L^2}{8}$ $= \frac{(0.190\text{ kip/ft})(18\text{ ft})^2}{8}$ $= 7.70\text{ kip-ft}$	$w_a = 0.025\text{ kip/ft} + 0.10\text{ kip/ft}$ $= 0.125\text{ kip/ft}$ From AISC *Manual* Table 3-23, Case 1: $M_a = \frac{w_a L^2}{8}$ $= \frac{(0.125\text{ kip/ft})(18\text{ ft})^2}{8}$ $= 5.06\text{ kip-ft}$

Nominal Flexural Strength

Flexural Yielding

From AISC *Specification* Section F12.1, the nominal flexural strength based on the limit state of flexural yielding is,

$$F_n = F_y \qquad \text{(}\textit{Spec}\text{. Eq. F12-2)}$$

$$= 36\text{ ksi}$$

$$M_n = F_n S_{min} \qquad \text{(}\textit{Spec}\text{. Eq. F12-1)}$$

$$= (36\text{ ksi})(13.2\text{ in.}^3)$$

$$= 475\text{ kip-in.}$$

Local Buckling

There are no specific local buckling provisions for Z-shapes in the AISC *Specification*. Use provisions for rolled channels from AISC *Specification* Table B4.1b, Cases 10 and 15.

Flange Slenderness

Conservatively neglecting the end return,

$$\begin{aligned}\lambda &= \frac{b}{t_f}\\ &= \frac{2.50\text{ in.}}{\tfrac{1}{4}\text{ in.}}\\ &= 10.0\end{aligned}$$

$$\begin{aligned}\lambda_p &= 0.38\sqrt{\frac{E}{F_y}}\\ &= 0.38\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}}\\ &= 10.8\end{aligned}$$ (*Spec*. Table B4.1b, Case 10)

$\lambda < \lambda_p$; therefore, the flange is compact

Web Slenderness

$$\begin{aligned}\lambda &= \frac{h}{t_w}\\ &= \frac{11.5\text{ in.}}{\tfrac{1}{4}\text{ in.}}\\ &= 46.0\end{aligned}$$

$$\begin{aligned}\lambda_p &= 3.76\sqrt{\frac{E}{F_y}}\\ &= 3.76\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}}\\ &= 107\end{aligned}$$ (*Spec*. Table B4.1b, Case 15)

$\lambda < \lambda_p$; therefore, the web is compact

Therefore, the local buckling limit state does not apply.

Lateral-Torsional Buckling

Per the User Note in AISC *Specification* Section F12, take the critical lateral-torsional buckling stress as half that of the equivalent channel. This is a conservative approximation of the lateral-torsional buckling strength which accounts for the rotation between the geometric and principal axes of a Z-shaped cross section, and is adopted from the *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI, 2016).

Calculate limiting unbraced lengths.

For bracing at 6 ft on center,

$$\begin{aligned}L_b &= (6\text{ ft})(12\text{ in./ft})\\ &= 72.0\text{ in.}\end{aligned}$$

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \qquad \text{(}\textit{Spec}\text{. Eq. F2-5)}$$

$$= 1.76\left(0.826 \text{ in.}\right)\sqrt{\frac{29{,}000 \text{ ksi}}{36 \text{ ksi}}}$$

$$= 41.3 \text{ in.} < 72.0 \text{ in.}$$

Per the User Note in AISC *Specification* Section F2, the square root term in AISC *Specification* Equation F2-4 can conservatively be taken equal to one. Therefore, Equation F2-6 can also be simplified. Substituting $0.7F_y$ for F_{cr} (where F_{cr} is half of the critical lateral-torsional buckling stress of the equivalent channel) in Equation F2-4 and solving for $L_b = L_r$, AISC *Specification* Equation F2-6 becomes:

$$L_r = \pi r_{ts}\sqrt{\frac{0.5E}{0.7F_y}}$$

$$= \pi\left(0.543 \text{ in.}\right)\sqrt{\frac{0.5\left(29{,}000 \text{ ksi}\right)}{0.7\left(36 \text{ ksi}\right)}}$$

$$= 40.9 \text{ in.} < 72.0 \text{ in.}$$

Calculate one half of the critical lateral-torsional buckling stress of the equivalent channel.

$L_b > L_r$, therefore,

$$F_{cr} = \left(0.5\right)\frac{C_b \pi^2 E}{\left(\dfrac{L_b}{r_{ts}}\right)^2}\sqrt{1+0.078\left(\frac{Jc}{S_x h_o}\right)\left(\frac{L_b}{r_{ts}}\right)^2} \qquad \text{(from }\textit{Spec}\text{. Eq. F2-4)}$$

Conservatively taking the square root term as 1.0,

$$F_{cr} = \left(0.5\right)\left[\frac{C_b \pi^2 E}{\left(\dfrac{L_b}{r_{ts}}\right)^2}\right]\left(1.0\right)$$

$$= \left(0.5\right)\left[\frac{1.0\,\pi^2\left(29{,}000 \text{ ksi}\right)}{\left(\dfrac{72.0 \text{ in.}}{0.543 \text{ in.}}\right)^2}\right]\left(1.0\right)$$

$$= 8.14 \text{ ksi}$$

$$F_n = F_{cr} \le F_y \qquad \text{(}\textit{Spec}\text{. Eq. F12-3)}$$

$$= 8.14 \text{ ksi} \le 36 \text{ ksi} \quad \textbf{o.k.}$$

$$M_n = F_n S_{min} \qquad \text{(}\textit{Spec}\text{. Eq. F12-1)}$$

$$= \left(8.14 \text{ ksi}\right)\left(13.2 \text{ in.}^3\right)$$

$$= 107 \text{ kip-in.}$$

The lateral-torsional buckling limit state controls.

M_n = 107 kip-in. or 8.92 kip-ft

Available Flexural Strength

From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90\left(8.92\text{ kip-ft}\right)$ $= 8.03\text{ kip-ft} > 7.70\text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{8.92\text{ kip-ft}}{1.67}$ $= 5.34\text{ kip-ft} > 5.06\text{ kip-ft}$ **o.k.**

Because the beam is loaded through the shear center, consideration of a torsional moment is unnecessary. If the loading produced torsion, the torsional effects should be evaluated using AISC Design Guide 9, *Torsional Analysis of Structural Steel Members* (Seaburg and Carter, 1997).

EXAMPLE F.15 PLATE GIRDER FLEXURAL MEMBER

Given:

Verify the built-up plate girder for the span and loads as shown in Figure F.15-1 with a cross section as shown in Figure F.15-2. The beam has a concentrated dead and live load at midspan and a uniformly distributed self weight. The plate girder is simply supported and is laterally braced at quarter and end points. The deflection of the girder is limited to 1 in. The plate girder is ASTM A572 Grade 50 material. The flange-to-web welds will be designed for both continuous and intermittent fillet welds using 70-ksi electrodes.

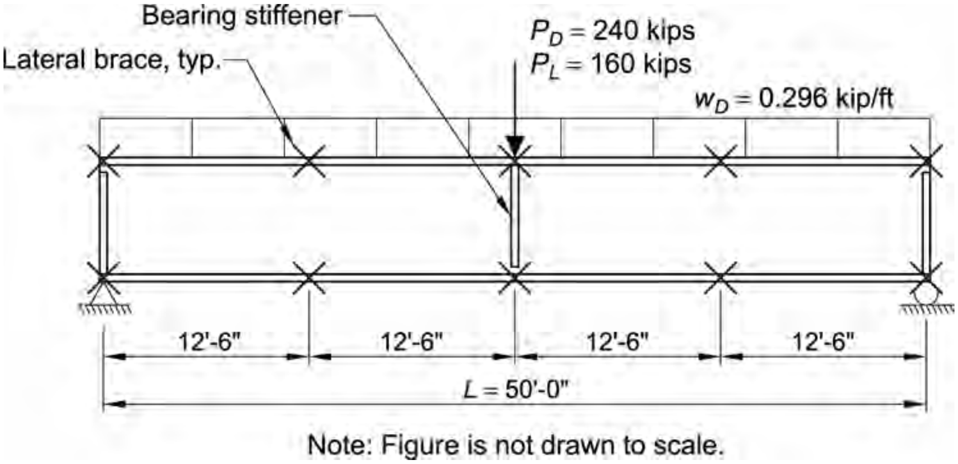


Fig. F.15-1. Beam loading and bracing diagram.

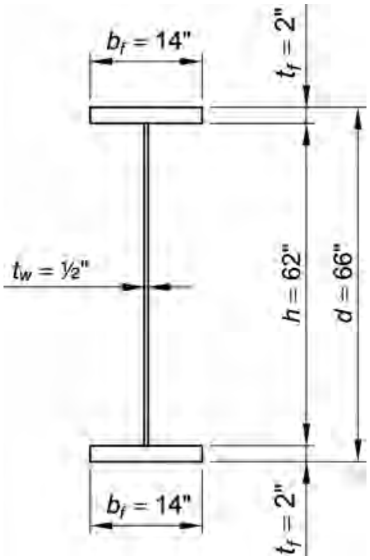


Fig. F.15-2. Plate girder geometry.

Solution:

From AISC *Manual* Table 2-5, the material properties are as follows:

ASTM A572 Grade 50
F_y = 50 ksi
F_u = 65 ksi

From ASCE/SEI 7, Chapter 2, the required shear and flexural strengths are:

LRFD	ASD
$P_u = 1.2(240\text{ kips}) + 1.6(160\text{ kips})$ $= 544\text{ kips}$ $w_u = 1.2(0.296\text{ kip/ft})$ $= 0.355\text{ kip/ft}$ $V_u = \dfrac{P_u}{2} + \dfrac{w_u L}{2}$ $= \dfrac{544\text{ kips}}{2} + \dfrac{(0.355\text{ kip/ft})(50\text{ ft})}{2}$ $= 281\text{ kips}$ $M_u = \dfrac{P_u L}{4} + \dfrac{w_u L^2}{8}$ $= \dfrac{(544\text{ kips})(50\text{ ft})}{4} + \dfrac{(0.355\text{ kip/ft})(50\text{ ft})^2}{8}$ $= 6{,}910\text{ kip-ft}$	$P_a = 240\text{ kips} + 160\text{ kips}$ $= 400\text{ kips}$ $w_a = 0.296\text{ kip/ft}$ $V_a = \dfrac{P_a}{2} + \dfrac{w_a L}{2}$ $= \dfrac{400\text{ kips}}{2} + \dfrac{(0.296\text{ kip/ft})(50\text{ ft})}{2}$ $= 207\text{ kips}$ $M_a = \dfrac{P_a L}{4} + \dfrac{w_a L^2}{8}$ $= \dfrac{(400\text{ kips})(50\text{ ft})}{4} + \dfrac{(0.296\text{ kip/ft})(50\text{ ft})^2}{8}$ $= 5{,}090\text{ kip-ft}$

Proportioning Limits

The proportioning limits from AISC *Specification* Section F13.2 are evaluated as follows, where *a* is the clear distance between transverse stiffeners.

$$\begin{aligned}\frac{a}{h} &= \frac{(25\text{ ft})(12\text{ in./ft})}{62\text{ in.}}\\ &= 4.84\end{aligned}$$

Because $a/h > 1.5$, use AISC *Specification* Equation F13-4.

$$\begin{aligned}\left(\frac{h}{t_w}\right)_{max} &= \frac{0.40E}{F_y} &&\text{(}\textit{Spec}\text{. Eq. F13-3)}\\ &= \frac{0.40(29{,}000\text{ ksi})}{50\text{ ksi}}\\ &= 232\end{aligned}$$

$$\begin{aligned}\frac{h}{t_w} &= \frac{62\text{ in.}}{\frac{1}{2}\text{ in.}}\\ &= 124 < 232 \quad \textbf{o.k.}\end{aligned}$$

From AISC *Specification* Section F13.2, the following limit applies to all built-up I-shaped members:

$$\begin{aligned}\frac{h_c t_w}{b_f t_f} &= \frac{(62\text{ in.})(\frac{1}{2}\text{ in.})}{(14\text{ in.})(2\text{ in.})} < 10\\ &= 1.11 < 10 \quad \textbf{o.k.}\end{aligned}$$

Section Properties

$$\begin{aligned}I_x &= \sum \frac{bh^3}{12} + \sum Ad^2 \\ &= \frac{(\text{½ in.})(62\text{ in.})^3}{12} + 2\left[\frac{(14\text{ in.})(2\text{ in.})^3}{12}\right] + 2\left[(2\text{ in.})(14\text{ in.})(32.0\text{ in.})^2\right] \\ &= 67{,}300\text{ in.}^4\end{aligned}$$

$$\begin{aligned}S_{xt} &= S_{xc} \\ &= \frac{I_x}{(d/2)} \\ &= \frac{67{,}300\text{ in.}^4}{(66\text{ in.}/2)} \\ &= 2{,}040\text{ in.}^3\end{aligned}$$

$$\begin{aligned}Z_x &= \sum A\bar{y} \\ &= (2)(\text{½ in.})(31.0\text{ in.})(31.0\text{ in.}/2) + (2)(2\text{ in.})(14\text{ in.})(32.0\text{ in.}) \\ &= 2{,}270\text{ in.}^3\end{aligned}$$

$$\begin{aligned}J &= \sum \frac{bt^3}{3} \\ &= 2\left[\frac{(14\text{ in.})(2\text{ in.})^3}{3}\right] + \frac{(62\text{ in.})(\text{½ in.})^3}{3} \\ &= 77.3\text{ in.}^4\end{aligned}$$

$$\begin{aligned}h_o &= h + t_f \\ &= 62\text{ in.} + 2\text{ in.} \\ &= 64.0\text{ in.}\end{aligned}$$

Deflection

The maximum deflection is:

$$\begin{aligned}\Delta &= \frac{(P_D + P_L)L^3}{48EI} + \frac{5w_D L^4}{384EI} \\ &= \frac{(240\text{ kips} + 160\text{ kips})(50\text{ ft})^3(12\text{ in./ft})^3}{48(29{,}000\text{ ksi})(67{,}300\text{ in.}^4)} + \frac{5(0.296\text{ kip/ft})(50\text{ ft})^4(12\text{ in./ft})^3}{384(29{,}000\text{ ksi})(67{,}300\text{ in.}^4)} \\ &= 0.944\text{ in.} < 1.00\text{ in.} \quad \textbf{o.k.}\end{aligned}$$

Web Slenderness

$$\begin{aligned}\lambda &= \frac{h}{t_w} \\ &= \frac{62\text{ in.}}{\text{½ in.}} \\ &= 124\end{aligned}$$

The limiting width-to-thickness ratios for the web are:

$$\begin{aligned}\lambda_{pw} &= 3.76\sqrt{\frac{E}{F_y}} \quad \text{from AISC } \textit{Specification} \text{ Table B4.1b, Case 15} \\ &= 3.76\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} \\ &= 90.6\end{aligned}$$

$$\begin{aligned}\lambda_{rw} &= 5.70\sqrt{\frac{E}{F_y}} \text{from AISC } \textit{Specification} \text{ Table B4.1b, Case 15} \\ &= 5.70\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} \\ &= 137\end{aligned}$$

$\lambda_{pw} < \lambda < \lambda_{rw}$, therefore the web is noncompact and AISC *Specification* Section F4 applies.

Flange Slenderness

$$\begin{aligned}\lambda &= \frac{b}{t} \\ &= \frac{b_f}{2t_f} \\ &= \frac{14\text{ in.}}{2\left(2\text{ in.}\right)} \\ &= 3.50\end{aligned}$$

$$\begin{aligned}\lambda_{pf} &= 0.38\sqrt{\frac{E}{F_y}} \quad \text{from AISC } \textit{Specification} \text{ Table B4.1b, Case 11} \\ &= 0.38\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} \\ &= 9.15 > \lambda, \text{ therefore the flanges are compact}\end{aligned}$$

Nominal Flexural Strength

Compression Flange Yielding

The web plastification factor is determined using AISC *Specification* Section F4.2(c)(6).

$$
\begin{aligned}
I_{yc} &= \frac{t_f b_f^3}{12} \\
&= \frac{(2 \text{ in.})(14 \text{ in.})^3}{12} \\
&= 457 \text{ in.}^4
\end{aligned}
$$

$$
\begin{aligned}
I_y &= 2\left(\frac{t_f b_f^3}{12}\right) + \frac{h t_w^3}{12} \\
&= 2\left[\frac{(2 \text{ in.})(14 \text{ in.})^3}{12}\right] + \frac{(62 \text{ in.})(\tfrac{1}{2} \text{ in.})^3}{12} \\
&= 915 \text{ in.}^4
\end{aligned}
$$

$$
\begin{aligned}
\frac{I_{yc}}{I_y} &= \frac{457 \text{ in.}^4}{915 \text{ in.}^4} \\
&= 0.499
\end{aligned}
$$

Because $I_{yc}/I_y > 0.23$, AISC *Specification* Section F4.2(c)(6)(i) applies.

$$
\begin{aligned}
M_p &= F_y Z_x \leq 1.6 F_y S_x \\
&= (50 \text{ ksi})\left(2{,}270 \text{ in.}^3\right)(1 \text{ ft/12 in.}) \leq 1.6(50 \text{ ksi})\left(2{,}040 \text{ in.}^3\right)(1 \text{ ft/12 in.}) \\
&= 9{,}460 \text{ kip-ft} < 13{,}600 \text{ kip-ft} \\
&= 9{,}460 \text{ kip-ft}
\end{aligned}
$$

$$
\begin{aligned}
M_{yc} &= F_y S_{xc} \\
&= (50 \text{ ksi})(2{,}040 \text{ kip-in.})(1 \text{ ft/12 in.}) \\
&= 8{,}500 \text{ kip-ft}
\end{aligned}
\qquad (Spec. \text{ Eq. F4-4})
$$

$$
\begin{aligned}
h_c &= h \\
&= 62 \text{ in.}
\end{aligned}
$$

$$
\begin{aligned}
\lambda &= \frac{h_c}{t_w} \\
&= \frac{62 \text{ in.}}{\tfrac{1}{2} \text{ in.}} \\
&= 124 > \lambda_{pw} = 90.6; \text{ therefore use AISC } \textit{Specification} \text{ Equation F4-9b}
\end{aligned}
$$

$$
\begin{aligned}
R_{pc} &= \frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right) \leq \frac{M_p}{M_{yc}} \qquad (Spec. \text{ Eq. F4-9b}) \\
&= \frac{9{,}460 \text{ kip-ft}}{8{,}500 \text{ kip-ft}} - \left(\frac{9{,}460 \text{ kip-ft}}{8{,}500 \text{ kip-ft}} - 1\right)\left(\frac{124 - 90.6}{137 - 90.6}\right) \leq \frac{9{,}460 \text{ kip-ft}}{8{,}500 \text{ kip-ft}} \\
&= 1.03 < 1.11 \\
&= 1.03
\end{aligned}
$$

The nominal flexural strength is calculated as:

$$\begin{aligned} M_n &= R_{pc}M_{yc} \\ &= (1.03)(8{,}500\text{ kip-ft}) \\ &= 8{,}760\text{ kip-ft} \end{aligned}$$ (*Spec*. Eq. F4-1)

From AISC *Specification* Section F4.1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(8{,}760\text{ kip-ft})$ $= 7{,}880\text{ kip-ft} > 6{,}910\text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{8{,}760\text{ kip-ft}}{1.67}$ $= 5{,}250\text{ kip-ft} > 5{,}090\text{ kip-ft}$ **o.k.**

Lateral-Torsional Buckling

The middle unbraced lengths control by inspection. For bracing at quarter points,

$$\begin{aligned} L_b &= (12.5\text{ ft})(12\text{ in./ft}) \\ &= 150\text{ in.} \end{aligned}$$

$$\begin{aligned} a_w &= \frac{h_c t_w}{b_{fc} t_{fc}} \\ &= \frac{(62\text{ in.})(½\text{ in.})}{(14\text{ in.})(2\text{ in.})} \\ &= 1.11 \end{aligned}$$ (*Spec*. Eq. F4-12)

$$\begin{aligned} r_t &= \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{6}a_w\right)}} \\ &= \frac{14.0\text{ in.}}{\sqrt{12\left[1+\left(\frac{1.11}{6}\right)\right]}} \\ &= 3.71\text{ in.} \end{aligned}$$ (*Spec*. Eq. F4-11)

From AISC *Specification* Equation F4-7:

$$\begin{aligned} L_p &= 1.1r_t\sqrt{\frac{E}{F_y}} \\ &= 1.1(3.71\text{ in.})\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} \\ &= 98.3 < 150\text{ in.; therefore, lateral-torsional buckling applies} \end{aligned}$$ (*Spec*. Eq. F4-7)

From AISC *Specification* Section F4.2(c)(3):

$$\frac{S_{xt}}{S_{xc}} = \frac{2{,}040 \text{ in.}^3}{2{,}040 \text{ in.}^3}$$
$$= 1.00 > 0.7; \text{ therefore, AISC } \textit{Specification} \text{ Equation F4-6a applies}$$

$$\begin{aligned} F_L &= 0.7F_y \\ &= 0.7(50 \text{ ksi}) \\ &= 35.0 \text{ ksi} \end{aligned}$$

(*Spec.* Eq. F4-6a)

From AISC *Specification* Equation F4-8:

$$L_r = 1.95 r_t \frac{E}{F_L}\sqrt{\frac{J}{S_{xc}h_o} + \sqrt{\left(\frac{J}{S_{xc}h_o}\right)^2 + 6.76\left(\frac{F_L}{E}\right)^2}}$$

(*Spec.* Eq. F4-8)

$$= 1.95(3.71 \text{ in.})\left(\frac{29{,}000 \text{ ksi}}{35.0 \text{ ksi}}\right)\sqrt{\frac{77.3 \text{ in.}^4}{\left(2{,}040 \text{ in.}^3\right)(64.0 \text{ in.})} + \sqrt{\left[\frac{77.3 \text{ in.}^4}{\left(2{,}040 \text{ in.}^3\right)(64.0 \text{ in.})}\right]^2 + 6.76\left(\frac{35.0 \text{ ksi}}{29{,}000 \text{ ksi}}\right)^2}}$$
$$= 369 \text{ in.}$$

$L_p < L_b \le L_r$; therefore, use AISC *Specification* Equation F4-2

The lateral-torsional buckling modification factor is determined by solving for the moment in the beam using statics. Note: The following solution uses LRFD load combinations. Using ASD load combinations will give approximately the same solution for C_b.

$$\begin{aligned} M_{max} &= 6{,}910 \text{ kip-ft} \\ M_A &= 4{,}350 \text{ kip-ft} \\ M_B &= 5{,}210 \text{ kip-ft} \\ M_C &= 6{,}060 \text{ kip-ft} \end{aligned}$$

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C}$$

(*Spec.* Eq. F1-1)

$$= \frac{12.5(6{,}910 \text{ kip-ft})}{2.5(6{,}910 \text{ kip-ft}) + 3(4{,}350 \text{ kip-ft}) + 4(5{,}210 \text{ kip-ft}) + 3(6{,}060 \text{ kip-ft})}$$
$$= 1.25$$

The nominal flexural strength is calculated as:

$$M_n = C_b\left[R_{pc}M_{yc} - \left(R_{pc}M_{yc} - F_LS_{xc}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le R_{pc}M_{yc}$$

(*Spec.* Eq. F4-2)

$$\begin{aligned} &= 1.25\left\{8{,}760 \text{ kip-ft} - \left[8{,}760 \text{ kip-ft} - (35.0 \text{ ksi})\left(2{,}040 \text{ in.}^3\right)(1 \text{ ft/12 in.})\right]\left(\frac{150 \text{ in.} - 98.3 \text{ in.}}{369 \text{ in.} - 98.3 \text{ in.}}\right)\right\} \le 8{,}760 \text{ kip-ft} \\ &= 10{,}300 \text{ kip-ft} > 8{,}760 \text{ kip-ft} \\ &= 8{,}760 \text{ kip-ft} \end{aligned}$$

From AISC *Specification* Section F4.2, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(8{,}760 \text{ kip-ft})$ $= 7{,}880 \text{ kip-ft} > 6{,}910 \text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{8{,}760 \text{ kip-ft}}{1.67}$ $= 5{,}250 \text{ kip-ft} > 5{,}090 \text{ kip-ft}$ **o.k.**

Compression Flange Local Buckling

From AISC *Specification* Section F4.3(a), this limit state does not apply because the flanges are compact.

Tension Flange Yielding

From AISC *Specification* Section F4.4(a), because $S_{xt} = S_{xc}$, this limit state does not apply.

Nominal Shear Strength

Determine the nominal shear strength without tension field action, using AISC *Specification* Section G2.1. For built-up I-shaped members, determine C_{v1} and k_v from AISC *Specification* Section G2.1(b).

$$\begin{aligned}\frac{a}{h} &= \frac{(25.0 \text{ ft})(12 \text{ in./ft}) - \text{½ in.}}{62 \text{ in.}}\\ &= 4.83 > 3.0\end{aligned}$$

From AISC *Specification* Section G2.1(b)(2):

$k_v = 5.34$

$$\begin{aligned}1.10\sqrt{\frac{k_v E}{F_y}} &= 1.10\sqrt{\frac{5.34(29{,}000 \text{ ksi})}{50 \text{ ksi}}}\\ &= 61.2 < h/t_w = 124; \text{ therefore, AISC } \textit{Specification} \text{ Equation G2-4 applies}\end{aligned}$$

$$\begin{aligned}C_{v1} &= \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \qquad \text{(}\textit{Spec}\text{. Eq. G2-4)}\\ &= \frac{61.2}{124}\\ &= 0.494\end{aligned}$$

The nominal shear strength is calculated as follows:

$$\begin{aligned}V_n &= 0.6F_y A_w C_{v1} \qquad \text{(}\textit{Spec}\text{. Eq. G2-1)}\\ &= 0.6(50 \text{ ksi})(66 \text{ in.})(\text{½ in.})(0.494)\\ &= 489 \text{ kips}\end{aligned}$$

From AISC *Specification* Section G.1, the available shear strength is:

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(489 \text{ kips})$ $= 440 \text{ kips} > 281 \text{ kips}$ **o.k.**	$\Omega_v = 1.67$ $\dfrac{V_n}{\Omega_v} = \dfrac{489 \text{ kips}}{1.67}$ $= 293 \text{ kips} > 207 \text{ kips}$ **o.k.**

Flange-to-Web Fillet Weld—Continuous Weld

Calculate the required shear flow using VQ/I_x because the stress distribution is linearly elastic away from midspan.

$$\begin{aligned} Q &= A\bar{y} \\ &= b_f t_f \left(\frac{h}{2} + \frac{t_f}{2} \right) \\ &= (14 \text{ in.})(2 \text{ in.})\left(\frac{62 \text{ in.}}{2} + \frac{2 \text{ in.}}{2} \right) \\ &= 896 \text{ in.}^3 \end{aligned}$$

LRFD	ASD
$R_u = \dfrac{V_u Q}{I_x}$ $= \dfrac{(281 \text{ kips})(896 \text{ in.}^3)}{67{,}300 \text{ in.}^4}$ $= 3.74 \text{ kip/in.}$	$R_a = \dfrac{V_a Q}{I_x}$ $= \dfrac{(207 \text{ kips})(896 \text{ in.}^3)}{67{,}300 \text{ in.}^4}$ $= 2.76 \text{ kip/in.}$

From AISC *Specification* Table J2.4, the minimum fillet weld size that can be used on the ½-in.-thick web is:

$$w_{min} = \tfrac{3}{16} \text{ in.}$$

From AISC *Manual* Part 8, the required fillet weld size is:

LRFD	ASD
$D_{req} = \dfrac{R_u}{1.392(2 \text{ sides})}$ (from *Manual* Eq. 8-2a) $= \dfrac{3.74 \text{ kip/in.}}{1.392(2 \text{ sides})}$ $= 1.34 \text{ sixteenths} < 3 \text{ sixteenths}$ Use $w = \tfrac{3}{16}$ in.	$D_{req} = \dfrac{R_a}{0.928(2 \text{ sides})}$ (from *Manual* Eq. 8-2b) $= \dfrac{2.76 \text{ kip/in.}}{0.928(2 \text{ sides})}$ $= 1.49 \text{ sixteenths} < 3 \text{ sixteenths}$ Use $w = \tfrac{3}{16}$ in.

From AISC *Specification* Equation J2-2, the available shear rupture strength of the web in kip/in. is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = \phi F_{nBM} A_{BM}$ $= \phi 0.60 F_u t_w$ $= 0.75(0.60)(65\text{ ksi})(\tfrac{1}{2}\text{ in.})$ $= 14.6\text{ kip/in.} > 3.74\text{ kip/in.}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{F_{nBM} A_{BM}}{\Omega}$ $= \dfrac{0.60 F_u t_w}{\Omega}$ $= \dfrac{0.60(65\text{ ksi})(\tfrac{1}{2}\text{ in.})}{2.00}$ $= 9.75\text{ kip/in.} > 2.76\text{ kip/in.}$ **o.k.**

Flange-to-Web Fillet Weld—Intermittent Weld

The two sided intermittent weld is designed using the minimum fillet weld size determined previously, $w_{min} = \tfrac{3}{16}$ in., and spaced at 12 in. center-to-center.

LRFD	ASD
$R_u = \phi R_n$ (from *Manual* Eq. 8-2a) $= 1.392D(2\text{ sides})\left(\dfrac{l_{req}}{s}\right)$ Solving for l_{req}, $l_{req} = \dfrac{R_u s}{1.392D(2\text{ sides})}$ $= \dfrac{(3.74\text{ kip-in.})(12\text{ in.})}{1.392(3\text{ sixteenth})(2\text{ sides})}$ $= 5.37\text{ in.}$ Use l = 6 in. at 12 in. o.c.	$R_u = \dfrac{R_n}{\Omega}$ (from *Manual* Eq. 8-2b) $= 0.928D(2\text{ sides})\left(\dfrac{l_{req}}{s}\right)$ Solving for l_{req}, $l_{req} = \dfrac{R_u s}{0.928D(2\text{ sides})}$ $= \dfrac{(2.76\text{ kip-in.})(12\text{ in.})}{0.928(3\text{ sixteenth})(2\text{ sides})}$ $= 5.95\text{ in.}$ Use l = 6 in. at 12 in. o.c.

The limitations for a intermittent fillet weld are checked using AISC *Specification* Section J2.2b(e):

$$l \geq 4D$$
$$6\text{ in.} \geq 4(\tfrac{3}{16}\text{ in.})$$
$$6\text{ in.} > 0.75\text{ in.} \quad \textbf{o.k.}$$

$$l \geq 1\tfrac{1}{2}\text{ in.}$$
$$6\text{ in.} > 1\tfrac{1}{2}\text{ in.} \quad \textbf{o.k.}$$

CHAPTER F DESIGN EXAMPLE REFERENCES

AISI (2016), *North American Specification for the Design of Cold-Formed Steel Structural Members*, ANSI/AISI Standard S100, American Iron and Steel Institute, Washington D.C.

Seaburg, P.A. and Carter, C.J. (1997), *Torsional Analysis of Structural Steel Members*, Design Guide 9, AISC, Chicago, IL.

Chapter G
Design of Members for Shear

INTRODUCTION

This *Specification* chapter addresses webs of singly or doubly symmetric members subject to shear in the plane of the web, single angles and HSS subject to shear, and shear in the weak direction of singly or doubly symmetric shapes.

G1. GENERAL PROVISIONS

The design shear strength, $\phi_v V_n$, and the allowable shear strength, V_n/Ω_v, are determined as follows:

V_n = nominal shear strength based on shear yielding or shear buckling
ϕ_v = 0.90 (LRFD)
Ω_v = 1.67 (ASD)

Exception: For all current ASTM A6, W, S and HP shapes except W44×230, W40×149, W36×135, W33×118, W30×90, W24×55, W16×26 and W12×14 for F_y = 50 ksi:

ϕ_v = 1.00 (LRFD)
Ω_v = 1.50 (ASD)

Strong axis shear values are tabulated for W-shapes in AISC *Manual* Tables 3-2, 3-6 and 6-2, for S-shapes in AISC *Manual* Table 3-7, for C-shapes in AISC *Manual* Table 3-8, and for MC-shapes in AISC *Manual* Table 3-9. Strong axis shear values are tabulated for rectangular HSS, round HSS and pipe in Part IV. Weak axis shear values for W-shapes, S-shapes, C-shapes and MC-shapes, and shear values for angles, rectangular HSS and box members are not tabulated.

G2. I-SHAPED MEMBERS AND CHANNELS

This section includes provisions for shear strength of webs without the use of tension field action and for interior web panels considering tension field action. Provisions for the design of transverse stiffeners are also included in Section G2.

As indicated in the User Note of this section, virtually all W, S and HP shapes are not subject to shear buckling and are also eligible for the more liberal safety and resistance factors, ϕ_v = 1.00 (LRFD) and Ω_v = 1.50 (ASD). This is presented in Example G.1 for a W-shape. A channel shear strength design is presented in Example G.2. A built-up girder with a thin web and transverse stiffeners is presented in Example G.8.

G3. SINGLE ANGLES AND TEES

A single angle example is illustrated in Example G.3.

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G4. RECTANGULAR HSS, BOX SECTIONS, AND OTHER SINGLY AND DOUBLY SYMMETRIC MEMBERS

The shear height for HSS, h, is taken as the clear distance between the flanges less the inside corner radius on each side. If the corner radii are unknown, h shall be taken as the corresponding outside dimension minus 3 times the design thickness. A rectangular HSS example is provided in Example G.4.

G5. ROUND HSS

For all round HSS of ordinary length listed in the AISC *Manual*, F_{cr} can be taken as $0.6F_y$ in AISC *Specification* Equation G5-1. A round HSS example is illustrated in Example G.5.

G6. WEAK AXIS SHEAR IN DOUBLY SYMMETRIC AND SINGLY SYMMETRIC SHAPES

For examples of weak axis shear, see Example G.6 and Example G.7.

G7. BEAMS AND GIRDERS WITH WEB OPENINGS

For a beam and girder with web openings example, see AISC Design Guide 2, *Design of Steel and Composite Beams with Web Openings* (Darwin, 1990).

EXAMPLE G.1A W-SHAPE IN STRONG AXIS SHEAR

Given:

Using AISC *Manual* tables, determine the available shear strength and adequacy of an ASTM A992 W24×62 with end shears of 48 kips from dead load and 145 kips from live load.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From Chapter 2 of ASCE/SEI 7, the required shear strength is:

LRFD	ASD
$V_u = 1.2(48\text{ kips}) + 1.6(145\text{ kips})$ $= 290\text{ kips}$	$V_a = 48\text{ kips} + 145\text{ kips}$ $= 193\text{ kips}$

From AISC *Manual* Table 3-2, the available shear strength is:

LRFD	ASD
$\phi_v V_n = 306\text{ kips} > 290\text{ kips}$ **o.k.**	$\dfrac{V_n}{\Omega_v} = 204\text{ kips} > 193\text{ kips}$ **o.k.**

EXAMPLE G.1B W-SHAPE IN STRONG AXIS SHEAR

Given:

The available shear strength of the W-shape in Example G.1A was easily determined using tabulated values in the AISC *Manual*. This example demonstrates the calculation of the available strength by directly applying the provisions of the AISC *Specification*.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W24×62
d = 23.7 in.
t_w = 0.430 in.

Nominal Shear Strength

Except for very few sections, which are listed in the User Note, AISC *Specification* Section G2.1(a) is applicable to the I-shaped beams published in the AISC *Manual* for F_y = 50 ksi. The W-shape sections that do not meet the criteria of AISC *Specification* Section G2.1(a) are indicated with footnote “v” in Tables 1-1, 3-2 and 6-2.

$C_{v1} = 1.0$ (*Spec*. Eq. G2-2)

From AISC *Specification* Section G2.1, area of the web, A_w, is determined as follows:

$$\begin{aligned} A_w &= dt_w \\ &= (23.7 \text{ in.})(0.430 \text{ in.}) \\ &= 10.2 \text{ in.}^2 \end{aligned}$$

From AISC *Specification* Section G2.1, the nominal shear strength is:

$$\begin{aligned} V_n &= 0.6F_yA_wC_{v1} \\ &= 0.6(50 \text{ ksi})(10.2 \text{ in.}^2)(1.0) \\ &= 306 \text{ kips} \end{aligned}$$ (*Spec*. Eq. G2-1)

Available Shear Strength

From AISC *Specification* Section G2.1, the available shear strength is:

LRFD	ASD
$\phi_v = 1.00$ $\phi_v V_n = 1.00(306 \text{ kips}) = 306 \text{ kips}$	$\Omega_v = 1.50$ $\dfrac{V_n}{\Omega_v} = \dfrac{306 \text{ kips}}{1.50} = 204 \text{ kips}$

EXAMPLE G.2A CHANNEL IN STRONG AXIS SHEAR

Given:

Using AISC *Manual* tables, verify the available shear strength and adequacy of an ASTM A36 C15×33.9 channel with end shears of 17.5 kips from dead load and 52.5 kips from live load.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From Chapter 2 of ASCE/SEI 7, the required shear strength is:

LRFD	ASD
$V_u = 1.2(17.5\text{ kips}) + 1.6(52.5\text{ kips})$ $= 105\text{ kips}$	$V_a = 17.5\text{ kips} + 52.5\text{ kips}$ $= 70.0\text{ kips}$

From AISC *Manual* Table 3-8, the available shear strength is:

LRFD	ASD
$\phi_v V_n = 117\text{ kips} > 105\text{ kips}$ **o.k.**	$\dfrac{V_n}{\Omega_v} = 77.6\text{ kips} > 70.0\text{ kips}$ **o.k.**

EXAMPLE G.2B CHANNEL IN STRONG AXIS SHEAR

Given:

The available shear strength of the channel in Example G.2A was easily determined using tabulated values in the AISC *Manual*. This example demonstrates the calculation of the available strength by directly applying the provisions of the AISC *Specification*.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-5, the geometric properties are as follows:

C15×33.9
d = 15.0 in.
t_w = 0.400 in.

Nominal Shear Strength

All ASTM A36 channels listed in the AISC *Manual* have $h/t_w \leq 1.10\sqrt{k_v E / F_y}$; therefore,

$$C_{v1} = 1.0 \qquad \text{(}\textit{Spec}\text{. Eq. G2-3)}$$

From AISC *Specification* Section G2.1, the area of the web, A_w, is determined as follows:

$$\begin{aligned} A_w &= dt_w \\ &= (15.0\text{ in.})(0.400\text{ in.}) \\ &= 6.00\text{ in.}^2 \end{aligned}$$

From AISC *Specification* Section G2.1, the nominal shear strength is:

$$\begin{aligned} V_n &= 0.6F_y A_w C_{v1} \\ &= 0.6(36\text{ ksi})(6.00\text{ in.}^2)(1.0) \\ &= 130\text{ kips} \end{aligned} \qquad \text{(}\textit{Spec}\text{. Eq. G2-1)}$$

Available Shear Strength

Because AISC *Specification* Section G2.1(a) does not apply for channels, the values of ϕ_v = 1.00 (LRFD) and Ω_v = 1.50 (ASD) may not be used. Instead ϕ_v = 0.90 (LRFD) and Ω_v = 1.67 (ASD) from AISC *Specification* Section G1(a) must be used.

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LRFD	ASD
$\phi_v = 0.90$	$\Omega_v = 1.67$
$\phi_v V_n = 0.90(130\text{ kips})$ $= 117\text{ kips}$	$\dfrac{V_n}{\Omega_v} = \dfrac{130\text{ kips}}{1.67}$ $= 77.8\text{ kips}$

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EXAMPLE G.3 ANGLE IN SHEAR

Given:

Determine the available shear strength and adequacy of an ASTM A36 L5×3×¼ (long leg vertical) with end shears of 3.5 kips from dead load and 10.5 kips from live load.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-7, the geometric properties are as follows:

L5×3×¼
b = 5.00 in.
t = ¼ in.

From Chapter 2 of ASCE/SEI 7, the required shear strength is:

LRFD	ASD
$V_u = 1.2(3.5\text{ kips}) + 1.6(10.5\text{ kips})$ $= 21.0\text{ kips}$	$V_a = 3.5\text{ kips} + 10.5\text{ kips}$ $= 14.0\text{ kips}$

Nominal Shear Strength

Note: There are no tables in the AISC *Manual* for angles in shear, but the nominal shear strength can be calculated according to AISC *Specification* Section G3, as follows:

From AISC *Specification* Section G3:

k_v = 1.2

Determine C_{v2} from AISC *Specification* Section G2.2.

$$\frac{h}{t_w} = \frac{b}{t} = \frac{5.00\text{ in.}}{¼\text{ in.}} = 20.0$$

$$1.10\sqrt{\frac{k_v E}{F_y}} = 1.10\sqrt{\frac{1.2(29{,}000\text{ ksi})}{36\text{ ksi}}} = 34.2 > 20.0;\ \text{therefore, use AISC } \textit{Specification} \text{ Equation G2-9}$$

C_{v2} = 1.0 (*Spec*. Eq. G2-9)

From AISC *Specification* Section G3, the nominal shear strength is:

$$
\begin{aligned}
V_n &= 0.6F_y btC_{v2} &&\text{(}\textit{Spec}\text{. Eq. G3-1)}\\
&= 0.6(36\text{ ksi})(5.00\text{ in.})(\tfrac{1}{4}\text{ in.})(1.0)\\
&= 27.0\text{ kips}
\end{aligned}
$$

Available Shear Strength

From AISC *Specification* Section G1, the available shear strength is:

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(27.0\text{ kips})$ $= 24.3\text{ kips} > 21.0\text{ kips}$ **o.k.**	$\Omega_v = 1.67$ $\dfrac{V_n}{\Omega_v} = \dfrac{27.0\text{ kips}}{1.67}$ $= 16.2\text{ kips} > 14.0\text{ kips}$ **o.k.**

EXAMPLE G.4 RECTANGULAR HSS IN SHEAR

Given:

Determine the available shear strength by directly applying the provisions of the AISC *Specification* for an ASTM A500 Grade C HSS6×4×⅜ (long leg vertical) beam with end shears of 11 kips from dead load and 33 kips from live load.

Note: There are tables in Part IV of this document that provide the shear strength of square and rectangular HSS shapes.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500 Grade C, rectangular
F_y = 50 ksi
F_u = 62 ksi

From AISC *Manual* Table 1-11, the geometric properties are as follows:

HSS6×4×⅜
H = 6.00 in.
B = 4.00 in.
t = 0.349 in.

From Chapter 2 of ASCE/SEI 7, the required shear strength is:

LRFD	ASD
$V_u = 1.2(11\text{ kips}) + 1.6(33\text{ kips})$ $= 66.0\text{ kips}$	$V_a = 11\text{ kips} + 33\text{ kips}$ $= 44.0\text{ kips}$

Nominal Shear Strength

The nominal shear strength can be determined from AISC *Specification* Section G4 as follows:

The web shear buckling strength coefficient, C_{v2}, is found using AISC *Specification* Section G2.2 with $h/t_w = h/t$ and $k_v = 5$.

From AISC *Specification* Section G4, if the exact radius is unknown, h shall be taken as the corresponding outside dimension minus three times the design thickness.

$$\begin{aligned} h &= H - 3t \\ &= 6.00\text{ in.} - 3(0.349\text{ in.}) \\ &= 4.95\text{ in.} \end{aligned}$$

$$\begin{aligned} \frac{h}{t} &= \frac{4.95\text{ in.}}{0.349\text{ in.}} \\ &= 14.2 \end{aligned}$$

$$\begin{aligned} 1.10\sqrt{\frac{k_v E}{F_y}} &= 1.10\sqrt{\frac{5(29{,}000\text{ ksi})}{50\text{ ksi}}} \\ &= 59.2 > 14.2;\ \text{therefore use AISC } \textit{Specification} \text{ Equation G2-9} \end{aligned}$$

$C_{v2} = 1.0$ (*Spec*. Eq. G2-9)

Note: Most standard HSS sections listed in the AISC *Manual* have $C_{v2} = 1.0$ at $F_y \le 50$ ksi.

Calculate A_w.

$$\begin{aligned} A_w &= 2ht \\ &= 2(4.95\text{ in.})(0.349\text{ in.}) \\ &= 3.46\text{ in.}^2 \end{aligned}$$

Calculate V_n.

$$\begin{aligned} V_n &= 0.6F_y A_w C_{v2} \\ &= 0.6(50\text{ ksi})\left(3.46\text{ in.}^2\right)(1.0) \\ &= 104\text{ kips} \end{aligned}$$ (*Spec*. Eq. G4-1)

Available Shear Strength

From AISC *Specification* Section G1, the available shear strength is:

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(104\text{ kips})$ $= 93.6\text{ kips} > 66.0\text{ kips}$ **o.k.**	$\Omega_v = 1.67$ $\dfrac{V_n}{\Omega_v} = \dfrac{104\text{ kips}}{1.67}$ $= 62.3\text{ kips} > 44.0\text{ kips}$ **o.k.**

EXAMPLE G.5 ROUND HSS IN SHEAR

Given:

Determine the available shear strength by directly applying the provisions of the AISC *Specification* for an ASTM A500 Grade C round HSS16.000×0.375 beam spanning 32 ft with end shears of 30 kips from uniform dead load and 90 kips from uniform live load.

Note: There are tables in Part IV of this document that provide the shear strength of round HSS shapes.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500 Grade C, round HSS
F_y = 46 ksi
F_u = 62 ksi

From AISC *Manual* Table 1-13, the geometric properties are as follows:

HSS16.000×0.375
A = 17.2 in.^{2}
D/t = 45.8

From Chapter 2 of ASCE/SEI 7, the required shear strength is:

LRFD	ASD
$V_u = 1.2(30\text{ kips}) + 1.6(90\text{ kips})$ $= 180\text{ kips}$	$V_a = 30\text{ kips} + 90\text{ kips}$ $= 120\text{ kips}$

Nominal Shear Strength

The nominal strength can be determined from AISC *Specification* Section G5, as follows:

Using AISC *Specification* Section G5, calculate F_{cr} as the larger of:

$$F_{cr} = \frac{1.60E}{\sqrt{\dfrac{L_v}{D}}\left(\dfrac{D}{t}\right)^{\frac{5}{4}}} \qquad \text{(}\textit{Spec}\text{. Eq. G5-2a)}$$

and

$$F_{cr} = \frac{0.78E}{\left(\dfrac{D}{t}\right)^{\frac{3}{2}}}, \text{ but not to exceed } 0.6F_y \qquad \text{(}\textit{Spec}\text{. Eq. G5-2b)}$$

where L_v is taken as the distance from maximum shear force to zero; in this example, half the span.

$$\begin{aligned} L_v &= 0.5(32\text{ ft})(12\text{ in./ft}) \\ &= 192\text{ in.} \end{aligned}$$

$$F_{cr} = \frac{1.60E}{\sqrt{\frac{L_v}{D}}\left(\frac{D}{t}\right)^{\frac{5}{4}}} \quad \text{(Spec. Eq. G5-2a)}$$

$$= \frac{1.60(29{,}000 \text{ ksi})}{\sqrt{\frac{192 \text{ in.}}{16.0 \text{ in.}}}(45.8)^{5/4}}$$

$$= 112 \text{ ksi}$$

$$F_{cr} = \frac{0.78E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}} \quad \text{(Spec. Eq. G5-2b)}$$

$$= \frac{0.78(29{,}000 \text{ ksi})}{(45.8)^{3/2}}$$

$$= 73.0 \text{ ksi}$$

The maximum value of F_{cr} permitted is,

$$F_{cr} = 0.6F_y$$

$$= 0.6(46 \text{ ksi})$$

$$= 27.6 \text{ ksi} \quad \textbf{controls}$$

Note: AISC *Specification* Equations G5-2a and G5-2b will not normally control for the sections published in the AISC *Manual* except when high strength steel is used or the span is unusually long.

Calculate V_n using AISC *Specification* Section G5.

$$V_n = \frac{F_{cr}A_g}{2} \quad \text{(Spec. Eq. G5-1)}$$

$$= \frac{(27.6 \text{ ksi})(17.2 \text{ in.}^2)}{2}$$

$$= 237 \text{ kips}$$

Available Shear Strength

From AISC *Specification* Section G1, the available shear strength is:

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(237 \text{ kips})$ $= 213 \text{ kips} > 180 \text{ kips}$ **o.k.**	$\Omega_v = 1.67$ $\frac{V_n}{\Omega_v} = \frac{237 \text{ kips}}{1.67}$ $= 142 \text{ kips} > 120 \text{ kips}$ **o.k.**

EXAMPLE G.6 DOUBLY SYMMETRIC SHAPE IN WEAK AXIS SHEAR

Given:

Verify the available shear strength and adequacy of an ASTM A992 W21×48 beam with end shears of 20.0 kips from dead load and 60.0 kips from live load in the weak direction.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W21×48
$b_f = 8.14$ in.
$t_f = 0.430$ in.

From Chapter 2 of ASCE/SEI 7, the required shear strength is:

LRFD	ASD
$V_u = 1.2(20.0\text{ kips}) + 1.6(60.0\text{ kips})$ $= 120\text{ kips}$	$V_a = 20.0\text{ kips} + 60.0\text{ kips}$ $= 80.0\text{ kips}$

Nominal Shear Strength

From AISC *Specification* Section G6, for weak axis shear, use AISC *Specification* Equation G6-1.

Calculate C_{v2} using AISC *Specification* Section G2.2 with $h/t_w = b_f/2t_f$ and $k_v = 1.2$.

$$\begin{aligned}\frac{h}{t_w} &= \frac{b_f}{2t_f}\\ &= \frac{8.14\text{ in.}}{2(0.430\text{ in.})}\\ &= 9.47\end{aligned}$$

$$\begin{aligned}1.10\sqrt{\frac{k_v E}{F_y}} &= 1.10\sqrt{\frac{1.2(29{,}000\text{ ksi})}{50\text{ ksi}}}\\ &= 29.0 > 9.47\end{aligned}$$

Therefore, use AISC *Specification* Equation G2-9:

$$C_{v2} = 1.0$$

Note: From the User Note in AISC *Specification* Section G6, $C_{v2} = 1.0$ for all ASTM A6 W-, S-, M- and HP-shapes when $F_y \le 70$ ksi.

Calculate V_n. (Multiply the flange area by two to account for both shear resisting elements.)

$$V_n = 0.6F_y b_f t_f C_{v2}(2) \qquad \text{(from } \textit{Spec}\text{. Eq. G6-1)}$$
$$= 0.6(50\text{ ksi})(8.14\text{ in.})(0.430\text{ in.})(1.0)(2)$$
$$= 210\text{ kips}$$

Available Shear Strength

From AISC *Specification* Section G1, the available shear strength is:

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(210\text{ kips})$ $= 189\text{ kips} > 120\text{ kips}$ **o.k.**	$\Omega_v = 1.67$ $\dfrac{V_n}{\Omega_v} = \dfrac{210\text{ kips}}{1.67}$ $= 126\text{ kips} > 80.0\text{ kips}$ **o.k.**

EXAMPLE G.7 SINGLY SYMMETRIC SHAPE IN WEAK AXIS SHEAR

Given:

Verify the available shear strength and adequacy of an ASTM A36 C9×20 channel with end shears of 5 kips from dead load and 15 kips from live load in the weak direction.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-5, the geometric properties are as follows:

C9×20
b_f = 2.65 in.
t_f = 0.413 in.

From Chapter 2 of ASCE/SEI 7, the required shear strength is:

LRFD	ASD
$V_u = 1.2(5\text{ kips}) + 1.6(15\text{ kips})$ $= 30.0\text{ kips}$	$V_u = 5\text{ kips} + 15\text{ kips}$ $= 20.0\text{ kips}$

Nominal Shear Strength

Note: There are no AISC *Manual* tables for weak-axis shear in channel sections, but the available strength can be determined from AISC *Specification* Section G6.

Calculate C_{v2} using AISC *Specification* Section G2.2 with $h/t_w = b_f/t_f$ and k_v = 1.2.

$$\begin{aligned}\frac{h}{t_w} &= \frac{b_f}{t_f}\\ &= \frac{2.65\text{ in.}}{0.413\text{ in.}}\\ &= 6.42\end{aligned}$$

$$\begin{aligned}1.10\sqrt{\frac{k_v E}{F_y}} &= 1.10\sqrt{\frac{1.2(29{,}000\text{ ksi})}{36\text{ ksi}}}\\ &= 34.2 > 6.42\end{aligned}$$

Therefore, use AISC *Specification* Equation G2-9:

$$C_{v2} = 1.0$$

Calculate V_n. (Multiply the flange area by two to account for both shear resisting elements.)

$$\begin{aligned} V_n &= 0.6F_y b_f t_f C_{v2}(2) \\ &= 0.6(36 \text{ ksi})(2.65 \text{ in.})(0.413 \text{ in.})(1.0)(2) \\ &= 47.3 \text{ kips} \end{aligned}$$ (from *Spec*. Eq. G6-1)

Available Shear Strength

From AISC *Specification* Section G1, the available shear strength is:

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(47.3 \text{ kips})$ $= 42.6 \text{ kips} > 30.0 \text{ kips}$ **o.k.**	$\Omega_v = 1.67$ $\dfrac{V_n}{\Omega_v} = \dfrac{47.3 \text{ kips}}{1.67}$ $= 28.3 \text{ kips} > 20.0 \text{ kips}$ **o.k.**

EXAMPLE G.8A BUILT-UP GIRDER WITH TRANSVERSE STIFFENERS

Given:

Determine the available shear strength of a built-up I-shaped girder for the span and loading as shown in Figure G.8A. The girder is ASTM A36 material and 36 in. deep with 16-in. × 1½-in. flanges and a 5⁄16-in.-thick web. The compression flange is continuously braced. Determine if the member has sufficient available shear strength to support the end shear, without and with tension field action. Use transverse stiffeners, as required.

Note: This built-up girder was purposely selected with a thin web in order to illustrate the design of transverse stiffeners. A more conventionally proportioned plate girder may have at least a ½-in.-thick web and slightly smaller flanges.

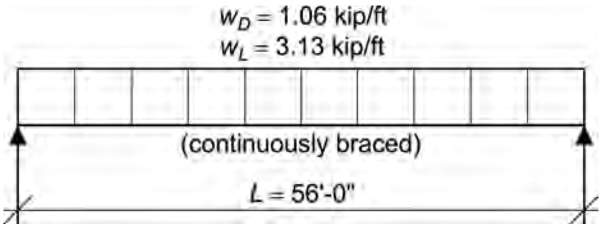

Fig. G.8A. Beam loading and bracing diagram.

Solution:

From AISC *Manual* Table 2-5, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

The geometric properties are as follows:

Built-up girder
t_w = 5⁄16 in.
d = 36.0 in.
$b_{ft} = b_{fc}$ = 16.0 in.
t_f = 1½ in.
h = 33.0 in.

From Chapter 2 of ASCE/SEI 7, the required shear strength at the support is:

LRFD	ASD
$w_u = 1.2(1.06 \text{ kip/ft}) + 1.6(3.13 \text{ kip/ft})$ $= 6.28 \text{ kip/ft}$ $V_u = \dfrac{w_u L}{2}$ $= \dfrac{(6.28 \text{ kip/ft})(56 \text{ ft})}{2}$ $= 176 \text{ kips}$	$w_a = 1.06 \text{ kip/ft} + 3.13 \text{ kip/ft}$ $= 4.19 \text{ kip/ft}$ $V_a = \dfrac{w_a L}{2}$ $= \dfrac{(4.19 \text{ kip/ft})(56 \text{ ft})}{2}$ $= 117 \text{ kips}$

Stiffener Requirement Check

From AISC *Specification* Section G2.1:

$$\begin{aligned} A_w &= dt_w \\ &= (36.0\text{ in.})(\tfrac{5}{16}\text{ in.}) \\ &= 11.3\text{ in.}^2 \end{aligned}$$

For webs without transverse stiffeners, k_v = 5.34 from AISC *Specification* Section G2.1(b)(2)(i).

$$\begin{aligned} \frac{h}{t_w} &= \frac{33.0\text{ in.}}{\tfrac{5}{16}\text{ in.}} \\ &= 106 \end{aligned}$$

$$\begin{aligned} 1.10\sqrt{\frac{k_v E}{F_y}} &= 1.10\sqrt{\frac{(5.34)(29{,}000\text{ ksi})}{36\text{ ksi}}} \\ &= 72.1 < 106 \end{aligned}$$

Therefore, use AISC *Specification* Equation G2-4:

$$\begin{aligned} C_{v1} &= \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \\ &= \frac{72.1}{106} \\ &= 0.680 \end{aligned} \qquad (\textit{Spec}.\text{ Eq. G2-4})$$

Calculate V_n.

$$\begin{aligned} V_n &= 0.6F_y A_w C_{v1} \\ &= 0.6(36\text{ ksi})(11.3\text{ in.}^2)(0.680) \\ &= 166\text{ kips} \end{aligned} \qquad (\textit{Spec}.\text{ Eq. G2-1})$$

From AISC *Specification* Section G1, the available shear strength without stiffeners is:

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(166\text{ kips})$ $= 149\text{ kips} < 176\text{ kips}$ **n.g.** **Therefore, stiffeners are required.**	$\Omega_v = 1.67$ $\dfrac{V_n}{\Omega_v} = \dfrac{166\text{ kips}}{1.67}$ $= 99.4\text{ kips} < 117\text{ kips}$ **n.g.** **Therefore, stiffeners are required.**

AISC *Manual* Tables 3-16a and 3-16b can be used to select the stiffener spacing needed to develop the required stress in the web.

Stiffener Spacing for End Panel

Tension field action is not permitted for end panels, therefore use AISC *Manual* Table 3-16a.

LRFD	ASD
Use $V_u = \phi_v V_n$ to determine the required stress in the web by dividing by the web area. $\dfrac{\phi_v V_n}{A_w} = \dfrac{V_u}{A_w}$ $= \dfrac{176 \text{ kips}}{11.3 \text{ in.}^2}$ $= 15.6 \text{ ksi}$	Use $V_a = V_n / \Omega_v$ to determine the required stress in the web by dividing by the web area. $\dfrac{V_n}{\Omega_v A_w} = \dfrac{V_a}{A_w}$ $= \dfrac{117 \text{ kips}}{11.3 \text{ in.}^2}$ $= 10.4 \text{ ksi}$

Use Table 3-16a from the AISC *Manual* to select the required stiffener ratio a/h based on the h/t_w ratio of the girder and the required stress. Interpolate and follow an available stress curve, $\phi_v V_n/A_w$= 15.6 ksi for LRFD, $V_n/\Omega_v A_w$ = 10.4 ksi for ASD, until it intersects the horizontal line for an h/t_w value of 106. Project down from this intersection and approximate the value for a/h as 1.40 from the axis across the bottom. Because h = 33.0 in., stiffeners are required at (1.40)(33.0 in.) = 46.2 in. maximum. Conservatively, use a 42-in. spacing.

Stiffener Spacing for the Second Panel

From AISC *Specification* Section G2.2, tension field action is allowed because the second panel is an interior web panel. However, a web panel aspect ratio, a/h, must not exceed three. The required shear strength at the start of the second panel, 42 in. from the end, is:

LRFD	ASD
$V_u = 176 \text{ kips} - (6.28 \text{ kip/ft})(42.0 \text{ in.})(1 \text{ ft}/12 \text{ in.})$ $= 154 \text{ kips}$	$V_a = 117 \text{ kips} - (4.19 \text{ kip/ft})(42.0 \text{ in.})(1 \text{ ft}/12 \text{ in.})$ $= 102 \text{ kips}$

From AISC *Specification* Section G1, the available shear strength without stiffeners is:

LRFD	ASD
$\phi_v = 0.90$ From previous calculations, $\phi_v V_n = 149 \text{ kips} < 154 \text{ kips}$ **n.g.** **Therefore, additional stiffeners are required.** Use $V_u = \phi_v V_n$ to determine the required stress in the web by dividing by the web area. $\dfrac{\phi_v V_n}{A_w} = \dfrac{V_u}{A_w}$ $= \dfrac{154 \text{ kips}}{11.3 \text{ in.}^2}$ $= 13.6 \text{ ksi}$	$\Omega_v = 1.67$ From previous calculations, $\dfrac{V_n}{\Omega_v} = 99.4 \text{ kips} < 102 \text{ kips}$ **n.g.** **Therefore, additional stiffeners are required.** Use $V_a = V_n / \Omega_v$ to determine the required stress in the web by dividing by the web area. $\dfrac{V_n}{\Omega_v A_w} = \dfrac{V_a}{A_w}$ $= \dfrac{102 \text{ kips}}{11.3 \text{ in.}^2}$ $= 9.03 \text{ ksi}$

Table 3-16b from the AISC *Manual*, including tension field action, may be used to select the required stiffener ratio a/h based on the h/t_w ratio of the girder and the required stress, provided that the limitations of $2A_w / (A_{fc} + A_{ft}) \le 2.5$, $h/b_{fc} \le 6.0$, and $h/b_{ft} \le 6.0$ are met.

$$\frac{2A_w}{A_{fc}+A_{ft}} = \frac{2\left(11.3\text{ in.}^2\right)}{(16.0\text{ in.})(1\tfrac{1}{2}\text{ in.})+(16.0\text{ in.})(1\tfrac{1}{2}\text{ in.})}$$

$$= 0.471 < 2.5 \quad \textbf{o.k.}$$

$$\frac{h}{b_{fc}} = \frac{h}{b_{ft}}$$

$$= \frac{33.0\text{ in.}}{16.0\text{ in.}}$$

$$= 2.06 < 6.0 \quad \textbf{o.k.}$$

The limitations have been met. Table 3-16b may be used.

Interpolate and follow an available stress curve, $\phi_v V_n/A_w$ = 13.6 ksi for LRFD, $V_n/\Omega_v A_w$ = 9.03 ksi for ASD, until it intersects the horizontal line for an h/t_w value of 106. Because the available stress does not intersect the h/t_w value of 106, the maximum value of 3.0 for a/h may be used. Because h = 33.0 in., an additional stiffener is required at (3.0)(33.0 in.) = 99.0 in. maximum from the previous one. Conservatively, 90.0 in. spacing may be used.

Stiffener Spacing for the Third Panel

From AISC *Specification* Section G2.2, tension field action is allowed because the next panel is not an end panel.

The required shear strength at the start of the third panel, 132 in. from the end is:

LRFD	ASD
$V_u = 176\text{ kips} - (6.28\text{ kip/ft})(132\text{ in.})(1\text{ ft/12 in.})$ $= 107\text{ kips}$	$V_a = 117\text{ kips} - (4.19\text{ kip/ft})(132\text{ in.})(1\text{ ft/12 in.})$ $= 70.9\text{ kips}$

From AISC *Specification* Section G1, the available shear strength without stiffeners is:

LRFD	ASD
$\phi_v = 0.90$ From previous calculations, $\phi_v V_n = 149\text{ kips} > 107\text{ kips}$ **o.k.** **Therefore, additional stiffeners are not required.**	$\Omega_v = 1.67$ From previous calculations, $\dfrac{V_n}{\Omega_v} = 99.4\text{ kips} > 70.9\text{ kips}$ **o.k.** **Therefore, additional stiffeners are not required.**

The six tables in the AISC *Manual*, 3-16a, 3-16b, 3-16c, 3-17a, 3-17b and 3-17c, are useful because they permit a direct solution for the required stiffener spacing. Alternatively, you can select a stiffener spacing and check the resulting strength, although this process is likely to be iterative. In Example G.8B, the stiffener spacings used are taken from this example.

EXAMPLE G.8B BUILT-UP GIRDER WITH TRANSVERSE STIFFENERS

Given:

Verify the available shear strength and adequacy of the stiffener spacings from Example G.8A, which were easily determined from the tabulated values of the AISC *Manual*, by directly applying the provisions of the AISC *Specification*. Stiffeners are spaced at 42 in. in the first panel and 90 in. in the second panel.

Solution:

From AISC *Manual* Table 2-5, the material properties are as follows:

ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From Example G.8A, the required shear strength at the support is:

LRFD	ASD
$V_u = 176$ kips	$V_a = 117$ kips

Shear Strength of End Panel

The web plate bucking coefficient, k_v, is determined from AISC *Specification* Equation G2-5.

$$\frac{h}{t_w} = \frac{33.0\text{ in.}}{5/16\text{ in.}}$$
$$= 106$$

$$k_v = 5 + \frac{5}{(a/h)^2} \qquad (\textit{Spec}.\text{ Eq. G2-5})$$
$$= 5 + \frac{5}{(42.0\text{ in.}/33.0\text{ in.})^2}$$
$$= 8.09$$

$$1.10\sqrt{\frac{k_v E}{F_y}} = 1.10\sqrt{\frac{8.09(29{,}000\text{ ksi})}{36\text{ ksi}}}$$
$$= 88.8 < 106$$

Therefore, use AISC *Specification* Equation G2-4.

$$C_{v1} = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \qquad (\textit{Spec}.\text{ Eq. G2-4})$$
$$= \frac{88.8}{106}$$
$$= 0.838$$

Calculate V_n.

From Example G.8A:

A_w = 11.3 in.^{2}

$$\begin{aligned} V_n &= 0.6F_y A_w C_{v1} \\ &= 0.6(36\text{ ksi})(11.3\text{ in.}^2)(0.838) \\ &= 205\text{ kips} \end{aligned}$$ (*Spec*. Eq. G2-1)

From AISC *Specification* Section G1, the available shear strength for the end panel is:

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(205\text{ kips})$ $= 185\text{ kips} > 176\text{ kips}$ **o.k.**	$\Omega_v = 1.67$ $\dfrac{V_n}{\Omega_v} = \dfrac{205\text{ kips}}{1.67}$ $= 123\text{ kips} > 117\text{ kips}$ **o.k.**

Shear Strength of the Second Panel

From Example G.8A, the required shear strength at the start of the second panel is:

LRFD	ASD
$V_u = 154\text{ kips}$	$V_a = 102\text{ kips}$

The web plate bucking coefficient, k_v, is determined from AISC *Specification* Equation G2-5.

$$\begin{aligned} k_v &= 5 + \frac{5}{(a/h)^2} \\ &= 5 + \frac{5}{(90.0\text{ in.}/33.0\text{ in.})^2} \\ &= 5.67 \end{aligned}$$ (*Spec*. Eq. G2-5)

$$\begin{aligned} 1.37\sqrt{\frac{k_v E}{F_y}} &= 1.37\sqrt{\frac{5.67(29{,}000\text{ ksi})}{36\text{ ksi}}} \\ &= 92.6 < 106 \end{aligned}$$

Therefore, use AISC *Specification* Equation G2-11 to calculate C_{v2}.

$$\begin{aligned} C_{v2} &= \frac{1.51 k_v E}{(h/t_w)^2 F_y} \\ &= \frac{1.51(5.67)(29{,}000\text{ ksi})}{(106)^2(36\text{ ksi})} \\ &= 0.614 \end{aligned}$$ (*Spec*. Eq. G2-11)

The limitations of AISC *Specification* Section G2.2(b)(1) are checked as follows:

$$\frac{2A_w}{A_{fc}+A_{ft}} = \frac{2\left(11.3\text{ in.}^2\right)}{(16.0\text{ in.})(1\tfrac{1}{2}\text{ in.})+(16.0\text{ in.})(1\tfrac{1}{2}\text{ in.})}$$
$$= 0.471 < 2.5$$

$$\frac{h}{b_{fc}} = \frac{h}{b_{ft}}$$
$$= \frac{33.0\text{ in.}}{16.0\text{ in.}}$$
$$= 2.06 < 6.0$$

Because $2A_w$ / $(A_{fc} + A_{ft}) \le 2.5$, $h/b_{fc} \le 6.0$, and $h/b_{ft} \le 6.0$, use AISC *Specification* Equation G2-7 with a = 90.0 in..

$$V_n = 0.6F_yA_w\left[C_{v2}+\frac{1-C_{v2}}{1.15\sqrt{1+(a/h)^2}}\right] \qquad \text{(}\textit{Spec}\text{. Eq. G2-7)}$$
$$= 0.6(36\text{ ksi})\left(11.3\text{ in.}^2\right)\left[0.614+\frac{1-0.614}{1.15\sqrt{1+\left(\frac{90.0\text{ in.}}{33.0\text{ in.}}\right)^2}}\right]$$
$$= 178\text{ kips}$$

From AISC *Specification* Section G1, the available shear strength for the second panel is:

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(178\text{ kips})$ $= 160\text{ kips} > 154\text{ kips}$ **o.k.**	$\Omega_v = 1.67$ $\frac{V_n}{\Omega_v} = \frac{178\text{ kips}}{1.67}$ $= 107\text{ kips} > 102\text{ kips}$ **o.k.**

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CHAPTER G DESIGN EXAMPLE REFERENCES

Darwin, D. (1990), *Steel and Composite Beams with Web Openings*, Design Guide 2, AISC, Chicago, IL.

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Chapter H
Design of Members for Combined Forces and Torsion

For all interaction equations in AISC *Specification* Chapter H, the required forces and moments must include second-order effects, as required by Chapter C of the AISC *Specification*. ASD users of the 1989 AISC *Specification* are accustomed to using an interaction equation that includes a partial second-order amplification. Second-order effects are now addressed in the analysis and are not included in these interaction equations.

EXAMPLE H.1A W-SHAPE SUBJECT TO COMBINED COMPRESSION AND BENDING ABOUT BOTH AXES (BRACED FRAME)

Given:

Using Table IV-5 (located in this document), determine if an ASTM A992 W14×99 has sufficient available strength to support the axial forces and moments listed as follows, obtained from a second-order analysis that includes *P*-δ effects. The unbraced length is 14 ft and the member has pinned ends.

LRFD	ASD
$P_u = 400\text{ kips}$ $M_{ux} = 250\text{ kip-ft}$ $M_{uy} = 80.0\text{ kip-ft}$	$P_a = 267\text{ kips}$ $M_{ax} = 167\text{ kip-ft}$ $M_{ay} = 53.3\text{ kip-ft}$

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

The effective length of the member is:

$$\begin{aligned} L_{cx} &= L_{cy} \\ &= KL \\ &= 1.0(14\text{ ft}) \\ &= 14.0\text{ ft} \end{aligned}$$

For $L_c = 14$ ft, the combined strength parameters from Table IV-5 are:

LRFD	ASD
$p = \dfrac{0.887}{10^3\text{ kips}}$	$p = \dfrac{1.33}{10^3\text{ kips}}$
$b_x = \dfrac{1.38}{10^3\text{ kip-ft}}$	$b_x = \dfrac{2.08}{10^3\text{ kip-ft}}$
$b_y = \dfrac{2.85}{10^3\text{ kip-ft}}$	$b_y = \dfrac{4.29}{10^3\text{ kip-ft}}$
Check P_r/P_c limit for AISC *Specification* Equation H1-1a.	Check P_r/P_c limit for AISC *Specification* Equation H1-1a.
$\dfrac{P_u}{\phi_c P_n} = pP_u = \left(\dfrac{0.887}{10^3\text{ kips}}\right)(400\text{ kips}) = 0.355$	$\dfrac{P_a}{P_n/\Omega_c} = pP_a = \left(\dfrac{1.33}{10^3\text{ kips}}\right)(267\text{ kips}) = 0.355$

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LRFD	ASD
Because $pP_u \geq 0.2$, $pP_u + b_x M_{ux} + b_y M_{uy} \leq 1.0$ (from Part IV, Eq. IV-8) $= 0.355 + \left(\dfrac{1.38}{10^3 \text{ kip-ft}}\right)(250 \text{ kip-ft})$ $+ \left(\dfrac{2.85}{10^3 \text{ kip-ft}}\right)(80.0 \text{ kip-ft}) \leq 1.0$ $= 0.928 < 1.0$ **o.k.**	Because $pP_a \geq 0.2$, $pP_a + b_x M_{ax} + b_y M_{ay} \leq 1.0$ (from Part IV, Eq. IV-8) $= 0.355 + \left(\dfrac{2.08}{10^3 \text{ kip-ft}}\right)(167 \text{ kip-ft})$ $+ \left(\dfrac{4.29}{10^3 \text{ kip-ft}}\right)(53.3 \text{ kip-ft}) \leq 1.0$ $= 0.931 < 1.0$ **o.k.**

Table IV-5 simplifies the calculation of AISC *Specification* Equations H1-1a and H1-1b. A direct application of these equations is shown in Example H.1B.

EXAMPLE H.1B W-SHAPE SUBJECT TO COMBINED COMPRESSION AND BENDING MOMENT ABOUT BOTH AXES (BRACED FRAME)

Given:

Using AISC *Manual* tables to determine the available compressive and flexural strengths, determine if an ASTM A992 W14×99 has sufficient available strength to support the axial forces and moments listed as follows, obtained from a second-order analysis that includes P-δ effects. The unbraced length is 14 ft and the member has pinned ends.

LRFD	ASD
$P_u = 400$ kips $M_{ux} = 250$ kip-ft $M_{uy} = 80$ kip-ft	$P_a = 267$ kips $M_{ax} = 167$ kip-ft $M_{ay} = 53.3$ kip-ft

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

The effective length of the member is:

$$\begin{aligned} L_{cx} &= L_{cy} \\ &= KL \\ &= 1.0(14 \text{ ft}) \\ &= 14.0 \text{ ft} \end{aligned}$$

For $L_c = 14.0$ ft, the available axial and flexural strengths from AISC *Manual* Table 6-2 are:

LRFD	ASD
$P_c = \phi_c P_n$ $= 1{,}130$ kips	$P_c = \dfrac{P_n}{\Omega_c}$ $= 750$ kips
$M_{cx} = \phi_b M_{nx}$ $= 642$ kip-ft	$M_{cx} = \dfrac{M_{nx}}{\Omega_b}$ $= 427$ kip-ft
$M_{cy} = \phi_b M_{ny}$ $= 311$ kip-ft	$M_{cy} = \dfrac{M_{ny}}{\Omega_b}$ $= 207$ kip-ft
$\dfrac{P_u}{\phi_c P_n} = \dfrac{400 \text{ kips}}{1{,}130 \text{ kips}}$ $= 0.354$	$\dfrac{P_a}{P_n / \Omega_c} = \dfrac{267 \text{ kips}}{750 \text{ kips}}$ $= 0.356$

LRFD	ASD
Because $\dfrac{P_u}{\phi_c P_n} \geq 0.2$, $\dfrac{P_r}{P_c} + \dfrac{8}{9}\left(\dfrac{M_{rx}}{M_{cx}} + \dfrac{M_{ry}}{M_{cy}}\right) \leq 1.0$ (*Spec*. Eq. H1-1a) $= \dfrac{400\text{ kips}}{1{,}130\text{ kips}} + \dfrac{8}{9}\left(\dfrac{250\text{ kip-ft}}{642\text{ kip-ft}} + \dfrac{80.0\text{ kip-ft}}{311\text{ kip-ft}}\right) \leq 1.0$ $= 0.928 < 1.0$ **o.k.**	Because $\dfrac{P_a}{P_n / \Omega_c} \geq 0.2$, $\dfrac{P_r}{P_c} + \dfrac{8}{9}\left(\dfrac{M_{rx}}{M_{cx}} + \dfrac{M_{ry}}{M_{cy}}\right) \leq 1.0$ (*Spec*. Eq. H1-1a) $= \dfrac{267\text{ kips}}{750\text{ kips}} + \dfrac{8}{9}\left(\dfrac{167\text{ kip-ft}}{427\text{ kip-ft}} + \dfrac{53.3\text{ kip-ft}}{207\text{ kip-ft}}\right)$ $= 0.932 < 1.0$ **o.k.**

EXAMPLE H.2 W-SHAPE SUBJECT TO COMBINED COMPRESSION AND BENDING MOMENT ABOUT BOTH AXES (BY AISC *SPECIFICATION* SECTION H2)

Given:

Using AISC *Specification* Section H2, determine if an ASTM A992 W14×99 has sufficient available strength to support the axial forces and moments listed as follows, obtained from a second-order analysis that includes P-δ effects. The unbraced length is 14 ft and the member has pinned ends. This example is included primarily to illustrate the use of AISC *Specification* Section H2.

LRFD	ASD
$P_u = 360$ kips $M_{ux} = 250$ kip-ft $M_{uy} = 80$ kip-ft	$P_a = 240$ kips $M_{ax} = 167$ kip-ft $M_{ay} = 53.3$ kip-ft

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W14×99
A = 29.1 in.^{2}
S_x = 157 in.^{3}
S_y = 55.2 in.^{3}

The required flexural and axial stresses are:

LRFD	ASD
$f_{ra} = \dfrac{P_u}{A}$ $= \dfrac{360\text{ kips}}{29.1\text{ in.}^2}$ $= 12.4$ ksi	$f_{ra} = \dfrac{P_a}{A}$ $= \dfrac{240\text{ kips}}{29.1\text{ in.}^2}$ $= 8.25$ ksi
$f_{rbx} = \dfrac{M_{ux}}{S_x}$ $= \dfrac{(250\text{ kip-ft})(12\text{ in./ft})}{157\text{ in.}^3}$ $= 19.1$ ksi	$f_{rbx} = \dfrac{M_{ax}}{S_x}$ $= \dfrac{(167\text{ kip-ft})(12\text{ in./ft})}{157\text{ in.}^3}$ $= 12.8$ ksi
$f_{rby} = \dfrac{M_{uy}}{S_y}$ $= \dfrac{(80\text{ kip-ft})(12\text{ in./ft})}{55.2\text{ in.}^3}$ $= 17.4$ ksi	$f_{rby} = \dfrac{M_{ay}}{S_y}$ $= \dfrac{(53.3\text{ kip-ft})(12\text{ in./ft})}{55.2\text{ in.}^3}$ $= 11.6$ ksi

The effective length of the member is:

$$\begin{aligned} L_{cx} &= L_{cy} \\ &= KL \\ &= 1.0(14\text{ ft}) \\ &= 14.0\text{ ft} \end{aligned}$$

For L_c = 14.0 ft, calculate the available axial and flexural stresses using the available strengths from AISC *Manual* Table 6-2.

LRFD	ASD
$F_{ca} = \phi_c F_{cr}$ $= \dfrac{\phi_c P_n}{A}$ $= \dfrac{1{,}130\text{ kips}}{29.1\text{ in.}^2}$ $= 38.8\text{ ksi}$	$F_{ca} = \dfrac{F_{cr}}{\Omega_c}$ $= \dfrac{P_n}{\Omega_c A}$ $= \dfrac{750\text{ kips}}{29.1\text{ in.}^2}$ $= 25.8\text{ ksi}$
$F_{cbx} = \dfrac{\phi_b M_{nx}}{S_x}$ $= \dfrac{(642\text{ kip-ft})(12\text{ in./ft})}{157\text{ in.}^3}$ $= 49.1\text{ ksi}$	$F_{cbx} = \dfrac{M_{nx}}{\Omega_b S_x}$ $= \dfrac{(427\text{ kip-ft})(12\text{ in./ft})}{157\text{ in.}^3}$ $= 32.6\text{ ksi}$
$F_{cby} = \dfrac{\phi_b M_{ny}}{S_y}$ $= \dfrac{(311\text{ kip-ft})(12\text{ in./ft})}{55.2\text{ in.}^3}$ $= 67.6\text{ ksi}$	$F_{cby} = \dfrac{M_{ny}}{\Omega_b S_y}$ $= \dfrac{(207\text{ kip-ft})(12\text{ in./ft})}{55.2\text{ in.}^3}$ $= 45.0\text{ ksi}$

As shown in the LRFD calculation of F_{cby} in the preceding text, the available flexural stresses can exceed the yield stress in cases where the available strength is governed by yielding and the yielding strength is calculated using the plastic section modulus.

Combined Stress Ratio

From AISC *Specification* Section H2, check the combined stress ratios as follows:

LRFD	ASD
$\left\lvert \dfrac{f_{ra}}{F_{ca}} + \dfrac{f_{rbx}}{F_{cbx}} + \dfrac{f_{rby}}{F_{cby}} \right\rvert \le 1.0$ (from *Spec*. Eq. H2-1) $\left\lvert \dfrac{12.4\text{ ksi}}{38.8\text{ ksi}} + \dfrac{19.1\text{ ksi}}{49.1\text{ ksi}} + \dfrac{17.4\text{ ksi}}{67.6\text{ ksi}} \right\rvert = 0.966 < 1.0$ **o.k.**	$\left\lvert \dfrac{f_{ra}}{F_{ca}} + \dfrac{f_{rbx}}{F_{cbx}} + \dfrac{f_{rby}}{F_{cby}} \right\rvert \le 1.0$ (from *Spec*. Eq. H2-1) $\left\lvert \dfrac{8.25\text{ ksi}}{25.8\text{ ksi}} + \dfrac{12.8\text{ ksi}}{32.6\text{ ksi}} + \dfrac{11.6\text{ ksi}}{45.0\text{ ksi}} \right\rvert = 0.970 < 1.0$ **o.k.**

A comparison of these results with those from Example H.1B shows that AISC *Specification* Equation H1-1a will produce less conservative results than AISC *Specification* Equation H2-1 when its use is permitted.

Note: This check is made at a point on the cross section (extreme fiber, in this example). The designer must therefore determine which point on the cross section is critical, or check multiple points if the critical point cannot be readily determined.

EXAMPLE H.3 W-SHAPE SUBJECT TO COMBINED AXIAL TENSION AND FLEXURE

Given:

Select an ASTM A992 W-shape with a 14-in.-nominal-depth to carry forces of 29 kips from dead load and 87 kips from live load in axial tension, as well as the following moments due to uniformly distributed loads:

$M_{xD} = 32 \text{ kip-ft}$
$M_{xL} = 96 \text{ kip-ft}$

$M_{yD} = 11.3 \text{ kip-ft}$
$M_{yL} = 33.8 \text{ kip-ft}$

The unbraced length is 30 ft and the ends are pinned. Assume the connections are made with no holes.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From ASCE/SEI 7, Chapter 2, the required strengths are:

LRFD	ASD
$P_u = 1.2(29 \text{ kips}) + 1.6(87 \text{ kips})$ $= 174 \text{ kips}$	$P_a = 29 \text{ kips} + 87 \text{ kips}$ $= 116 \text{ kips}$
$M_{ux} = 1.2(32 \text{ kip-ft}) + 1.6(96 \text{ kip-ft})$ $= 192 \text{ kip-ft}$	$M_{ax} = 32 \text{ kip-ft} + 96 \text{ kip-ft}$ $= 128 \text{ kip-ft}$
$M_{uy} = 1.2(11.3 \text{ kip-ft}) + 1.6(33.8 \text{ kip-ft})$ $= 67.6 \text{ kip-ft}$	$M_{ay} = 11.3 \text{ kip-ft} + 33.8 \text{ kip-ft}$ $= 45.1 \text{ kip-ft}$

Try a W14×82.

From AISC *Manual* Tables 1-1 and 3-2, the properties are as follows:

W14×82
A_g = 24.0 in.^{2}
S_x = 123 in.^{3}
Z_x = 139 in.^{3}
S_y = 29.3 in.^{3}
Z_y = 44.8 in.^{3}
I_y = 148 in.^{4}
L_p = 8.76 ft
L_r = 33.2 ft

Nominal Tensile Strength

From AISC *Specification* Section D2(a), the nominal tensile strength due to tensile yielding in the gross section is:

$$P_n = F_y A_g \qquad \text{(}\textit{Spec}\text{. Eq. D2-1)}$$
$$= (50 \text{ ksi})(24.0 \text{ in.}^2)$$
$$= 1,200 \text{ kips}$$

Note that for a member with holes, the rupture strength of the member would also have to be computed using AISC *Specification* Equation D2-2.

Nominal Flexural Strength for Bending About the Major Axis

Yielding

From AISC *Specification* Section F2.1, the nominal flexural strength due to yielding (plastic moment) is:

$$M_{nx} = M_p = F_y Z_x \qquad \text{(}\textit{Spec}\text{. Eq. F2-1)}$$
$$= (50 \text{ ksi})(139 \text{ in.}^3)$$
$$= 6,950 \text{ kip-in.}$$

Lateral-Torsional Buckling

From AISC *Specification* Section F2.2, the nominal flexural strength due to lateral-torsional buckling is determined as follows:

Because $L_p < L_b \le L_r$, i.e., 8.76 ft < 30 ft < 33.2 ft, AISC *Specification* Equation F2-2 applies.

Lateral-Torsional Buckling Modification Factor, C_b

From AISC *Manual* Table 3-1, C_b = 1.14, without considering the beneficial effects of the tension force. However, per AISC *Specification* Section H1.2, C_b may be modified because the column is in axial tension concurrently with flexure.

$$P_{ey} = \frac{\pi^2 E I_y}{L_b{}^2}$$
$$= \frac{\pi^2 (29,000 \text{ ksi})(148 \text{ in.}^4)}{[(30 \text{ ft})(12.0 \text{ in./ft})]^2}$$
$$= 327 \text{ kips}$$

LRFD	ASD
$\sqrt{1+\frac{\alpha P_u}{P_{ey}}} = \sqrt{1+\frac{1.0(174\text{ kips})}{327\text{ kips}}}$ $= 1.24$	$\sqrt{1+\frac{\alpha P_a}{P_{ey}}} = \sqrt{1+\frac{1.6(116\text{ kips})}{327\text{ kips}}}$ $= 1.25$

$$C_b = 1.24(1.14)$$
$$= 1.41$$

$$M_n = C_b\left[M_p - \left(M_p - 0.7F_yS_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \qquad (Spec.\ \text{Eq. F2-2})$$

$$= 1.41\left\{6{,}950\text{ kip-in.} - \left[6{,}950\text{ kip-in.} - 0.7\left(50\text{ ksi}\right)\left(123\text{ in.}^3\right)\right]\left(\frac{30\text{ ft} - 8.76\text{ ft}}{33.2\text{ ft} - 8.76\text{ ft}}\right)\right\} \le 6{,}950\text{ kip-in.}$$

$$= 6{,}560\text{ kip-in. or }547\text{ kip-ft} \quad \textbf{controls}$$

Local Buckling

Per AISC *Manual* Table 1-1, the cross section is compact at F_y = 50 ksi; therefore, the local buckling limit state does not apply.

Nominal Flexural Strength for Bending About the Minor Axis and the Interaction of Flexure and Tension

Because a W14×82 has compact flanges, only the limit state of yielding applies for bending about the minor axis.

$$M_{ny} = M_p = F_yZ_y \le 1.6F_yS_y \qquad (Spec.\ \text{Eq. F6-1})$$

$$= \left(50\text{ ksi}\right)\left(44.8\text{ in.}^3\right) \le 1.6\left(50\text{ ksi}\right)\left(29.3\text{ in.}^3\right)$$

$$= 2{,}240\text{ kip-in.} < 2{,}340\text{ kip-in.}$$

$$= 2{,}240\text{ kip-in. or }187\text{ kip-ft}$$

Available Strength

From AISC *Specification* Sections D2 and F1, the available strengths are:

LRFD	ASD
$\phi_b = \phi_t = 0.90$	$\Omega_b = \Omega_t = 1.67$
$P_c = \phi_t P_n$ $= 0.90\left(1{,}200\text{ kips}\right)$ $= 1{,}080\text{ kips}$	$P_c = \dfrac{P_n}{\Omega_t}$ $= \dfrac{1{,}200\text{ kips}}{1.67}$ $= 719\text{ kips}$
$M_{cx} = \phi_b M_{nx}$ $= 0.90\left(547\text{ kip-ft}\right)$ $= 492\text{ kip-ft}$	$M_{cx} = \dfrac{M_{nx}}{\Omega_b}$ $= \dfrac{547\text{ kip-ft}}{1.67}$ $= 328\text{ kip-ft}$
$M_{cy} = \phi_b M_{ny}$ $= 0.90\left(187\text{ kip-ft}\right)$ $= 168\text{ kip-ft}$	$M_{cy} = \dfrac{M_{ny}}{\Omega_b}$ $= \dfrac{187\text{ kip-ft}}{1.67}$ $= 112\text{ kip-ft}$

Interaction of Tension and Flexure

Check limit for AISC *Specification* Equation H1-1a.

LRFD	ASD
$\dfrac{P_r}{P_c}=\dfrac{P_u}{\phi_t P_n}$ $=\dfrac{174\text{ kips}}{1{,}080\text{ kips}}$ $=0.161<0.2$ Because $\dfrac{P_r}{P_c}<0.2,$ $\dfrac{P_r}{2P_c}+\left(\dfrac{M_{rx}}{M_{cx}}+\dfrac{M_{ry}}{M_{cy}}\right)\leq 1.0$ (*Spec*. Eq. H1-1b) $=\dfrac{174\text{ kips}}{2(1{,}080\text{ kips})}+\dfrac{192\text{ kip-ft}}{492\text{ kip-ft}}+\dfrac{67.6\text{ kip-ft}}{168\text{ kip-ft}}\leq 1.0$ $=0.873<1.0$ **o.k.**	$\dfrac{P_r}{P_c}=\dfrac{P_a}{P_n/\Omega_t}$ $=\dfrac{116\text{ kips}}{719\text{ kips}}$ $=0.161<0.2$ Because $\dfrac{P_r}{P_c}<0.2,$ $\dfrac{P_r}{2P_c}+\left(\dfrac{M_{rx}}{M_{cx}}+\dfrac{M_{ry}}{M_{cy}}\right)\leq 1.0$ (*Spec*. Eq. H1-1b) $=\dfrac{116\text{ kips}}{2(719\text{ kips})}+\dfrac{128\text{ kip-ft}}{328\text{ kip-ft}}+\dfrac{45.1\text{ kip-ft}}{112\text{ kip-ft}}\leq 1.0$ $=0.874<1.0$ **o.k.**

EXAMPLE H.4 W-SHAPE SUBJECT TO COMBINED AXIAL COMPRESSION AND FLEXURE

Given:

Select an ASTM A992 W-shape with a 10-in.-nominal-depth to carry axial compression forces of 5 kips from dead load and 15 kips from live load. The unbraced length is 14 ft and the ends are pinned. The member also has the following required moment strengths due to uniformly distributed loads, not including second-order effects:

$M_{xD} = 15$ kip-ft
$M_{xL} = 45$ kip-ft

$M_{yD} = 2$ kip-ft
$M_{yL} = 6$ kip-ft

The member is not subject to sidesway (no lateral translation).

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

From Chapter 2 of ASCE/SEI 7, the required strength (not considering second-order effects) is:

LRFD	ASD
$P_u = 1.2(5\text{ kips}) + 1.6(15\text{ kips})$ $= 30.0\text{ kips}$	$P_a = 5\text{ kips} + 15\text{ kips}$ $= 20.0\text{ kips}$
$M_{ux} = 1.2(15\text{ kip-ft}) + 1.6(45\text{ kip-ft})$ $= 90.0\text{ kip-ft}$	$M_{ax} = 15\text{ kip-ft} + 45\text{ kip-ft}$ $= 60.0\text{ kip-ft}$
$M_{uy} = 1.2(2\text{ kip-ft}) + 1.6(6\text{ kip-ft})$ $= 12.0\text{ kip-ft}$	$M_{ay} = 2\text{ kip-ft} + 6\text{ kip-ft}$ $= 8.00\text{ kip-ft}$

Try a W10×33.

From AISC *Manual* Tables 1-1 and 3-2, the properties are as follows:

W10×33
$A = 9.71\text{ in.}^2$
$S_x = 35.0\text{ in.}^3$
$Z_x = 38.8\text{ in.}^3$
$I_x = 171\text{ in.}^4$
$r_x = 4.19$ in.
$S_y = 9.20\text{ in.}^3$
$Z_y = 14.0\text{ in.}^3$
$I_y = 36.6\text{ in.}^4$
$r_y = 1.94$ in.
$L_p = 6.85$ ft
$L_r = 21.8$ ft

Available Axial Strength

From AISC *Specification* Commentary Table C-A-7.1, for a pinned-pinned condition, K = 1.0. Because $L_c = KL_x = KL_y$ = 14.0 ft and $r_x > r_y$, the y-y axis will govern.

From AISC *Manual* Table 6-2, the available axial strength is:

LRFD	ASD
$P_c = \phi_c P_n$ $= 253\text{ kips}$	$P_c = \dfrac{P_n}{\Omega_c}$ $= 168\text{ kips}$

Required Flexural Strength (including second-order amplification)

Use the approximate method of second-order analysis procedure from AISC *Specification* Appendix 8. Because the member is not subject to sidesway, only P-δ amplifiers need to be added.

$$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1$$ (*Spec*. Eq. A-8-3)

where C_m is conservatively taken per AISC *Specification* A-8.2.1(b):

$C_m = 1.0$

The x-x axis flexural magnifier is:

$$\begin{aligned} P_{e1x} &= \frac{\pi^2 EI_x}{\left(L_{c1x}\right)^2} \\ &= \frac{\pi^2 (29{,}000\text{ ksi})(171\text{ in.}^4)}{\left[(14\text{ ft})(12\text{ in./ft})\right]^2} \\ &= 1{,}730\text{ kips} \end{aligned}$$ (from *Spec*. Eq. A-8-5)

LRFD	ASD
$\alpha = 1.0$	$\alpha = 1.6$
$B_{1x} = \dfrac{C_m}{1 - \alpha P_r / P_{e1x}} \geq 1.0$ $= \dfrac{1.0}{1 - 1.0(30\text{ kips}/1{,}730\text{ kips})} \geq 1.0$ $= 1.02$	$B_{1x} = \dfrac{C_m}{1 - \alpha P_r / P_{e1x}} \geq 1.0$ $= \dfrac{1.0}{1 - 1.6(20\text{ kips}/1{,}730\text{ kips})} \geq 1.0$ $= 1.02$
$M_{ux} = 1.02(90\text{ kip-ft})$ $= 91.8\text{ kip-ft}$	$M_{ax} = 1.02(60\text{ kip-ft})$ $= 61.2\text{ kip-ft}$

The y-y axis flexural magnifier is:

$$P_{e1y}=\frac{\pi^2 EI_y}{\left(L_{c1y}\right)^2}$$ (modified *Spec.* Eq. A-8-5)

$$=\frac{\pi^2\left(29{,}000\text{ ksi}\right)\left(36.6\text{ in.}^4\right)}{\left[\left(14\text{ ft}\right)\left(12\text{ in./ft}\right)\right]^2}$$

$$=371\text{ kips}$$

LRFD	ASD
$\alpha=1.0$ $B_{1y}=\dfrac{C_m}{1-\alpha P_r/P_{e1y}}\geq 1.0$ $=\dfrac{1.0}{1-1.0\left(30\text{ kips}/371\text{ kips}\right)}\geq 1.0$ $=1.09$ $M_{uy}=1.09\left(12\text{ kip-ft}\right)$ $=13.1\text{ kip-ft}$	$\alpha=1.6$ $B_{1y}=\dfrac{C_m}{1-\alpha P_r/P_{e1y}}\geq 1.0$ $=\dfrac{1.0}{1-1.6\left(20\text{ kips}/371\text{kips}\right)}\geq 1.0$ $=1.09$ $M_{ay}=1.09\left(8\text{ kip-ft}\right)$ $=8.72\text{ kip-ft}$

Nominal Flexural Strength about the Major Axis

Yielding

$$M_{nx}=M_p=F_yZ_x$$ (*Spec*. Eq. F2-1)

$$=\left(50\text{ ksi}\right)\left(38.8\text{ in.}^3\right)$$

$$=1{,}940\text{ kip-in.}$$

Lateral-Torsional Buckling

Because $L_p < L_b \leq L_r$, i.e., 6.85 ft < 14.0 ft < 21.8 ft, AISC *Specification* Equation F2-2 applies.

From AISC *Manual* Table 3-1, C_b = 1.14

$$M_{nx}=C_b\left[M_p-\left(M_p-0.7F_yS_x\right)\left(\frac{L_b-L_p}{L_r-L_p}\right)\right]\leq M_p$$ (*Spec*. Eq. F2-2)

$$=1.14\left\{1{,}940\text{ kip-in.}-\left[1{,}940\text{ kip-in.}-0.7\left(50\text{ ksi}\right)\left(35.0\text{ in.}^3\right)\right]\left(\frac{14\text{ ft}-6.85\text{ ft}}{21.8\text{ ft}-6.85\text{ ft}}\right)\right\}$$

$$=1{,}820\text{ kip-in.}<1{,}940\text{ kip-in.}$$

$$=1{,}820\text{ kip-in. or }152\text{ kip-ft}$$ **controls**

Local Buckling

Per AISC *Manual* Table 1-1, the member is compact for F_y = 50 ksi, so the local buckling limit state does not apply.

Nominal Flexural Strength about the Minor Axis

Determine the nominal flexural strength for bending about the minor axis from AISC *Specification* Section F6. Because a W10×33 has compact flanges, only the yielding limit state applies.

From AISC *Specification* Section F6.1:

$$
\begin{aligned}
M_{nx} &= M_p = F_y Z_x \le 1.6 F_y S_y && \text{(}\textit{Spec}\text{. Eq. F6-1)}\\
&= (50\text{ ksi})(14.0\text{ in.}^3) \le 1.6(50\text{ ksi})(9.20\text{ in.}^3)\\
&= 700\text{ kip-in.} < 736\text{ kip-in.}\\
&= 700\text{ kip-in. or } 58.3\text{ kip-ft}
\end{aligned}
$$

From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
$M_{cx} = \phi_b M_{nx}$ $= 0.90(152\text{ kip-ft})$ $= 137\text{ kip-ft}$	$M_{cx} = \dfrac{M_{nx}}{\Omega_b}$ $= \dfrac{152\text{ kip-ft}}{1.67}$ $= 91.0\text{ kip-ft}$
$M_{cy} = \phi_b M_{ny}$ $= 0.90(58.3\text{ kip-ft})$ $= 52.5\text{ kip-ft}$	$M_{cy} = \dfrac{M_{ny}}{\Omega_b}$ $= \dfrac{58.3\text{ kip-ft}}{1.67}$ $= 34.9\text{ kip-ft}$

Check limit for AISC *Specification* Equations H1-1a and H1-1b.

LRFD	ASD
$\dfrac{P_r}{P_c} = \dfrac{P_u}{\phi_c P_n}$ $= \dfrac{30\text{ kips}}{253\text{ kips}}$ $= 0.119 < 0.2$	$\dfrac{P_r}{P_c} = \dfrac{P_a}{P_n / \Omega_c}$ $= \dfrac{20\text{ kips}}{168\text{ kips}}$ $= 0.119 < 0.2$
Because $\dfrac{P_r}{P_c} < 0.2,$	Because $\dfrac{P_r}{P_c} < 0.2,$
$\dfrac{P_r}{2P_c} + \left(\dfrac{M_{rx}}{M_{cx}} + \dfrac{M_{ry}}{M_{cy}}\right) \le 1.0$ (*Spec*. Eq. H1-1b) $= \dfrac{30\text{ kips}}{2(253\text{ kips})} + \left(\dfrac{91.8\text{ kip-ft}}{137\text{ kip-ft}} + \dfrac{13.1\text{ kip-ft}}{52.5\text{ kip-ft}}\right) \le 1.0$ $= 0.979 < 1.0$ **o.k.**	$\dfrac{P_r}{2P_c} + \left(\dfrac{M_{rx}}{M_{cx}} + \dfrac{M_{ry}}{M_{cy}}\right) \le 1.0$ (*Spec*. Eq. H1-1b) $= \dfrac{20\text{ kips}}{2(168\text{ kips})} + \left(\dfrac{61.2\text{ kip-ft}}{91.0\text{ kip-ft}} + \dfrac{8.72\text{ kip-ft}}{34.9\text{ kip-ft}}\right) \le 1.0$ $= 0.982 < 1.0$ **o.k.**

EXAMPLE H.5A RECTANGULAR HSS TORSIONAL STRENGTH

Given:

Determine the available torsional strength of an ASTM A500, Grade C, HSS6×4×¼.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500 Grade C
F_y = 50 ksi
F_u = 62 ksi

From AISC *Manual* Table 1-11, the geometric properties are as follows:

HSS6×4×¼
t = 0.233 in.
b/t = 14.2
h/t = 22.8
C = 10.1 in.^{3}

The available torsional strength for rectangular HSS is stipulated in AISC *Specification* Section H3.1. The critical stress, F_{cr}, is determined from AISC *Specification* Section H3.1(b).

Because $h/t > b/t$, h/t governs.

$$2.45\sqrt{\frac{E}{F_y}} = 2.45\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}}$$

$$= 59.0 > 22.8; \text{ therefore, use AISC } \textit{Specification} \text{ Equation H3-3 to determine } F_{cr}$$

$$\begin{aligned} F_{cr} &= 0.6F_y \\ &= 0.6(50 \text{ ksi}) \\ &= 30.0 \text{ ksi} \end{aligned}$$ (*Spec*. Eq. H3-3)

The nominal torsional strength is:

$$\begin{aligned} T_n &= F_{cr}C \\ &= (30.0 \text{ ksi})(10.1 \text{ in.}^3) \\ &= 303 \text{ kip-in.} \end{aligned}$$ (*Spec*. Eq. H3-1)

From AISC *Specification* Section H3.1, the available torsional strength is:

LRFD	ASD
$\phi_T = 0.90$ $\phi_T T_n = 0.90(303 \text{ kip-in.})$ $= 273 \text{ kip-in.}$	$\Omega_T = 1.67$ $\dfrac{T_n}{\Omega_T} = \dfrac{303 \text{ kip-in.}}{1.67}$ $= 181 \text{ kip-in.}$

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Note: For more complete guidance on designing for torsion, see AISC Design Guide 9, *Torsional Analysis of Structural Steel Members* (Seaburg and Carter, 1997).

EXAMPLE H.5B ROUND HSS TORSIONAL STRENGTH

Given:

Determine the available torsional strength of an ASTM A500, Grade C, HSS5.000×0.250 that is 14 ft long.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500 Grade C
F_y = 46 ksi
F_u = 62 ksi

From AISC *Manual* Table 1-13, the geometric properties are as follows:

HSS5.000×0.250
D = 5.00 in.
t = 0.233 in.
D/t = 21.5
C = 7.95 in.^{3}

The available torsional strength for round HSS is stipulated in AISC *Specification* Section H3.1.The critical stress, F_{cr}, is determined from AISC *Specification* Section H3.1(a).

Calculate the critical stress as the larger of:

$$F_{cr} = \frac{1.23E}{\sqrt{\dfrac{L}{D}}\left(\dfrac{D}{t}\right)^{5/4}} \qquad \text{(}\textit{Spec}\text{. Eq. H3-2a)}$$

$$= \frac{1.23(29{,}000\text{ ksi})}{\sqrt{\dfrac{(14\text{ ft})(12\text{ in./ft})}{5.00\text{ in.}}}(21.5)^{5/4}}$$

$$= 133\text{ ksi}$$

and

$$F_{cr} = \frac{0.60E}{\left(\dfrac{D}{t}\right)^{3/2}} \qquad \text{(}\textit{Spec}\text{. Eq. H3-2b)}$$

$$= \frac{0.60(29{,}000\text{ ksi})}{(21.5)^{3/2}}$$

$$= 175\text{ ksi}$$

However, F_{cr} shall not exceed the following:

$$0.6F_y = 0.6(46\text{ ksi})$$
$$= 27.6\text{ ksi}$$

Therefore, $F_{cr} = 27.6$ ksi.

The nominal torsional strength is:

$$\begin{aligned} T_n &= F_{cr}C \qquad \text{(Spec. Eq. H3-1)} \\ &= (27.6\text{ ksi})(7.95\text{ in.}^3) \\ &= 219\text{ kip-in.} \end{aligned}$$

From AISC *Specification* Section H3.1, the available torsional strength is:

LRFD	ASD
$\phi_T = 0.90$ $\phi_T T_n = 0.90(219\text{ kip-in.})$ $= 197\text{ kip-in.}$	$\Omega_T = 1.67$ $\dfrac{T_n}{\Omega_T} = \dfrac{219\text{ kip-in.}}{1.67}$ $= 131\text{ kip-in.}$

Note: For more complete guidance on designing for torsion, see AISC Design Guide 9, *Torsional Analysis of Structural Steel Members* (Seaburg and Carter, 1997).

EXAMPLE H.5C RECTANGULAR HSS COMBINED TORSIONAL AND FLEXURAL STRENGTH

Given:

Verify the strength of an ASTM A500, Grade C, HSS6×4×¼ loaded as shown. The beam is simply supported and is torsionally fixed at the ends. Bending is about the strong axis.

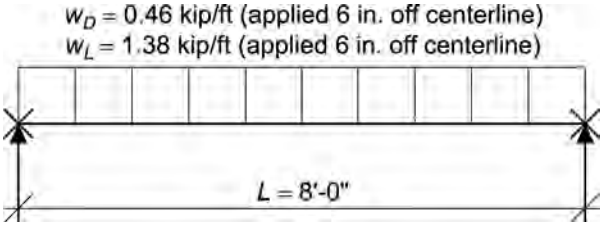


Fig. H.5C. Beam loading and bracing diagram.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500 Grade C
F_y = 50 ksi
F_u = 62 ksi

From AISC *Manual* Table 1-11, the geometric properties are as follows:

HSS6×4×¼
t = 0.233 in.
A_g = 4.30 in.^{2}
b/t = 14.2
h/t = 22.8
r_y = 1.61 in.
Z_x = 8.53 in.^{3}
J = 23.6 in.^{4}

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$w_u = 1.2(0.46\text{ kip/ft}) + 1.6(1.38\text{ kip/ft})$ $= 2.76\text{ kip/ft}$	$w_a = 0.46\text{ kip/ft} + 1.38\text{ kip/ft}$ $= 1.84\text{ kip/ft}$

Calculate the maximum shear (at the supports) using AISC *Manual* Table 3-23, Case 1.

LRFD	ASD
$V_r = V_u$ $= \dfrac{w_u L}{2}$ $= \dfrac{(2.76\text{ kip/ft})(8\text{ ft})}{2}$ $= 11.0\text{ kips}$	$V_r = V_a$ $= \dfrac{w_a L}{2}$ $= \dfrac{(1.84\text{ kip/ft})(8\text{ ft})}{2}$ $= 7.36\text{ kips}$

Calculate the maximum torsion (at the supports).

LRFD	ASD
$T_r = T_u$ $= \dfrac{w_u L e}{2}$ $= \dfrac{(2.76 \text{ kip/ft})(8 \text{ ft})(6 \text{ in.})}{2}$ $= 66.2 \text{ kip-in.}$	$T_r = T_a$ $= \dfrac{w_a L e}{2}$ $= \dfrac{(1.84 \text{ kip/ft})(8 \text{ ft})(6 \text{ in.})}{2}$ $= 44.2 \text{ kip-in.}$

Available Shear Strength

Determine the available shear strength from AISC *Specification* Section G4. Using the provisions given in AISC *Specification* Section B4.1b(d), determine the web depth, *d*, as follows:

$$\begin{aligned} h &= 6.00 \text{ in.} - 3(0.233 \text{ in.}) \\ &= 5.30 \text{ in.} \end{aligned}$$

From AISC *Specification* Section G4:

$$\begin{aligned} A_w &= 2ht \\ &= 2(5.30 \text{ in.})(0.233 \text{ in.}) \\ &= 2.47 \text{ in.}^2 \end{aligned}$$

$$k_v = 5$$

The web shear buckling coefficient is determined from AISC *Specification* Section G2.2.

$$\begin{aligned} 1.10\sqrt{\frac{k_v E}{F_y}} &= 1.10\sqrt{\frac{5(29{,}000 \text{ ksi})}{50 \text{ ksi}}} \\ &= 59.2 > 22.8; \text{ therefore use AISC } \textit{Specification} \text{ Section G2.2(b)(i)} \end{aligned}$$

$$C_{v2} = 1.0 \qquad \text{(}\textit{Spec}\text{. Eq. G2-9)}$$

The nominal shear strength from AISC *Specification* Section G4 is:

$$\begin{aligned} V_n &= 0.6 F_y A_w C_2 \qquad \text{(}\textit{Spec}\text{. Eq. G4-1)} \\ &= 0.6(50 \text{ ksi})(2.47 \text{ in.}^2)(1.0) \\ &= 74.1 \text{ kips} \end{aligned}$$

From AISC *Specification* Section G1, the available shear strength is:

LRFD	ASD
$\phi_v = 0.90$ $V_c = \phi_v V_n$ $= 0.90(74.1\text{ kips})$ $= 66.7\text{ kips}$	$\Omega_v = 1.67$ $V_c = \dfrac{V_n}{\Omega_v}$ $= \dfrac{74.1\text{ kips}}{1.67}$ $= 44.4\text{ kips}$

Available Flexural Strength

The available flexural strength is determined from AISC *Specification* Section F7 for rectangular HSS. For the limit state of flexural yielding, the nominal flexural strength is:

$$\begin{aligned} M_n &= M_p \\ &= F_y Z_x \\ &= (50\text{ ksi})(8.53\text{ in.}^3) \\ &= 427\text{ kip-in.} \end{aligned} \qquad \text{(\textit{Spec}. Eq. F7-1)}$$

Determine if the limit state of flange local buckling applies as follows:

$$\begin{aligned} \lambda &= \frac{b}{t} \\ &= 14.2 \end{aligned}$$

Determine the flange compact slenderness limit from AISC *Specification* Table B4.1b, Case 17.

$$\begin{aligned} \lambda_p &= 1.12\sqrt{\frac{E}{F_y}} \\ &= 1.12\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} \\ &= 27.0 \end{aligned}$$

$\lambda < \lambda_p$; therefore, the flange is compact and the flange local buckling limit state does not apply

Determine if the limit state of web local buckling applies as follows:

$$\begin{aligned} \lambda &= \frac{h}{t} \\ &= 22.8 \end{aligned}$$

Determine the web compact slenderness limit from AISC *Specification* Table B4.1b, Case 19.

$$\begin{aligned} \lambda_p &= 2.42\sqrt{\frac{E}{F_y}} \\ &= 2.42\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} \\ &= 58.3 \end{aligned}$$

$\lambda < \lambda_p$; therefore, the web is compact and the web local buckling limit state does not apply

Determine if lateral-torsional buckling applies as follows:

$$L_p = 0.13Er_y \frac{\sqrt{JA_g}}{M_p} \qquad \text{(}\textit{Spec}\text{. Eq. F7-12)}$$

$$= 0.13(29{,}000 \text{ ksi})(1.61 \text{ in.})\frac{\sqrt{\left(23.6 \text{ in.}^4\right)\left(4.30 \text{ in.}^2\right)}}{427 \text{ kip-in.}}$$

$$= 143 \text{ in. or } 11.9 \text{ ft}$$

Since L_b = 8 ft < L_p = 11.9 ft, lateral-torsional buckling is not applicable and M_n = 427 kip-in., controlled by the flexural yielding limit state. From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $M_c = \phi_b M_n$ $= 0.90(427 \text{ kip-in.})$ $= 384 \text{ kip-in.}$	$\Omega_b = 1.67$ $M_c = \dfrac{M_n}{\Omega_b}$ $= \dfrac{427 \text{ kip-in.}}{1.67}$ $= 256 \text{ kip-in.}$

From Example H.5A, the available torsional strength is:

LRFD	ASD
$T_c = \phi_T T_n$ $= 273 \text{ kip-in.}$	$T_c = \dfrac{T_n}{\Omega_T}$ $= 181 \text{ kip-in.}$

Using AISC *Specification* Section H3.2, check combined strength at several locations where $T_r > 0.2T_c$. First check at the supports, which is the point of maximum shear and torsion:

LRFD	ASD
$\dfrac{T_r}{T_c} = \dfrac{66.2 \text{ kip-in.}}{273 \text{ kip-in.}}$ $= 0.242 > 0.2$ Therefore, use AISC *Specification* Equation H3-6: $\left(\dfrac{P_r}{P_c} + \dfrac{M_r}{M_c}\right) + \left(\dfrac{V_r}{V_c} + \dfrac{T_r}{T_c}\right)^2 \le 1.0$ (*Spec* Eq. H3-6) $= (0+0) + \left(\dfrac{11.0 \text{ kips}}{66.7 \text{ kips}} + \dfrac{66.2 \text{ kip-in.}}{273 \text{ kip-in.}}\right)^2$ $= 0.166 < 1.0$ **o.k.**	$\dfrac{T_r}{T_c} = \dfrac{44.2 \text{ kip-in.}}{181 \text{ kip-in.}}$ $= 0.244 > 0.2$ Therefore, use AISC *Specification* Equation H3-6: $\left(\dfrac{P_r}{P_c} + \dfrac{M_r}{M_c}\right) + \left(\dfrac{V_r}{V_c} + \dfrac{T_r}{T_c}\right)^2 \le 1.0$ (*Spec* Eq. H3-6) $= (0+0) + \left(\dfrac{7.36 \text{ kips}}{44.4 \text{ kips}} + \dfrac{44.2 \text{ kip-in.}}{181 \text{ kip-in.}}\right)^2$ $= 0.168 < 1.0$ **o.k.**

Check the combined strength near the location where T_r = 0.2T_c. This is the location with the largest bending moment required to be considered in the interaction. Calculate the shear and moment at this location, x.

LRFD	ASD
$\dfrac{T_r}{T_c} = 0.20$ Therefore at x: $T_r = 0.20(273\text{ kip-in.})$ $= 54.6\text{ kip-in.}$ $x = \dfrac{(T_r\text{ at support}) - (T_r\text{ at }x)}{w_u e}$ $= \dfrac{66.2\text{ kip-in.} - 54.6\text{ kip-in.}}{(2.76\text{ kip/ft})(6\text{ in.})}$ $= 0.700\text{ ft}$ $V_r = 11.0\text{ kips} - (0.700\text{ ft})(2.76\text{ kip/ft})$ $= 9.07\text{ kips}$ $M_r = \dfrac{w_u x}{2}(l - x)$ $= \dfrac{(2.76\text{ kip/ft})(0.700\text{ ft})}{2}(8\text{ ft} - 0.700\text{ ft})$ $= 7.05\text{ kip-ft or }84.6\text{ kip-in.}$ $\left(\dfrac{P_r}{P_c}+\dfrac{M_r}{M_c}\right)+\left(\dfrac{V_r}{V_c}+\dfrac{T_r}{T_c}\right)^2 \le 1.0$ (*Spec* Eq. H3-6) $= \left(0+\dfrac{84.6\text{ kip-in.}}{384\text{ kip-in.}}\right)+\left(\dfrac{9.07\text{ kips}}{66.7\text{ kips}}+0.20\right)^2$ $= 0.333 < 1.0$ **o.k.**	$\dfrac{T_r}{T_c} = 0.20$ Therefore at x: $T_r = 0.20(181\text{ kip-in.})$ $= 36.2\text{ kip-in.}$ $x = \dfrac{(T_r\text{ at support}) - (T_r\text{ at }x)}{w_a e}$ $= \dfrac{44.2\text{ kip-in.} - 36.2\text{ kip-in.}}{(1.84\text{ kip/ft})(6\text{ in.})}$ $= 0.725\text{ ft}$ $V_r = 7.36\text{ kips} - (0.725\text{ ft})(1.84\text{ kips/ft})$ $= 6.03\text{ kips}$ $M_r = \dfrac{w_a x}{2}(l - x)$ $= \dfrac{(1.84\text{ kip/ft})(0.725\text{ ft})}{2}(8\text{ ft} - 0.725\text{ ft})$ $= 4.85\text{ kip-ft or }58.2\text{ kip-in.}$ $\left(\dfrac{P_r}{P_c}+\dfrac{M_r}{M_c}\right)+\left(\dfrac{V_r}{V_c}+\dfrac{T_r}{T_c}\right)^2 \le 1.0$ (*Spec* Eq. H3-6) $= \left(0+\dfrac{58.2\text{ kip-in.}}{256\text{ kip-in.}}\right)+\left(\dfrac{6.03\text{ kips}}{44.4\text{ kips}}+0.20\right)^2$ $= 0.340 < 1.0$ **o.k.**

Note: The remainder of the beam, where $T_r \le 0.2T_c$, must also be checked to determine if the strength without torsion controls over the interaction with torsion.

EXAMPLE H.6 W-SHAPE TORSIONAL STRENGTH

Given:

As shown in Figure H.6-1, an ASTM A992 W10×49 spans 15 ft and supports concentrated loads at midspan that act at a 6-in. eccentricity with respect to the shear center. Determine the stresses on the cross section, the adequacy of the section to support the loads, and the maximum rotation.

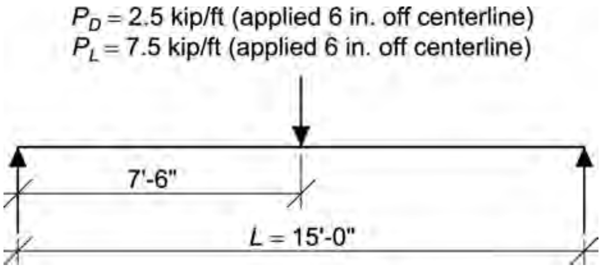


Fig. H.6-1. Beam loading diagram.

The end conditions are assumed to be flexurally pinned and unrestrained for warping torsion. The eccentric load can be resolved into a torsional moment and a load applied through the shear center.

A similar design example appears in AISC Design Guide 9, *Torsional Analysis of Structural Steel Members* (Seaburg and Carter, 1997).

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W10×49
t_w = 0.340 in.
t_f = 0.560 in.
I_x = 272 in.^{4}
S_x = 54.6 in.^{3}
Z_x = 60.4 in.^{3}
J = 1.39 in.^{4}
C_w = 2,070 in.^{6}

From the AISC Shapes Database, the additional torsional properties are as follows:

W10×49
S_{w1} = 33.0 in.^{4}
W_{no} = 23.6 in.^{2}
Q_f = 12.8 in.^{3}
Q_w = 29.8 in.^{3}

From AISC Design Guide 9, the torsional property, *a*, is calculated as follows:

$$a = \sqrt{\frac{EC_w}{GJ}} \qquad \text{(Design Guide 9, Eq. 3.6)}$$

$$= \sqrt{\frac{(29{,}000 \text{ ksi})(2{,}070 \text{ in.}^6)}{(11{,}200 \text{ ksi})(1.39 \text{ in.}^4)}}$$

$$= 62.1 \text{ in.}$$

From ASCE/SEI 7, Chapter 2, and AISC *Manual* Table 3-23, Case 7, the required strengths are:

LRFD	ASD
$P_u = 1.2(2.5 \text{ kips}) + 1.6(7.5 \text{ kips})$ $= 15.0 \text{ kips}$	$P_a = 2.5 \text{ kips} + 7.5 \text{ kips}$ $= 10.0 \text{ kips}$
$V_u = \dfrac{P_u}{2}$ $= \dfrac{15.0 \text{ kips}}{2}$ $= 7.50 \text{ kips}$	$V_a = \dfrac{P_a}{2}$ $= \dfrac{10.0 \text{ kips}}{2}$ $= 5.00 \text{ kips}$
$M_u = \dfrac{P_u L}{4}$ $= \dfrac{(15.0 \text{ kips})(15 \text{ ft})(12 \text{ in./ft})}{4}$ $= 675 \text{ kip-in.}$	$M_a = \dfrac{P_a L}{4}$ $= \dfrac{(10.0 \text{ kips})(15 \text{ ft})(12 \text{ in./ft})}{4}$ $= 450 \text{ kip-in.}$
$T_u = P_u e$ $= (15.0 \text{ kips})(6 \text{ in.})$ $= 90.0 \text{ kip-in.}$	$T_a = P_a e$ $= (10.0 \text{ kips})(6 \text{ in.})$ $= 60.0 \text{ kip-in.}$

Normal and Shear Stresses from Flexure

The normal and shear stresses from flexure are determined from AISC Design Guide 9, as follows:

LRFD	ASD
$\sigma_{ub} = \dfrac{M_u}{S_x}$ (from Design Guide 9, Eq. 4.5) $= \dfrac{675 \text{ kip-in.}}{54.6 \text{ in.}^3}$ $= 12.4 \text{ ksi}$ (compression at top, tension at bottom)	$\sigma_{ab} = \dfrac{M_a}{S_x}$ (from Design Guide 9, Eq. 4.5) $= \dfrac{450 \text{ kip-in.}}{54.6 \text{ in.}^3}$ $= 8.24 \text{ ksi}$ (compression at top, tension at bottom)
$\tau_{ub\ web} = \dfrac{V_u Q_w}{I_x t_w}$ (from Design Guide 9, Eq. 4.6) $= \dfrac{(7.50 \text{ kips})(29.8 \text{ in.}^3)}{(272 \text{ in.}^4)(0.340 \text{ in.})}$ $= 2.42 \text{ ksi}$	$\tau_{ab\ web} = \dfrac{V_a Q_w}{I_x t_w}$ (from Design Guide 9, Eq. 4.6) $= \dfrac{(5.00 \text{ kips})(29.8 \text{ in.}^3)}{(272 \text{ in.}^4)(0.340 \text{ in.})}$ $= 1.61 \text{ ksi}$

LRFD	ASD
$\tau_{ub\ flange}=\frac{V_u Q_f}{I_x t_f}$ (from Design Guide 9, Eq. 4.6) $=\frac{(7.50\text{ kips})(12.8\text{ in.}^3)}{(272\text{ in.}^4)(0.560\text{ in.})}$ $=0.630\text{ ksi}$	$\tau_{ab\ flange}=\frac{V_a Q_f}{I_x t_f}$ (from Design Guide 9, Eq. 4.6) $=\frac{(5.00\text{ kips})(12.8\text{ in.}^3)}{(272\text{ in.}^4)(0.560\text{ in.})}$ $=0.420\text{ ksi}$

Torsional Stresses

The following functions are taken from AISC Design Guide 9, Appendix B, Case 3, with α = 0.5 for the torsional load applied at midspan.

$$\frac{L}{a}=\frac{(15\text{ ft})(12\text{ in./ft})}{62.1\text{ in.}}$$
$$=2.90$$

Using the graphs in AISC Design Guide 9, Appendix B, select values for θ, θ', θ'' and θ'''.

At midspan (z/l = 0.5):

For θ: $\theta\times\left(\frac{GJ}{T_r}\right)\left(\frac{1}{l}\right)=+0.09$ Solve for: $\theta=+0.09\frac{T_r l}{GJ}$

For θ': $\theta'\times\left(\frac{GJ}{T_r}\right)=0$ Therefore: $\theta'=0$

For θ'': $\theta''\times\left(\frac{GJ}{T_r}\right)a=-0.44$ Solve for: $\theta''=-0.44\frac{T_r}{GJa}$

For θ''': $\theta'''\times\left(\frac{GJ}{T_r}\right)a^2=-0.50$ Solve for: $\theta'''=-0.50\frac{T_r}{GJa^2}$

At the support (z/l = 0):

For θ: $\theta\times\left(\frac{GJ}{T_r}\right)\left(\frac{1}{l}\right)=0$ Therefore: $\theta=0$

For θ': $\theta'\times\left(\frac{GJ}{T_r}\right)=+0.28$ Solve for: $\theta'=+0.28\frac{T_r}{GJ}$

For θ'': $\theta''\times\left(\frac{GJ}{T_r}\right)a=0$ Therefore: $\theta''=0$

For θ''': $\theta'''\times\left(\frac{GJ}{T_r}\right)a^2=-0.22$ Solve for: $\theta'''=-0.22\frac{T_r}{GJa^2}$

In the preceding calculations, note that the applied torque is negative based on the sign convention used in the AISC Design Guide 9 graphs.

Calculate T_r/GJ as follows:

LRFD	ASD
$\dfrac{T_u}{GJ} = \dfrac{-90.0\text{ kip-in.}}{(11{,}200\text{ ksi})(1.39\text{ in.}^4)}$ $= -5.78\times10^{-3}\text{ rad/in.}$	$\dfrac{T_a}{GJ} = \dfrac{-60.0\text{ kip-in.}}{(11{,}200\text{ ksi})(1.39\text{ in.}^4)}$ $= -3.85\times10^{-3}\text{ rad/in.}$

Shear Stresses Due to Pure Torsion

The shear stresses due to pure torsion are determined from AISC Design Guide 9 as follows:

$$\tau_t = Gt\theta' \qquad \text{(Design Guide 9, Eq. 4.1)}$$

LRFD	ASD
At midspan: $\theta' = 0;$ therefore $\tau_{ut} = 0$ At the support, for the web: $\tau_{ut} = (11{,}200\text{ ksi})(0.340\text{ in.})(0.28)\left(\dfrac{-5.78\text{ rad}}{10^3\text{ in.}}\right)$ $= -6.16\text{ ksi}$ At the support, for the flange: $\tau_{ut} = (11{,}200\text{ ksi})(0.560\text{ in.})(0.28)\left(\dfrac{-5.78\text{ rad}}{10^3\text{ in.}}\right)$ $= -10.2\text{ ksi}$	At midspan: $\theta' = 0;$ therefore $\tau_{at} = 0$ At the support, for the web: $\tau_{at} = (11{,}200\text{ ksi})(0.340\text{ in.})(0.28)\left(\dfrac{-3.85\text{ rad}}{10^3\text{ in.}}\right)$ $= -4.11\text{ ksi}$ At the support, for the flange: $\tau_{at} = (11{,}200\text{ ksi})(0.560\text{ in.})(0.28)\left(\dfrac{-3.85\text{ rad}}{10^3\text{ in.}}\right)$ $= -6.76\text{ ksi}$

Shear Stresses Due to Warping

The shear stresses due to warping are determined from AISC Design Guide 9 as follows:

$$\tau_w = \frac{-ES_{w1}\theta'''}{t_f} \qquad \text{(Design Guide 9, Eq. 4.2a)}$$

LRFD	ASD
At midspan: $\tau_{uw} = \dfrac{(-29{,}000\text{ ksi})(33.0\text{ in.}^4)}{0.560\text{ in.}}\left[\dfrac{-0.50(-5.78\text{ rad})}{(62.1\text{ in.})^2(10^3\text{ in.})}\right]$ $= -1.28\text{ ksi}$ At the support: $\tau_{uw} = \dfrac{(-29{,}000\text{ ksi})(33.0\text{ in.}^4)}{0.560\text{ in.}}\left[\dfrac{-0.22(-5.78\text{ rad})}{(62.1\text{ in.})^2(10^3\text{ in.})}\right]$ $= -0.563\text{ ksi}$	At midspan: $\tau_{aw} = \dfrac{(-29{,}000\text{ ksi})(33.0\text{ in.}^4)}{0.560\text{ in.}}\left[\dfrac{-0.50(-3.85\text{ rad})}{(62.1\text{ in.})^2(10^3\text{ in.})}\right]$ $= -0.853\text{ ksi}$ At the support: $\tau_{aw} = \dfrac{(-29{,}000\text{ ksi})(33.0\text{ in.}^4)}{0.560\text{ in.}}\left[\dfrac{-0.22(-3.85\text{ rad})}{(62.1\text{ in.})^2(10^3\text{ in.})}\right]$ $= -0.375\text{ ksi}$

Normal Stresses Due to Warping

The normal stresses due to warping are determined from AISC Design Guide 9 as follows:

$$\sigma_w = EW_{no}\theta'' \qquad \text{(Design Guide 9, Eq. 4.3a)}$$

LRFD	ASD
At midspan: $\sigma_{uw} = (29{,}000\text{ ksi})(23.6\text{ in.}^2)\left[\dfrac{-0.44(-5.78\text{ rad})}{(62.1\text{ in.})(10^3\text{ in.})}\right]$ $= 28.0\text{ ksi}$ At the support: Because $\theta'' = 0$, $\sigma_{uw} = 0$.	At midspan: $\sigma_{aw} = (29{,}000\text{ ksi})(23.6\text{ in.}^2)\left[\dfrac{-0.44(-3.85\text{ rad})}{(62.1\text{ in.})(10^3\text{ in.})}\right]$ $= 18.7\text{ ksi}$ At the support: Because $\theta'' = 0$, $\sigma_{aw} = 0$.

Combined Stresses

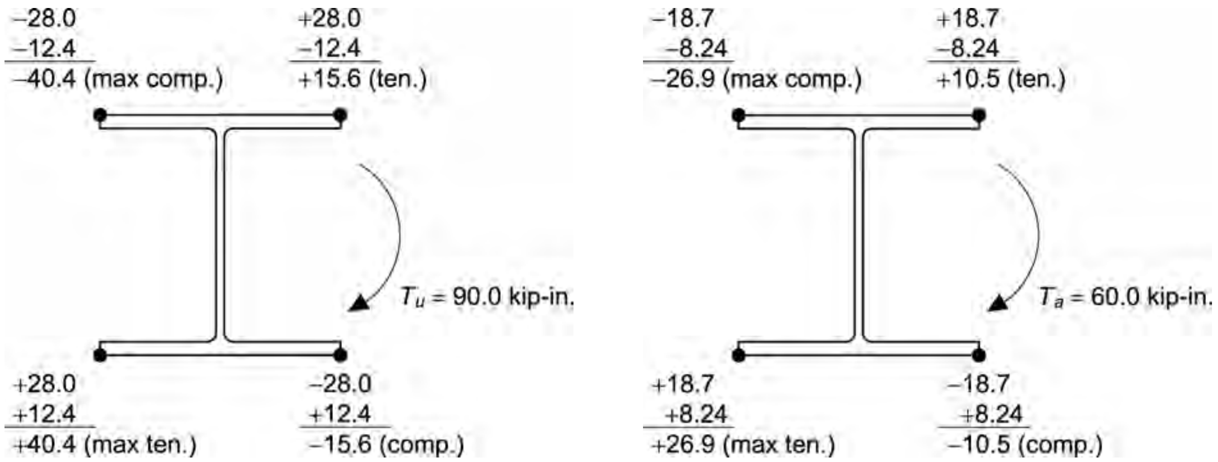
The stresses are summarized in Tables H.6-1A and H.6-1B and shown in Figure H.6-2.

Table H.6-1A
Summary of Stresses Due to Flexure and Torsion (LRFD), ksi

Location	**Normal Stress**			**Shear Stress**			
	σ_{uw}	σ_{ub}	f_{un}	τ_{ut}	τ_{uw}	τ_{ub}	f_{uv}
Midspan							
Flange	±28.0	±12.4	±40.4	0	–1.28	±0.630	–1.91
Web	–	–	–	0	–	±2.42	±2.42
Support							
Flange	0	0	0	–10.2	–0.563	±0.630	–11.4
Web	–	–	–	–6.16	–	±2.42	–8.58
Maximum			±40.4				–11.4

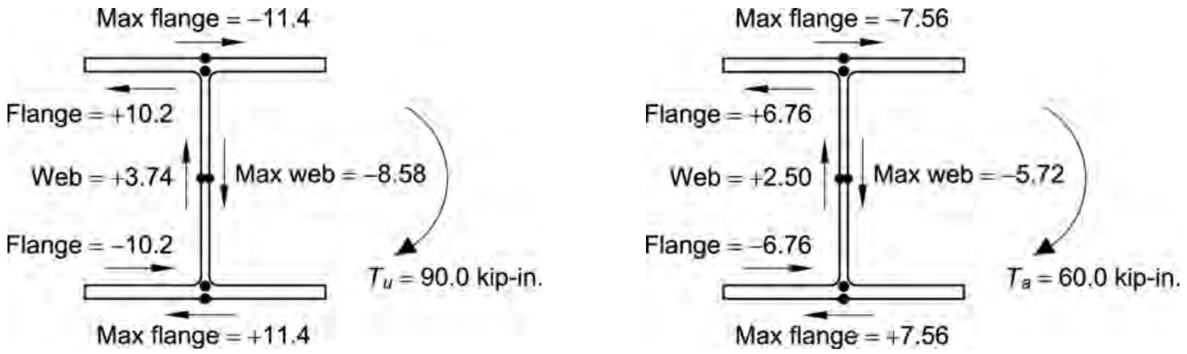
Table H.6-1B
Summary of Stresses Due to Flexure and Torsion (ASD), ksi

Location	**Normal Stress**			**Shear Stress**			
	σ_{aw}	σ_{ab}	f_{an}	τ_{at}	τ_{aw}	τ_{ab}	f_{av}
Midspan							
Flange	±18.7	±8.24	±26.9	0	–0.853	±0.420	–1.27
Web	–	–	–	0	–	±1.61	±1.61
Support							
Flange	0	0	0	–6.76	–0.375	±0.420	–7.56
Web	–	–	–	–4.11	–	±1.61	–5.72
Maximum			±26.9				–7.56



(a) Normal stresses due to flexure and torsion at midspan—LRFD

(b) Normal stresses due to flexure and torsion at midspan—ASD



(c) Shear stresses due to flexure and torsion at support—LRFD

(d) Shear stresses due to flexure and torsion at support—ASD

Fig. H.6-2. Stresses due to flexure and torsion.

LRFD	ASD
The maximum normal stress due to flexure and torsion occurs at the edge of the flange at midspan and is equal to 40.4 ksi. The maximum shear stress due to flexure and torsion occurs in the middle of the flange at the support and is equal to 11.4 ksi.	The maximum normal stress due to flexure and torsion occurs at the edge of the flange at midspan and is equal to 26.9 ksi. The maximum shear stress due to flexure and torsion occurs in the middle of the flange at the support and is equal to 7.56 ksi.

Available Torsional Strength

The available torsional strength is the lowest value determined for the limit states of yielding under normal stress, shear yielding under shear stress, or buckling in accordance with AISC *Specification* Section H3.3. The nominal torsional strength due to the limit states of yielding under normal stress and shear yielding under shear stress are compared to the applicable buckling limit states.

Buckling

For the buckling limit state, lateral-torsional buckling and local buckling must be evaluated. The nominal torsional strength due to the limit state of lateral-torsional buckling is determined as follows.

$C_b = 1.32$ from AISC *Manual* Table 3-1.

Compute F_n for a W10×49 using values from AISC *Manual* Table 3-10 with $L_b = 15$ ft and $C_b = 1.0$.

LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
$\phi_b M_n = 204\text{ kips}$	$\dfrac{M_n}{\Omega_b} = 136\text{ kip-ft}$
$F_n = F_{cr}$ (*Spec*. Eq. H3-9) $= C_b\left(\dfrac{M_n}{S_x}\right)$ $= 1.32\left[\dfrac{(204\text{ kip-ft})(12\text{ in./ft})}{0.90(54.6\text{ in.}^3)}\right]$ $= 65.8\text{ ksi}$	$F_n = F_{cr}$ (*Spec*. Eq. H3-9) $= C_b\left(\dfrac{M_n}{S_x}\right)$ $= 1.32\left[\dfrac{1.67(136\text{ kip-ft})(12\text{ in./ft})}{(54.6\text{ in.}^3)}\right]$ $= 65.9\text{ ksi}$

The limit state of local buckling does not apply because a W10×49 is compact in flexure per the user note in AISC *Specification* Section F2.

Yielding Under Normal Stress

The nominal torsional strength due to the limit state of yielding under normal stress is determined as follows:

$$
\begin{aligned}
F_n &= F_y \qquad \text{(}\textit{Spec}\text{. Eq. H3-7)}\\
&= 50\text{ ksi}
\end{aligned}
$$

Therefore, the limit state of yielding under normal stress controls over buckling. The available torsional strength for yielding under normal stress is determined as follows, from AISC *Specification* Section H3:

LRFD	ASD
$\phi_T = 0.90$	$\Omega_T = 1.67$
$\phi_T F_n = 0.90(50\text{ ksi})$ $= 45.0\text{ ksi} > 40.4\text{ ksi}$ **o.k.**	$\dfrac{F_n}{\Omega_T} = \dfrac{50\text{ ksi}}{1.67}$ $= 29.9\text{ ksi} > 26.9\text{ ksi}$ **o.k.**

Shear Yielding Under Shear Stress

The nominal torsional strength due to the limit state of shear yielding under shear stress is:

$$
\begin{aligned}
F_n &= 0.6F_y \qquad \text{(}\textit{Spec}\text{. Eq. H3-8)}\\
&= 0.6(50\text{ ksi})\\
&= 30.0\text{ ksi}
\end{aligned}
$$

The limit state of shear yielding under shear stress controls over buckling. The available torsional strength for shear yielding under shear stress is determined as follows, from AISC *Specification* Section H3:

LRFD	ASD
$\phi_T = 0.90$ $\phi_T F_n = 0.90(30 \text{ ksi})$ $= 27.0 \text{ ksi} > 11.4 \text{ ksi}$ **o.k.**	$\Omega_T = 1.67$ $\dfrac{F_n}{\Omega_T} = \dfrac{30 \text{ ksi}}{1.67}$ $= 18.0 \text{ ksi} > 7.56 \text{ ksi}$ **o.k.**

Maximum Rotation at Service Load

The maximum rotation occurs at midspan. The service load torque is:

$$\begin{aligned} T &= Pe \\ &= -(2.50 \text{ kips} + 7.50 \text{ kips})(6 \text{ in.}) \\ &= -60.0 \text{ kip-in.} \end{aligned}$$

As determined previously from AISC Design Guide 9, Appendix B, Case 3 with $\alpha = 0.5$, the maximum rotation is:

$$\begin{aligned} \theta &= +0.09\frac{Tl}{GJ} \\ &= \frac{0.09(-60.0 \text{ kip-in.})(15 \text{ ft})(12 \text{ in./ft})}{(11{,}200 \text{ ksi})(1.39 \text{ in.}^4)} \\ &= -0.0624 \text{ rad or } -3.58^\circ \end{aligned}$$

See AISC Design Guide 9, *Torsional Analysis of Structural Steel Members*, for additional guidance.

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CHAPTER H DESIGN EXAMPLE REFERENCES

Seaburg, P.A. and Carter, C.J. (1997), *Torsional Analysis of Structural Steel Members*, Design Guide 9, AISC, Chicago, IL.

Chapter I
Design of Composite Members

I1. GENERAL PROVISIONS

Design, detailing, and material properties related to the concrete and steel reinforcing portions of composite members are governed by ACI 318 (ACI 318, 2014) as modified with composite-specific provisions by the AISC *Specification*.

The available strength of composite sections may be calculated by one of four methods: the plastic stress distribution method, the strain-compatibility method, the elastic stress distribution method, or the effective stress-strain method. The composite design tables in Part IV of this document are based on the plastic stress distribution method.

Filled composite sections are classified for local buckling according to the slenderness of the compression steel elements as illustrated in AISC *Specification* Tables I1.1a and I1.1b, and Examples I.4, I.6 and I.7. Local buckling effects do not need to be considered for encased composite members.

Terminology used within the Examples for filled composite section geometry is illustrated in Figure I-1.

I2. AXIAL FORCE

The available compressive strength of a composite member is based on a summation of the strengths of all of the components of the column with reductions applied for member slenderness and local buckling effects where applicable.

For tension members, the concrete tensile strength is ignored and only the strength of the steel member and properly connected reinforcing is permitted to be used in the calculation of available tensile strength.

The available compressive strengths for filled composite sections are given in Part IV of this document and reflect the requirements given in AISC *Specification* Sections I1.4 and I2.2. The design of filled composite compression and tension members is presented in Examples I.4 and I.5, respectively.

The design of encased composite compression and tension members is presented in Examples I.9 and I.10, respectively. There are no tables in the AISC *Manual* for the design of these members.

Note that the AISC *Specification* stipulates that the available compressive strength need not be less than that specified for the bare steel member.

I3. FLEXURE

The design of typical composite beams with steel anchors is illustrated in Examples I.1 and I.2. AISC *Manual* Table 3-19 provides available flexural strengths for composite W-shape beams, Table 3-20 provides lower-bound moments of inertia for plastic composite sections, and Table 3-21 provides shear strengths of steel headed stud anchors utilized for composite action in composite beams.

The design of filled composite members for flexure is illustrated within Examples I.6 and I.7, and the design of encased composite members for flexure is illustrated within Example I.11.

I4. SHEAR

For composite beams with formed steel deck, the available shear strength is based upon the properties of the steel section alone in accordance with AISC *Specification* Chapter G as illustrated in Examples I.1 and I.2.

For filled and encased composite members, either the shear strength of the steel section alone, the steel section plus the reinforcing steel, or the reinforced concrete alone are permitted to be used in the calculation of available shear strength. The calculation of shear strength for filled composite members is illustrated within Examples I.6 and I.7 and for encased composite members within Example I.11.

I5. COMBINED FLEXURE AND AXIAL FORCE

Design for combined axial force and flexure may be accomplished using either the strain compatibility method or the plastic-distribution method. Several different procedures for employing the plastic-distribution method are outlined in the Commentary, and each of these procedures is demonstrated for filled composite members in Example I.6 and for encased composite members in Example I.11. Interaction calculations for noncompact and slender filled composite members are illustrated in Example I.7.

To assist in developing the interaction curves illustrated within the design examples, a series of equations is provided in AISC *Manual* Part 6, Tables 6-3a, 6-3b, 6-4 and 6-5. These equations define selected points on the interaction curve, without consideration of slenderness effects. Specific cases are outlined and the applicability of the equations to a cross section that differs should be carefully considered. As an example, the equations in AISC *Manual* Table 6-3a are appropriate for the case of side bars located at the centerline, but not for other side bar locations. In contrast, these equations are appropriate for any amount of reinforcing at the extreme reinforcing bar location. In AISC *Manual* Table 6-3b the equations are appropriate only for the case of four reinforcing bars at the corners of the encased section. When design cases deviate from those presented the appropriate interaction equations can be derived from first principles.

I6. LOAD TRANSFER

The AISC *Specification* provides several requirements to ensure that the concrete and steel portions of the section act together. These requirements address both force allocation—how much of the applied loads are resisted by the steel versus the reinforced concrete; and force transfer mechanisms—how the force is transferred between the two materials. These requirements are illustrated in Example I.3 for filled composite members and Example I.8 for encased composite members.

I7. COMPOSITE DIAPHRAGMS AND COLLECTOR BEAMS

The Commentary provides guidance on design methodologies for both composite diaphragms and composite collector beams.

I8. STEEL ANCHORS

AISC *Specification* Section I8 addresses the strength of steel anchors in composite beams and in composite components. Examples I.1 and I.2 illustrates the design of composite beams with steel headed stud anchors.

The application of steel anchors in composite component provisions have strict limitations as summarized in the User Note provided at the beginning of AISC *Specification* Section I8.3. These provisions do not apply to typical composite beam designs nor do they apply to hybrid construction where the steel and concrete do not resist loads together via composite action such as in embed plates. The most common application for these provisions is for the transfer of longitudinal shear within the load introduction length of composite columns as demonstrated in Example I.8. The application of these provisions to an isolated anchor within an applicable composite system is illustrated in Example I.12.

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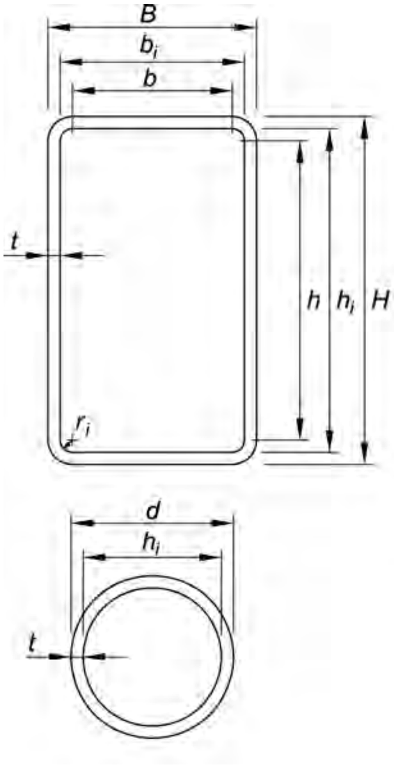

B = Overall width of section parallel to the axis of bending, in.
H = Overall height of section perpendicular to the axis of bending, in.
b = Width of stiffened compression element, in.
= $B - 3t$ per AISC *Specification* Section B4.1b(d)
b_i = Inside width of section, in.
= $B - 2t$
d = Outside diameter of round HSS, in.
h = Width of stiffened compression element, in.
= $H - 3t$ per AISC *Specification* Section B4.1b(d)
h_i = Inside diameter of round HSS, in.
= $d - 2t$
h_i = Inside height of section, in.
= $H - 2t$
r_i = $0.75t$ for b/t and h/t, in.
r_i = $1.0t$ for all area, section modulus, and moment of inertia calculations, in.
t = $0.93t_{nom}$, in.

Fig. I-1. Terminology used for filled members.

EXAMPLE I.1 COMPOSITE BEAM DESIGN

Given:

A typical bay of a composite floor system is illustrated in Figure I.1-1. Select an appropriate ASTM A992 W-shaped beam and determine the required number of ¾-in.-diameter steel headed stud anchors. The beam will not be shored during construction.

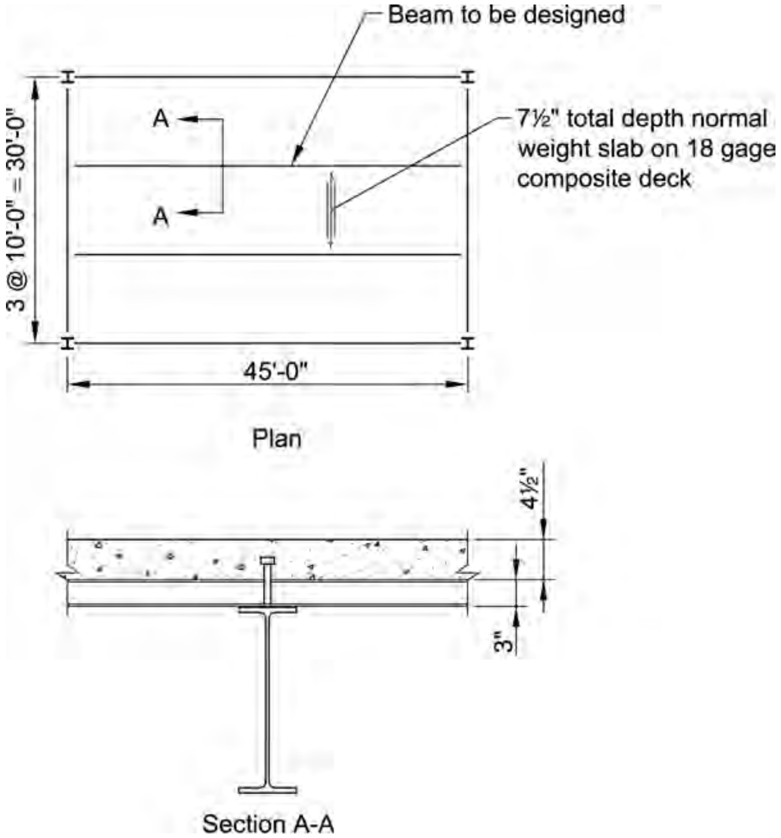

Fig. I.1-1. Composite bay and beam section.

To achieve a two-hour fire rating without the application of spray applied fire protection material to the composite deck, 4½ in. of normal weight (145 lb/ft^3) concrete will be placed above the top of the deck. The concrete has a specified compressive strength, f'_c = 4 ksi.

Applied loads are given in the following:

Dead Loads:
Pre-composite:
Slab = 75 lb/ft^2 (in accordance with metal deck manufacturer's data)
Self-weight = 5 lb/ft^2 (assumed uniform load to account for beam weight)

Composite (applied after composite action has been achieved):
Miscellaneous = 10 lb/ft^2 (HVAC, ceiling, floor covering, etc.)

Live Loads:
Pre-composite:
Construction = 25 lb/ft^2 (temporary loads during concrete placement)

Composite (applied after composite action has been achieved):
Non-reducible = 100 lb/ft^2 (assembly occupancy)

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Applied Loads

For slabs that are to be placed at a constant elevation, AISC Design Guide 3 (West and Fisher, 2003) recommends an additional 10% of the nominal slab weight be applied to account for concrete ponding due to deflections resulting from the wet weight of the concrete during placement. For the slab under consideration, this would result in an additional load of 8 lb/ft^2; however, for this design the slab will be placed at a constant thickness, and thus, no additional weight for concrete ponding is required.

For pre-composite construction live loading, 25 lb/ft^2 will be applied in accordance with recommendations from *Design Loads on Structures During Construction*, ASCE/SEI 37 (ASCE, 2014), for a light duty operational class that includes concrete transport and placement by hose and finishing with hand tools.

Composite Deck and Anchor Requirements

Check composite deck and anchor requirements stipulated in AISC *Specification* Sections I1.3, I3.2c and I8.

1. Concrete Strength: $3 \text{ ksi} \le f_c' \le 10 \text{ ksi}$ (for normal weight concrete) (*Spec.* Section I1.3)

   $f_c' = 4 \text{ ksi}$ **o.k.**

2. Rib height: $h_r \le 3 \text{ in.}$ (*Spec.* Section I3.2c)

   $h_r = 3 \text{ in.}$ **o.k.**

3. Average rib width: $w_r \ge 2 \text{ in.}$ (*Spec.* Section I3.2c)

   $w_r = 6 \text{ in.}$ (from deck manufacturer's literature) **o.k.**

4. Use steel headed stud anchors ¾ in. or less in diameter. (*Spec.* Section I8.1)

   Use ¾-in.-diameter steel anchors per problem statement. **o.k.**

5. Steel headed stud anchor diameter: $d_{sa} \le 2.5t_f$ (*Spec.* Section I8.1)

   In accordance with AISC *Specification* Section I8.1, this limit only applies if steel headed stud anchors are not welded to the flange directly over the web. The ¾-in.-diameter anchors will be placed in pairs transverse to the web in some locations, thus this limit must be satisfied. Select a beam size with a minimum flange thickness of 0.300 in., as determined in the following:

$$\begin{aligned} t_f &\ge \frac{d_{sa}}{2.5} \\ &\ge \frac{\text{¾ in.}}{2.5} \\ &\ge 0.300 \text{ in.} \end{aligned}$$

6. In accordance with AISC *Specification* I3.2c, steel headed stud anchors, after installation, shall extend not less than 1½ in. above the top of the steel deck. A minimum anchor length of 4½ in. is required to meet this requirement for 3 in. deep deck. From steel headed stud anchor manufacturer's data, a standard stock length of 4⅞ in. is selected. Using a ⅜-in. length reduction to account for burn off during anchor installation through the deck yields a final installed length of 4½ in.

7. Minimum length of stud anchors $= 4d_{sa}$ (*Spec.* Section I8.2)

   $4\tfrac{1}{2}\text{ in.} > 4\left(\tfrac{3}{4}\text{ in.}\right) = 3.00\text{ in.}$ **o.k.**

8. In accordance with AISC *Specification* Section I3.2c, there shall be at least ½ in. of specified concrete cover above the top of the headed stud anchors.

   As discussed in AISC *Specification* Commentary to Section I3.2c, it is advisable to provide greater than ½ in. minimum cover to assure anchors are not exposed in the final condition, particularly for intentionally cambered beams.

   $7\tfrac{1}{2}\text{ in.} - 4\tfrac{1}{2}\text{ in.} = 3.00\text{ in.} > \tfrac{1}{2}\text{ in.}$ **o.k.**

9. In accordance with AISC *Specification* Section I3.2c, slab thickness above steel deck shall not be less than 2 in.

   $4\tfrac{1}{2}\text{ in.} > 2\text{ in.}$ **o.k.**

Design for Pre-Composite Condition

Construction (Pre-Composite) Loads

The beam is uniformly loaded by its tributary width as follows:

$$\begin{aligned} w_D &= \left[(10\text{ ft})\left(75\text{ lb/ft}^2 + 5\text{ lb/ft}^2\right)\right](1\text{ kip/1,000 lb}) \\ &= 0.800\text{ kip/ft} \end{aligned}$$

$$\begin{aligned} w_L &= \left[(10\text{ ft})\left(25\text{ lb/ft}^2\right)\right](1\text{ kip/1,000 lb}) \\ &= 0.250\text{ kip/ft} \end{aligned}$$

Construction (Pre-Composite) Flexural Strength

From ASCE/SEI 7, Chapter 2, the required flexural strength is:

LRFD	ASD
$w_u = 1.2(0.800\text{ kip/ft}) + 1.6(0.250\text{ kip/ft})$ $= 1.36\text{ kip/ft}$ $M_u = \dfrac{w_u L^2}{8}$ $= \dfrac{(1.36\text{ kip/ft})(45\text{ ft})^2}{8}$ $= 344\text{ kip-ft}$	$w_a = 0.800\text{ kip/ft} + 0.250\text{ kip/ft}$ $= 1.05\text{ kip/ft}$ $M_a = \dfrac{w_a L^2}{8}$ $= \dfrac{(1.05\text{ kip/ft})(45\text{ ft})^2}{8}$ $= 266\text{ kip-ft}$

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Beam Selection

Assume that attachment of the deck perpendicular to the beam provides adequate bracing to the compression flange during construction, thus the beam can develop its full plastic moment capacity. The required plastic section modulus, Z_x, is determined as follows, from AISC *Specification* Equation F2-1:

LRFD	ASD
$\phi_b = 0.90$ $Z_{x,min} = \dfrac{M_u}{\phi_b F_y}$ $= \dfrac{(344 \text{ kip-ft})(12 \text{ in./ft})}{0.90(50 \text{ ksi})}$ $= 91.7 \text{ in.}^3$	$\Omega_b = 1.67$ $Z_{x,min} = \dfrac{\Omega_b M_a}{F_y}$ $= \dfrac{1.67(266 \text{ kip-ft})(12 \text{ in./ft})}{50 \text{ ksi}}$ $= 107 \text{ in.}^3$

From AISC *Manual* Table 3-2, select a W21×50 with a Z_x value of 110 in.^3

Note that for the member size chosen, the self-weight on a pounds per square foot basis is $50 \text{ plf}/10 \text{ ft} = 5.00 \text{ psf}$; thus the initial self-weight assumption is adequate.

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W21×50
$A = 14.7 \text{ in.}^2$
$t_f = 0.535 \text{ in.}$
$h/t_w = 49.4$
$I_x = 984 \text{ in.}^4$

Pre-Composite Deflections

AISC Design Guide 3 (West and Fisher, 2003) recommends deflections due to concrete plus self-weight not exceed the minimum of $L/360$ or 1.0 in.

From AISC *Manual* Table 3-23, Case 1:

$$\Delta_{nc} = \frac{5 w_D L^4}{384 EI}$$

Substituting for the moment of inertia of the non-composite section, $I = 984 \text{ in.}^4$, yields a dead load deflection of:

$$\begin{aligned}\Delta_{nc} &= \frac{5(0.800 \text{ kip/ft})(1 \text{ ft/12 in.})\left[(45 \text{ ft})(12 \text{ in./ft})\right]^4}{384(29{,}000 \text{ ksi})(984 \text{ in.}^4)} \\ &= 2.59 \text{ in.} \\ &= L/208 > L/360 \quad \textbf{n.g.}\end{aligned}$$

Pre-composite deflections exceed the recommended limit. One possible solution is to increase the member size. A second solution is to induce camber into the member. For this example, the second solution is selected, and the beam will be cambered to reduce the net pre-composite deflections.

Reducing the estimated simple span deflections to 80% of the calculated value to reflect the partial restraint of the end connections as recommended in AISC Design Guide 3 yields a camber of:

$$\begin{aligned}\text{Camber} &= 0.8(2.59\text{ in.})\\ &= 2.07\text{ in.}\end{aligned}$$

Rounding down to the nearest ¼-in. increment yields a specified camber of 2 in.

Select a W21×50 with 2 in. of camber.

Design for Composite Condition

Required Flexural Strength

Using tributary area calculations, the total uniform loads (including pre-composite dead loads in addition to dead and live loads applied after composite action has been achieved) are determined as:

$$\begin{aligned}w_D &= \left[(10\text{ ft})\left(75\text{ lb/ft}^2+5\text{ lb/ft}^2+10\text{ lb/ft}^2\right)\right](1\text{ kip/1,000 lb})\\ &= 0.900\text{ kip/ft}\end{aligned}$$

$$\begin{aligned}w_L &= \left[(10\text{ ft})\left(100\text{ lb/ft}^2\right)\right](1\text{ kip/1,000 lb})\\ &= 1.00\text{ kip/ft}\end{aligned}$$

From ASCE/SEI 7, Chapter 2, the required flexural strength is:

LRFD	ASD
$w_u = 1.2(0.900\text{ kip/ft})+1.6(1.00\text{ kip/ft})$ $= 2.68\text{ kip/ft}$ $M_u = \dfrac{w_u L^2}{8}$ $= \dfrac{(2.68\text{ kip/ft})(45\text{ ft})^2}{8}$ $= 678\text{ kip-ft}$	$w_a = 0.900\text{ kip/ft}+1.00\text{ kip/ft}$ $= 1.90\text{ kip/ft}$ $M_a = \dfrac{w_a L^2}{8}$ $= \dfrac{(1.90\text{ kip/ft})(45\text{ ft})^2}{8}$ $= 481\text{ kip-ft}$

Determine effective width, b

The effective width of the concrete slab is the sum of the effective widths to each side of the beam centerline as determined by the minimum value of the three widths set forth in AISC *Specification* Section I3.1a:

1. one-eighth of the beam span, center-to-center of supports

$$\frac{45\text{ ft}}{8}(2\text{ sides}) = 11.3\text{ ft}$$

2. one-half the distance to the centerline of the adjacent beam

$$\frac{10\text{ ft}}{2}(2\text{ sides}) = 10.0\text{ ft}\quad\textbf{controls}$$

3. distance to the edge of the slab

   The latter is not applicable for an interior member.

Available Flexural Strength

According to AISC *Specification* Section I3.2a, the nominal flexural strength shall be determined from the plastic stress distribution on the composite section when $h/t_w \le 3.76\sqrt{E/F_y}$.

$$\begin{aligned} 49.4 &\le 3.76\sqrt{(29{,}000\text{ ksi})/(50\text{ ksi})} \\ &< 90.6 \end{aligned}$$

Therefore, use the plastic stress distribution to determine the nominal flexural strength.

According to the User Note in AISC *Specification* Section I3.2a, this check is generally unnecessary as all current W-shapes satisfy this limit for $F_y \le 70$ ksi.

Flexural strength can be determined using AISC *Manual* Table 3-19 or calculated directly using the provisions of AISC *Specification* Chapter I. This design example illustrates the use of the *Manual* table only. For an illustration of the direct calculation procedure, refer to Design Example I.2.

To utilize AISC *Manual* Table 3-19, the distance from the compressive concrete flange force to beam top flange, $Y2$, must first be determined as illustrated by *Manual* Figure 3-3. Fifty percent composite action [$\Sigma Q_n \approx 0.50(A_sF_y)$] is used to calculate a trial value of the compression block depth, a_{trial}, for determining $Y2$ as follows:

$$\begin{aligned} a_{trial} &= \frac{\sum Q_n}{0.85 f_c' b} \\ &= \frac{0.50(A_s F_y)}{0.85 f_c' b} \\ &= \frac{0.50\left(14.7\text{ in.}^2\right)(50\text{ ksi})}{0.85(4\text{ ksi})(10\text{ ft})(12\text{ in./ft})} \\ &= 0.90\text{ in.} \rightarrow \text{say } 1.00\text{ in.} \end{aligned}$$

(from *Manual* Eq. 3-7)

Note that a trial value of a = 1 in. is a common starting point in many design problems.

$$Y2 = Y_{con} - \frac{a_{trial}}{2}$$

(from *Manual*. Eq 3-6)

where

$$\begin{aligned} Y_{con} &= \text{distance from top of steel beam to top of slab, in.} \\ &= 7.50\text{ in.} \end{aligned}$$

$$\begin{aligned} Y2 &= 7.50\text{ in.} - \frac{1\text{ in.}}{2} \\ &= 7.00\text{ in.} \end{aligned}$$

Enter AISC *Manual* Table 3-19 with the required strength and $Y2$ = 7.00 in. to select a plastic neutral axis location for the W21×50 that provides sufficient available strength.

Selecting PNA location 5 (BFL) with $\sum Q_n = 386$ kips provides a flexural strength of:

LRFD	ASD
$\phi_b M_n = 769 \text{ kip-ft} > 678 \text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = 512 \text{ kip-ft} > 481 \text{ kip-ft}$ **o.k.**

Based on the available flexural strength provided in Table 3-19, the required PNA location for ASD and LRFD design methodologies differ. This discrepancy is due to the live to dead load ratio in this example, which is not equal to the ratio of 3 at which ASD and LRFD design methodologies produce equivalent results as discussed in AISC *Specification* Commentary Section B3.2. The selected PNA location 5 is acceptable for ASD design, and more conservative for LRFD design.

The actual value for the compression block depth, *a*, is determined as follows:

$$a = \frac{\sum Q_n}{0.85 f_c' b} \qquad \text{(\textit{Manual} Eq. 3-7)}$$

$$= \frac{386 \text{ kips}}{0.85(4 \text{ ksi})(10 \text{ ft})(12 \text{ in./ft})}$$

$$= 0.946 \text{ in.} < a_{trial} = 1.00 \text{ in.} \quad \textbf{o.k.}$$

Live Load Deflection

Deflections due to live load applied after composite action has been achieved will be limited to $L/360$ under the design live load as required by Table 1604.3 of the *International Building Code* (IBC) (ICC, 2015), or 1 in. using a 50% reduction in design live load as recommended by AISC Design Guide 3.

Deflections for composite members may be determined using the lower bound moment of inertia provided by *Specification* Commentary Equation C-I3-1 and tabulated in AISC *Manual* Table 3-20. The *Specification* Commentary also provides an alternate method for determining deflections of a composite member through the calculation of an effective moment of inertia. This design example illustrates the use of the *Manual* table. For an illustration of the direct calculation procedure for each method, refer to Design Example I.2.

Entering Table 3-20, for a W21×50 with PNA location 5 and $Y2$ = 7.00 in., provides a lower bound moment of inertia of $I_{LB} = 2{,}520 \text{ in.}^4$

Inserting I_{LB} into AISC *Manual* Table 3-23, Case 1, to determine the live load deflection under the full design live load for comparison to the IBC limit yields:

$$\Delta_c = \frac{5 w_L L^4}{384 E I_{LB}}$$

$$= \frac{5(1.00 \text{ kip/ft})(1 \text{ ft/12 in.})\left[(45 \text{ ft})(12 \text{ in./ft})\right]^4}{384(29{,}000 \text{ ksi})(2{,}520 \text{ in.}^4)}$$

$$= 1.26 \text{ in.}$$

$$= L/429 < L/360 \quad \textbf{o.k.}$$

Performing the same check with 50% of the design live load for comparison to the AISC Design Guide 3 limit yields:

$$\begin{aligned}\Delta_c &= 0.50(1.26\text{ in.})\\ &= 0.630\text{ in.} < 1\text{ in.} \quad \textbf{o.k.}\end{aligned}$$

Steel Anchor Strength

Steel headed stud anchor strengths are tabulated in AISC *Manual* Table 3-21 for typical conditions. Conservatively assuming that all anchors are placed in the weak position, the strength for ¾-in.-diameter anchors in normal weight concrete with $f_c' = 4$ ksi and deck oriented perpendicular to the beam is:

1 anchor per rib: $Q_n = 17.2$ kips/anchor
2 anchors per rib: $Q_n = 14.6$ kips/anchor

Number and Spacing of Anchors

Deck flutes are spaced at 12 in. on center according to the deck manufacturer's literature. The minimum number of deck flutes along each half of the 45-ft-long beam, assuming the first flute begins a maximum of 12 in. from the support line at each end, is:

$$\begin{aligned}n_{flutes} &= n_{spaces} + 1\\ &= \frac{45\text{ ft} - 2(12\text{ in.})(1\text{ ft/12 in.})}{2(1\text{ ft per space})} + 1\\ &= 22.5 \rightarrow \text{say 22 flutes}\end{aligned}$$

According to AISC *Specification* Section I8.2c, the number of steel headed stud anchors required between the section of maximum bending moment and the nearest point of zero moment is determined by dividing the required horizontal shear, $\sum Q_n$, by the nominal shear strength per anchor, Q_n. Assuming one anchor per flute:

$$\begin{aligned}n_{anchors} &= \frac{\sum Q_n}{Q_n}\\ &= \frac{386\text{ kips}}{17.2\text{ kips/anchor}}\\ &= 22.4 \rightarrow \text{place 23 anchors on each side of the beam centerline}\end{aligned}$$

As the number of anchors exceeds the number of available flutes by one, place two anchors in the first flute. The revised horizontal shear capacity of the anchors taking into account the reduced strength for two anchors in one flute is:

$$\begin{aligned}\sum Q_n &= 2(14.6\text{ kips}) + 21(17.2\text{ kips})\\ &= 390\text{ kips} > 386\text{ kips} \quad \textbf{o.k.}\end{aligned}$$

Steel Anchor Ductility Check

As discussed in AISC *Specification* Commentary to Section I3.2d, beams are not susceptible to connector failure due to insufficient deformation capacity if they meet one or more of the following conditions:

1. Beams with span not exceeding 30 ft;
2. Beams with a degree of composite action of at least 50%; or

3. Beams with an average nominal shear connector capacity of at least 16 kips per foot along their span, corresponding to a ¾-in.-diameter steel headed stud anchor placed at 12 in. spacing on average.

The span is 45 ft, which exceeds the 30 ft limit. The percent composite action is:

$$\begin{aligned}\frac{\sum Q_n}{\min\left\{0.85 f_c' A_c, F_y A_s\right\}} &= \frac{390\text{ kips}}{\min\left\{0.85(4\text{ ksi})(10\text{ ft})(12\text{ in./ft})(4.5\text{ in.}),\ (50\text{ ksi})(14.7\text{ in.}^2)\right\}}(100) \\ &= \frac{390\text{ kips}}{735\text{ kips}}(100) \\ &= 53.1\%\end{aligned}$$

which exceeds the minimum degree of composite action of 50%. The average shear connector capacity is:

$$\frac{(42\text{ anchors})(17.2\text{ kips/anchor})+(4\text{ anchors})(14.6\text{ kips/anchor})}{45\text{ ft}} = 17.4\text{ kip/ft}$$

which exceeds the minimum capacity of 16 kips per foot. Since at least one of the conditions has been met (in fact, two have been met), the shear connectors meet the ductility requirements. The final anchor pattern chosen is illustrated in Figure I.1-2.

Review steel headed stud anchor spacing requirements of AISC *Specification* Sections I8.2d and I3.2c.

1. Maximum anchor spacing along beam [Section I8.2d(e)]:

   $$\begin{aligned}8t_{slab} &= 8(7.50\text{ in.}) \\ &= 60.0\text{ in.}\end{aligned}$$

   or

   $$36\text{ in.}$$

   The maximum anchor spacing permitted is 36 in.

   $36\text{ in.} > 12\text{ in.}$ **o.k.**

2. Minimum anchor spacing along beam [Section I8.2d(d)]:

   $$\begin{aligned}4d_{sa} &= 4(\tfrac{3}{4}\text{ in.}) \\ &= 3.00\text{ in.} < 12\text{ in.}\end{aligned}$$ **o.k.**

3. Minimum transverse spacing between anchor pairs [Section I8.2d(d)]:

   $$\begin{aligned}4d_{sa} &= 4(\tfrac{3}{4}\text{ in.}) \\ &= 3.00\text{ in.} \le 3.00\text{ in.}\end{aligned}$$ **o.k.**

4. Minimum distance to free edge in the direction of the horizontal shear force:

   AISC *Specification* Section I8.2d requires that the distance from the center of an anchor to a free edge in the direction of the shear force be a minimum of 8 in. for normal weight concrete slabs.

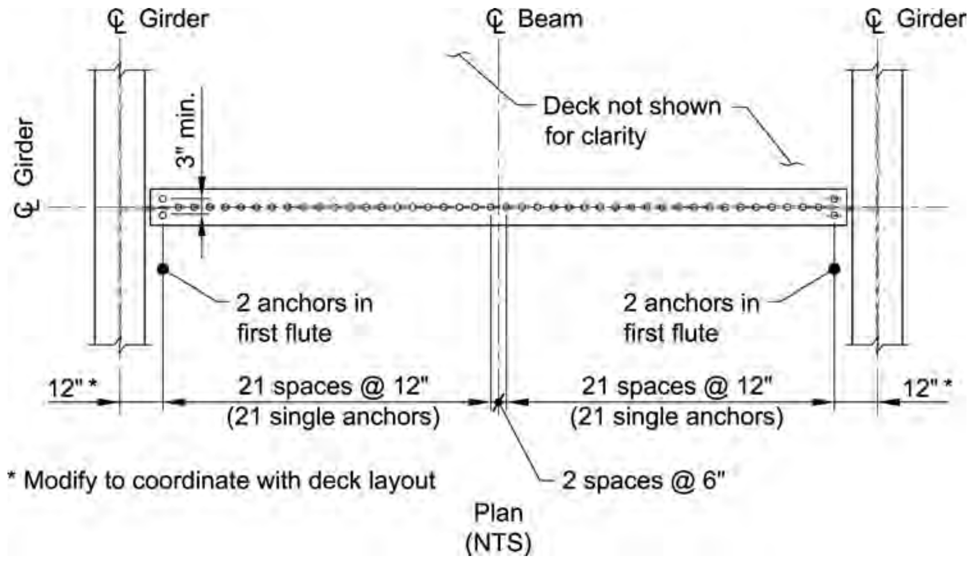

Fig. I.1-2. Steel headed stud anchor layout.

5. Maximum spacing of deck attachment:

   AISC *Specification* Section I3.2c.1(d) requires that steel deck be anchored to all supporting members at a maximum spacing of 18 in. The stud anchors are welded through the metal deck at a maximum spacing of 12 inches in this example, thus this limit is met without the need for additional puddle welds or mechanical fasteners.

Available Shear Strength

According to AISC *Specification* Section I4.2, the beam should be assessed for available shear strength as a bare steel beam using the provisions of Chapter G.

Applying the loads previously determined for the governing ASCE/SEI 7 load combinations and using available shear strengths from AISC *Manual* Table 3-2 for a W21×50 yields the following:

LRFD	ASD
$V_u = \dfrac{w_u L}{2}$ $= \dfrac{(2.68 \text{ kips/ft})(45 \text{ ft})}{2}$ $= 60.3 \text{ kips}$ $\phi_v V_n = 237 \text{ kips} > 60.3 \text{ kips}$ **o.k.**	$V_a = \dfrac{w_a L}{2}$ $= \dfrac{(1.90 \text{ kips/ft})(45 \text{ ft})}{2}$ $= 42.8 \text{ kips}$ $\dfrac{V_n}{\Omega_v} = 158 \text{ kips} > 42.8 \text{ kips}$ **o.k.**

Serviceability

Depending on the intended use of this bay, vibrations might need to be considered. Refer to AISC Design Guide 11 (Murray et al., 2016) for additional information.

Summary

From Figure I.1-2, the total number of stud anchors used is equal to (2)(2 + 21) = 46. A plan layout illustrating the final beam design is provided in Figure I.1-3. A W21×50 with 2 in. of camber and 46, ¾-in.-diameter by 4⅞-in.-long steel headed stud anchors is adequate to resist the imposed loads.

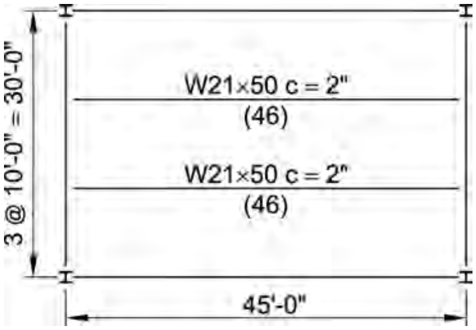

Fig. I.1-3. Revised plan.

EXAMPLE I.2 COMPOSITE GIRDER DESIGN

Given:

Two typical bays of a composite floor system are illustrated in Figure I.2-1. Select an appropriate ASTM A992 W-shaped girder and determine the required number of steel headed stud anchors. The girder will not be shored during construction. Use steel headed stud anchors made from ASTM A108 material, with F_u = 65 ksi.

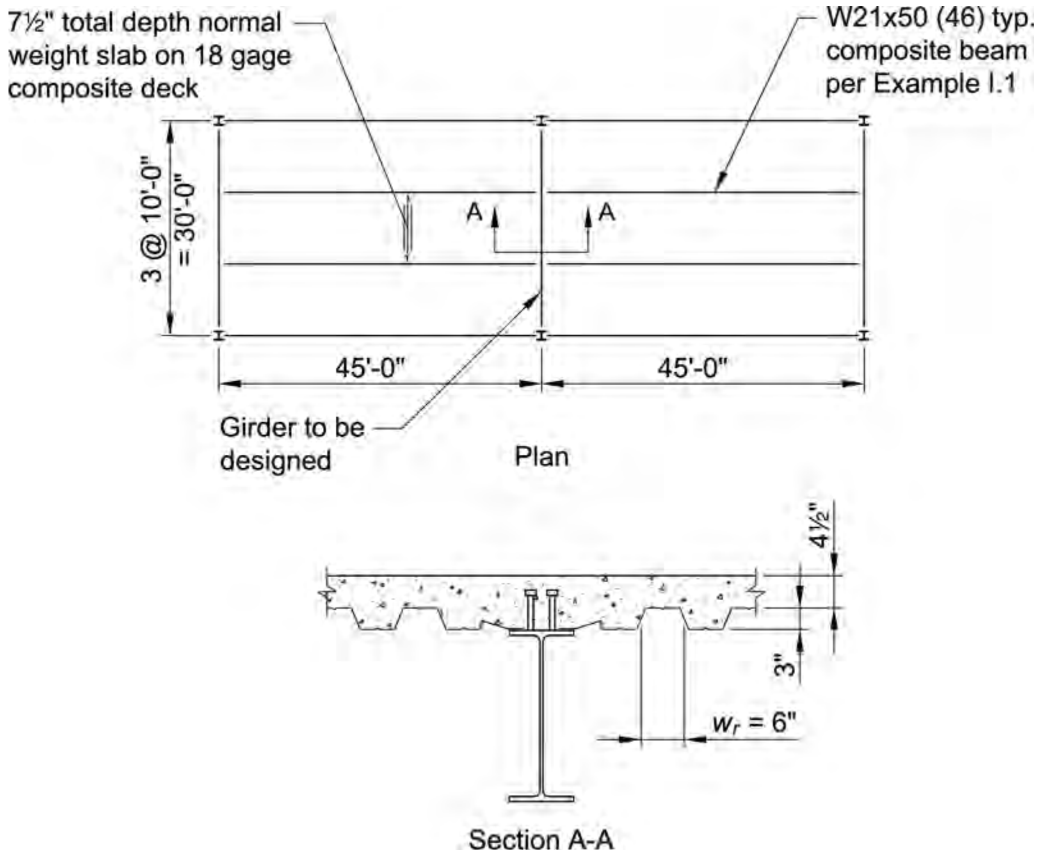

Fig. I.2-1. Composite bay and girder section.

To achieve a two-hour fire rating without the application of spray applied fire protection material to the composite deck, 4½ in. of normal weight (145 lb/ft^3) concrete will be placed above the top of the deck. The concrete has a specified compressive strength, f_c' = 4 ksi.

Applied loads are given in the following:

Dead Loads:
  Pre-composite:
    Slab = 75 lb/ft^2 (in accordance with metal deck manufacturer's data)
    Self-weight = 80 lb/ft (trial girder weight)
    = 50 lb/ft (beam weight from Design Example I.1)

  Composite (applied after composite action has been achieved):
    Miscellaneous = 10 lb/ft^2 (HVAC, ceiling, floor covering, etc.)

Live Loads:
  Pre-composite:
    Construction = 25 lb/ft^2 (temporary loads during concrete placement)

Composite (applied after composite action has been achieved):
Non-reducible = 100 lb/ft^2 (assembly occupancy)

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Applied Loads

For slabs that are to be placed at a constant elevation, AISC Design Guide 3 (West and Fisher, 2003) recommends an additional 10% of the nominal slab weight be applied to account for concrete ponding due to deflections resulting from the wet weight of the concrete during placement. For the slab under consideration, this would result in an additional load of 8 lb/ft^2; however, for this design the slab will be placed at a constant thickness, and thus, no additional weight for concrete ponding is required.

For pre-composite construction live loading, 25 lb/ft^2 will be applied in accordance with recommendations from *Design Loads on Structures During Construction*, ASCE/SEI 37 (ASCE, 2014), for a light duty operational class that includes concrete transport and placement by hose and finishing with hand tools.

Composite Deck and Anchor Requirements

Check composite deck and anchor requirements stipulated in AISC *Specification* Sections I1.3, I3.2c and I8.

1. Concrete strength: $3 \text{ ksi} \leq f_c' \leq 10 \text{ ksi}$ (for normal weight concrete) (*Spec.* Section I1.3)

   $f_c' = 4 \text{ ksi}$ **o.k.**

2. Rib height: $h_r \leq 3 \text{ in.}$ (*Spec.* Section I3.2c)

   $h_r = 3 \text{ in.}$ **o.k.**

3. Average rib width: $w_r \geq 2 \text{ in.}$ (*Spec.* Section I3.2c)

   $w_r = 6 \text{ in.}$ (See Figure I.2-1) **o.k.**

4. Use steel headed stud anchors ¾ in. or less in diameter. (*Spec.* Section I8.1)

   Select ¾-in.-diameter steel anchors. **o.k.**

5. Steel headed stud anchor diameter: $d_{sa} \leq 2.5t_f$ (*Spec.* Section I8.1)

   In accordance with AISC *Specification* Section I8.1, this limit only applies if steel headed stud anchors are not welded to the flange directly over the web. The ¾-in.-diameter anchors will be attached in a staggered pattern, thus this limit must be satisfied. Select a girder size with a minimum flange thickness of 0.300 in., as determined in the following:

$$\begin{aligned} t_f &\geq \frac{d_{sa}}{2.5} \\ &\geq \frac{\text{¾ in.}}{2.5} \\ &\geq 0.300 \text{ in.} \end{aligned}$$

6. In accordance with AISC *Specification* I3.2c, steel headed stud anchors, after installation, shall extend not less than 1½ in. above the top of the steel deck. A minimum anchor length of 4½ in. is required to meet this requirement for 3-in.-deep deck. From steel headed stud anchor manufacturer's data, a standard stock length of 4⅞ in. is selected. Using a $\frac{3}{16}$-in. length reduction to account for burn off during anchor installation directly to the girder flange yields a final installed length of 4$\frac{11}{16}$ in.

   4$\frac{11}{16}$ in. > 4½ in. **o.k.**

7. Minimum length of stud anchors = $4d_{sa}$ (*Spec.* Section I8.2)

   4$\frac{11}{16}$ in. > 4(¾ in.) = 3.00 in. **o.k.**

8. In accordance with AISC *Specification* Section I3.2c, there shall be at least ½ in. of specified concrete cover above the top of the headed stud anchors.

   As discussed in the *Specification* Commentary to Section I3.2c, it is advisable to provide greater than ½-in. minimum cover to assure anchors are not exposed in the final condition.

   $7\frac{1}{2}\text{ in.} - 4\frac{11}{16}\text{ in.} = 2\frac{13}{16}\text{ in.} > \frac{1}{2}\text{ in.}$ **o.k.**

9. In accordance with AISC *Specification* Section I3.2c, slab thickness above steel deck shall not be less than 2 in.

   $4\frac{1}{2}\text{ in.} > 2\text{ in.}$ **o.k.**

Design for Pre-Composite Condition

Construction (Pre-Composite) Loads

The girder will be loaded at third points by the supported beams. Determine point loads using tributary areas.

$$\begin{aligned} P_D &= \left[(45\text{ ft})(10\text{ ft})\left(75\text{ lb/ft}^2\right) + (45\text{ ft})(50\text{ lb/ft})\right](1\text{ kip/1,000 lb}) \\ &= 36.0\text{ kips} \end{aligned}$$

$$\begin{aligned} P_L &= \left[(45\text{ ft})(10\text{ ft})\left(25\text{ lb/ft}^2\right)\right](1\text{ kip/1,000 lb}) \\ &= 11.3\text{ kips} \end{aligned}$$

Construction (Pre-Composite) Flexural Strength

From ASCE/SEI 7, Chapter 2, the required flexural strength is:

LRFD	ASD
$P_u = 1.2(36.0\text{ kips}) + 1.6(11.3\text{ kips})$ $= 61.3\text{ kips}$ $w_u = 1.2(80\text{ lb/ft})(1\text{ kip/1,000 lb})$ $= 0.0960\text{ kip/ft}$	$P_a = 36.0\text{ kips} + 11.3\text{ kips}$ $= 47.3\text{ kips}$ $w_a = (80\text{ lb/ft})(1\text{ kip/1,000 lb})$ $= 0.0800\text{ kip/ft}$

LRFD	ASD
$M_u = P_u a + \dfrac{w_u L^2}{8}$ $= (61.3\text{ kips})(10\text{ ft}) + \dfrac{(0.0960\text{ kip/ft})(30\text{ ft})^2}{8}$ $= 624\text{ kip-ft}$	$M_a = P_a a + \dfrac{w_a L^2}{8}$ $= (47.3\text{ kips})(10\text{ ft}) + \dfrac{(0.0800\text{ kip/ft})(30\text{ ft})^2}{8}$ $= 482\text{ kip-ft}$

Girder Selection

Based on the required flexural strength under construction loading, a trial member can be selected utilizing AISC *Manual* Table 3-2. For the purposes of this example, the unbraced length of the girder prior to hardening of the concrete is taken as the distance between supported beams (one-third of the girder length).

Try a W24×76

$L_b = 10\text{ ft}$

$L_p = 6.78\text{ ft}$

$L_r = 19.5\text{ ft}$

LRFD	ASD
$\phi_b BF = 22.6\text{ kips}$ $\phi_b M_{px} = 750\text{ kip-ft}$ $\phi_b M_{rx} = 462\text{ kip-ft}$	$BF/\Omega_b = 15.1\text{ kips}$ $M_{px}/\Omega_b = 499\text{ kip-ft}$ $M_{rx}/\Omega_b = 307\text{ kip-ft}$

Because $L_p < L_b < L_r$, use AISC *Manual* Equations 3-4a and 3-4b with $C_b = 1.0$ within the center girder segment in accordance with AISC *Manual* Table 3-1:

LRFD	ASD
From AISC *Manual* Equation 3-4a: $\phi_b M_n = C_b\left[\phi_b M_{px} - \phi_b BF(L_b - L_p)\right] \le \phi_b M_{px}$ $= 1.0[750\text{ kip-ft} - (22.6\text{ kips})(10\text{ ft} - 6.78\text{ ft})]$ $\le 750\text{ kip-ft}$ $= 677\text{ kip-ft} < 750\text{ kip-ft}$ $= 677\text{ kip-ft}$ $\phi_b M_n \ge M_u$ $677\text{ kip-ft} > 624\text{ kip-ft}$ **o.k.**	From AISC *Manual* Equation 3-4b: $\dfrac{M_n}{\Omega_b} = C_b\left[\dfrac{M_{px}}{\Omega_b} - \dfrac{BF}{\Omega_b}(L_b - L_p)\right] \le \dfrac{M_{px}}{\Omega_b}$ $= 1.0[499\text{ kip-ft} - (15.1\text{ kips})(10\text{ ft} - 6.78\text{ ft})]$ $\le 499\text{ kip-ft}$ $= 450\text{ kip-ft} < 499\text{ kip-ft}$ $= 450\text{ kip-ft}$ $\dfrac{M_n}{\Omega_b} \ge M_a$ $450\text{ kip-ft} < 482\text{ kip-ft}$ **n.g.**

For this example, the relatively low live load to dead load ratio results in a lighter member when LRFD methodology is employed. When ASD methodology is employed, a heavier member is required, and it can be shown that a W24×84 is adequate for pre-composite flexural strength. This example uses a W24×76 member to illustrate the determination of flexural strength of the composite section using both LRFD and ASD methodologies; however, this is done for comparison purposes only, and calculations for a W24×84 would be required to provide a satisfactory ASD design. Calculations for the heavier section are not shown as they would essentially be a duplication of the calculations provided for the W24×76 member.

Note that for the member size chosen, 76 lb/ft < 80 lb/ft, thus the initial weight assumption is adequate.

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W24×76
A = 22.4 in.^{2}
h/t_w = 49.0
I_x = 2,100 in.^{4}
b_f = 8.99 in.
t_f = 0.680 in.
d = 23.9 in.

Pre-Composite Deflections

AISC Design Guide 3 (West and Fisher, 2003) recommends deflections due to concrete plus self-weight not exceed the minimum of L/360 or 1.0 in.

From the superposition of AISC *Manual* Table 3-23, Cases 1 and 9:

$$\Delta_{nc} = \frac{23P_D L^3}{648EI} + \frac{5w_D L^4}{384EI}$$

Substituting for the moment of inertia of the non-composite section, $I = 2{,}100\text{ in.}^4$, yields a dead load deflection of:

$$\begin{aligned}\Delta_{nc} &= \frac{23(36.0\text{ kips})\left[(30\text{ ft})(12\text{ in./ft})\right]^3}{648(29{,}000\text{ ksi})(2{,}100\text{ in.}^4)} + \frac{5(0.0760\text{ kip/ft})(1\text{ ft/12 in.})\left[(30\text{ ft})(12\text{ in./ft})\right]^4}{384(29{,}000\text{ ksi})(2{,}100\text{ in.}^4)}\\ &= 1.00\text{ in.}\\ &= L/360 \quad \textbf{o.k.}\end{aligned}$$

Pre-composite deflections barely meet the recommended value. Although technically acceptable, judgment leads one to consider ways to minimize pre-composite deflections. One possible solution is to increase the member size. A second solution is to introduce camber into the member. For this example, the second solution is selected, and the girder will be cambered to reduce pre-composite deflections.

Reducing the estimated simple span deflections to 80% of the calculated value to reflect the partial restraint of the end connections as recommended in AISC Design Guide 3 yields a camber of:

$$\begin{aligned}\text{Camber} &= 0.80(1.00\text{ in.})\\ &= 0.800\text{ in.}\end{aligned}$$

Rounding down to the nearest ¼-in. increment yields a specified camber of ¾ in.

Select a W24×76 with ¾ in. of camber.

Design for Composite Flexural Strength

Required Flexural Strength

Using tributary area calculations, the total applied point loads (including pre-composite dead loads in addition to dead and live loads applied after composite action has been achieved) are determined as:

$$P_D = \left[(45\text{ ft})(10\text{ ft})\left(75\text{ lb/ft}^2 + 10\text{ lb/ft}^2\right) + (45\text{ ft})(50\text{ lb/ft})\right](1\text{ kip/1,000 lb})$$
$$= 40.5\text{ kips}$$

$$P_L = \left[(45\text{ ft})(10\text{ ft})\left(100\text{ lb/ft}^2\right)\right](1\text{ kip/1,000 lb})$$
$$= 45.0\text{ kips}$$

The required flexural strength diagram is illustrated by Figure I.2-2:

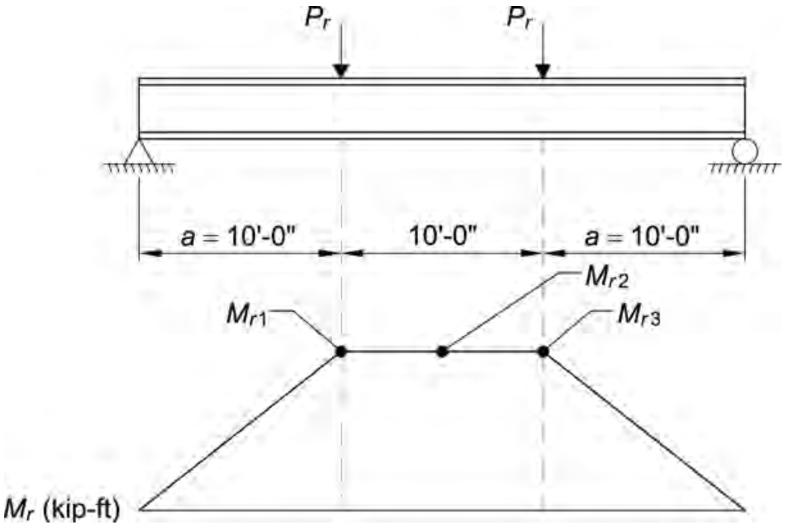

Fig. I.2-2. Required flexural strength.

From ASCE/SEI 7, Chapter 2, the required flexural strength is:

LRFD	ASD
$P_r = P_u$ $= 1.2(40.5\text{ kips}) + 1.6(45.0\text{ kips})$ $= 121\text{ kips}$ $w_u = 1.2(0.0760\text{ kip/ft})$ $= 0.0912$ kip/ft (from self weight of W24×76)	$P_r = P_a$ $= 40.5\text{ kips} + 45.0\text{ kips}$ $= 85.5\text{ kips}$ $w_a = 0.0760$ kip/ft (from self weight of W24×76)

LRFD	ASD
From AISC *Manual* Table 3-23, Case 1 and 9: $M_{u1} = M_{u3}$ $= P_u a + \dfrac{w_u a}{2}(L - a)$ $= (121\text{ kips})(10\text{ ft})$ $+ \dfrac{(0.0912\text{ kip/ft})(10\text{ ft})}{2}(30\text{ ft} - 10\text{ ft})$ $= 1{,}220\text{ kip-ft}$	From AISC *Manual* Table 3-23, Case 1 and 9: $M_{a1} = M_{a3}$ $= P_a a + \dfrac{w_a a}{2}(L - a)$ $= (85.5\text{ kips})(10\text{ ft})$ $+ \dfrac{(0.0760\text{ kip/ft})(10\text{ ft})}{2}(30\text{ ft} - 10\text{ ft})$ $= 863\text{ kip-ft}$

LRFD	ASD
$M_{u2} = P_u a + \dfrac{w_u L^2}{8}$ $= (121 \text{ kips})(10 \text{ ft}) + \dfrac{(0.0912 \text{ kip/ft})(30 \text{ ft})^2}{8}$ $= 1{,}220 \text{ kip-ft}$	$M_{a2} = P_a a + \dfrac{w_a L^2}{8}$ $= (85.5 \text{ kips})(10 \text{ ft}) + \dfrac{(0.0760 \text{ kip/ft})(30 \text{ ft})^2}{8}$ $= 864 \text{ kip-ft}$

Determine Effective Width, b

The effective width of the concrete slab is the sum of the effective widths to each side of the beam centerline as determined by the minimum value of the three conditions set forth in AISC *Specification* Section I3.1a:

1. one-eighth of the girder span center-to-center of supports

$$\left(\frac{30 \text{ ft}}{8}\right)(2 \text{ sides}) = 7.50 \text{ ft} \quad \textbf{controls}$$

2. one-half the distance to the centerline of the adjacent girder

$$\left(\frac{45 \text{ ft}}{2}\right)(2 \text{ sides}) = 45.0 \text{ ft}$$

3. distance to the edge of the slab

   The latter is not applicable for an interior member.

Available Flexural Strength

According to AISC *Specification* Section I3.2a, the nominal flexural strength shall be determined from the plastic stress distribution on the composite section when $h/t_w \le 3.76\sqrt{E/F_y}$.

$$\begin{aligned} 49.0 &\le 3.76\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} \\ &< 90.6 \end{aligned}$$

Therefore, use the plastic stress distribution to determine the nominal flexural strength.

According to the User Note in AISC *Specification* Section I3.2a, this check is generally unnecessary as all current W-shapes satisfy this limit for $F_y \le 70$ ksi.

AISC *Manual* Table 3-19 can be used to facilitate the calculation of flexural strength for composite beams. Alternately, the available flexural strength can be determined directly using the provisions of AISC *Specification* Chapter I. Both methods will be illustrated for comparison in the following calculations.

Method 1: AISC Manual

To utilize AISC *Manual* Table 3-19, the distance from the compressive concrete flange force to beam top flange, $Y2$, must first be determined as illustrated by *Manual* Figure 3-3. Fifty percent composite action [$\Sigma Q_n \approx 0.50(A_s F_y)$] is used to calculate a trial value of the compression block depth, a_{trial}, for determining $Y2$ as follows:

$$a_{trial} = \frac{\sum Q_n}{0.85 f_c' b} \qquad \text{(from } \textit{Manual} \text{ Eq. 3-7)}$$

$$= \frac{0.50(A_s F_y)}{0.85 f_c' b}$$

$$= \frac{0.50(22.4 \text{ in.}^2)(50 \text{ ksi})}{0.85(4 \text{ ksi})(7.50 \text{ ft})(12 \text{ in./ft})}$$

$$= 1.83 \text{ in.}$$

$$Y2 = Y_{con} - \frac{a_{trial}}{2} \qquad \text{(from } \textit{Manual}\text{. Eq. 3-6)}$$

where

Y_{con} = distance from top of steel beam to top of slab
= 7.50 in.

$$Y2 = 7.50 \text{ in.} - \frac{1.83 \text{ in.}}{2}$$

$$= 6.59 \text{ in.}$$

Enter AISC *Manual* Table 3-19 with the required strength and $Y2 = 6.59$ in. to select a plastic neutral axis location for the W24×76 that provides sufficient available strength. Based on the available flexural strength provided in Table 3-19, the required PNA location for ASD and LRFD design methodologies differ. This discrepancy is due to the live-to-dead load ratio in this example, which is not equal to the ratio of 3 at which ASD and LRFD design methodologies produce equivalent results as discussed in AISC *Specification* Commentary Section B3.2.

Selecting PNA location 5 (BFL) with $\sum Q_n = 509$ kips provides a flexural strength of:

LRFD	ASD
$\phi_b M_n = 1{,}240 \text{ kip-ft} > 1{,}220 \text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = 823 \text{ kip-ft} < 864 \text{ kip-ft}$ **n.g.**

The selected PNA location 5 is acceptable for LRFD design, but inadequate for ASD design. For ASD design, it can be shown that a W24×76 is adequate if a higher composite percentage of approximately 60% is employed. However, as discussed previously, this beam size is not adequate for construction loading and a larger section is necessary when designing utilizing ASD.

The actual value for the compression block depth, *a*, for the chosen PNA location is determined as follows:

$$a = \frac{\sum Q_n}{0.85 f_c' b} \qquad \text{(}\textit{Manual} \text{ Eq. 3-7)}$$

$$= \frac{509 \text{ kips}}{0.85(4 \text{ ksi})(7.50 \text{ ft})(12 \text{ in./ft})}$$

$$= 1.66 \text{ in.} < a_{trial} = 1.83 \text{ in.} \quad \textbf{o.k. for LRFD design}$$

Method 2: Direct Calculation

According to AISC *Specification* Commentary Section I3.2a, the number and strength of steel headed stud anchors will govern the compressive force, C, for a partially composite beam. The composite percentage is based on the minimum of the limit states of concrete crushing and steel yielding as follows:

1. Concrete crushing

A_c = Area of concrete slab within effective width. Assume that the deck profile is 50% void and 50% concrete fill.

$$\begin{aligned}&= b_{eff}\left(4\tfrac{1}{2}\text{ in.}\right)+\left(b_{eff}/2\right)\left(3\text{ in.}\right)\\&=\left(7.50\text{ ft}\right)\left(12\text{ in./ft}\right)\left(4\tfrac{1}{2}\text{ in.}\right)+\left[\frac{\left(7.50\text{ ft}\right)\left(12\text{ in./ft}\right)}{2}\right]\left(3\text{ in.}\right)\\&=540\text{ in.}^2\end{aligned}$$

$$C = 0.85 f_c' A_c \qquad \text{(Spec. Comm. Eq. C-I3-7)}$$

$$\begin{aligned}&=0.85\left(4\text{ ksi}\right)\left(540\text{ in.}^2\right)\\&=1{,}840\text{ kips}\end{aligned}$$

2. Steel yielding

$$C = A_s F_y \qquad \text{(Spec. Comm. Eq. C-I3-6)}$$

$$\begin{aligned}&=\left(22.4\text{ in.}^2\right)\left(50\text{ ksi}\right)\\&=1{,}120\text{ kips}\end{aligned}$$

3. Shear transfer

Fifty percent is used as a trial percentage of composite action as follows:

$$C = \Sigma Q_n \qquad \text{(Spec. Comm. Eq. C-I3-8)}$$

$$\begin{aligned}&=50\%\left(\min\begin{Bmatrix}1{,}840\text{ kips}\\1{,}120\text{ kips}\end{Bmatrix}\right)\\&=560\text{ kips to achieve 50\% composite action}\end{aligned}$$

Location of the Plastic Neutral Axis

The plastic neutral axis (PNA) is located by determining the axis above and below which the sum of horizontal forces is equal. This concept is illustrated in Figure I.2-3, assuming the trial PNA location is within the top flange of the girder.

$$\Sigma F_{\,above\ PNA} = \Sigma F_{\,below\ PNA}$$

$$C + xb_f F_y = \left(A_s - b_f x\right)F_y$$

Solving for x:

$$
\begin{aligned}
x &= \frac{A_s F_y - C}{2 b_f F_y} \\
&= \frac{\left(22.4\text{ in.}^2\right)\left(50\text{ ksi}\right) - 560\text{ kips}}{2\left(8.99\text{ in.}\right)\left(50\text{ ksi}\right)} \\
&= 0.623\text{ in.} < t_f = 0.680\text{ in.; therefore, the PNA is in the flange}
\end{aligned}
$$

Determine the nominal moment resistance of the composite section following the procedure in AISC *Specification* Commentary Section I3.2a, as illustrated in Figure C-I3.3.

$$
\begin{aligned}
a &= \frac{C}{0.85 f_c' b} && \text{(}\textit{Spec}\text{. Comm. Eq. C-I3-9)} \\
&= \frac{560\text{ kips}}{0.85\left(4\text{ ksi}\right)\left(7.50\text{ ft}\right)\left(12\text{ in./ft}\right)} \\
&= 1.83\text{ in.} < 4.50\text{ in. (above top of deck)}
\end{aligned}
$$

$$
\begin{aligned}
d_1 &= t_{slab} - \frac{a}{2} \\
&= 7.50\text{ in.} - \frac{1.83\text{ in.}}{2} \\
&= 6.59\text{ in.}
\end{aligned}
$$

$$
\begin{aligned}
d_2 &= \frac{x}{2} \\
&= \frac{0.623\text{ in.}}{2} \\
&= 0.312\text{ in.}
\end{aligned}
$$

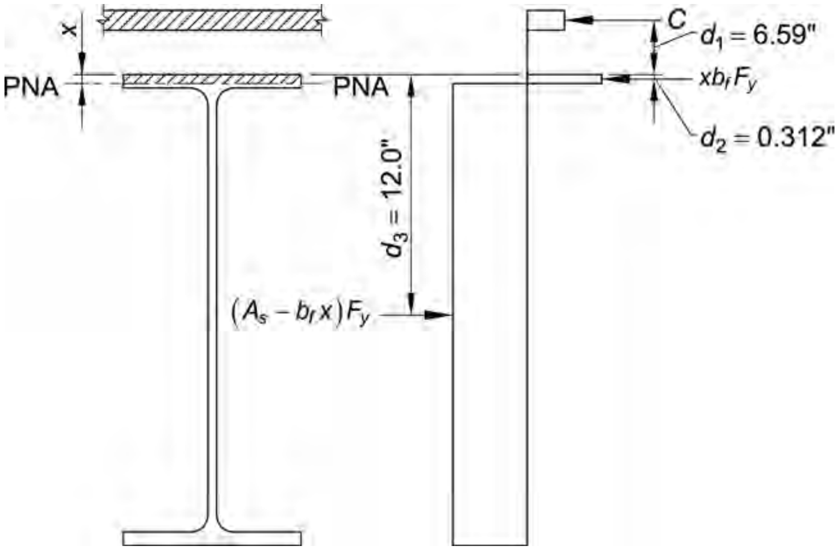

Fig. I.2-3. Plastic neutral axis location.

$$d_3 = \frac{d}{2} = \frac{23.9\text{ in.}}{2} = 12.0\text{ in.}$$

$$P_y = A_s F_y = \left(22.4\text{ in.}^2\right)\left(50\text{ ksi}\right) = 1{,}120\text{ kips}$$

$$M_n = C\left(d_1 + d_2\right) + P_y\left(d_3 - d_2\right) \qquad \text{(\textit{Spec}. Comm. Eq. C-I3-10)}$$
$$= \left(560\text{ kips}\right)\left(6.59\text{ in.} + 0.312\text{ in.}\right) + \left(1{,}120\text{ kips}\right)\left(12.0\text{ in.} - 0.312\text{ in.}\right)$$
$$= 17{,}000\text{ kip-in. or } 1{,}420\text{ kip-ft}$$

Note that Equation C-I3-10 is based on the summation of moments about the centroid of the compression force in the steel; however, the same answer may be obtained by summing moments about any arbitrary point.

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90\left(1{,}420\text{ kip-ft}\right) = 1{,}280\text{ kip-ft} > 1{,}220\text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $\frac{M_n}{\Omega_b} = \frac{1{,}420\text{ kip-ft}}{1.67} = 850\text{ kip-ft} < 864\text{ kip-ft}$ **n.g.**

As was determined previously using the Manual Tables, a W24×76 with 50% composite action is acceptable when LRFD methodology is employed, while for ASD design the beam is inadequate at this level of composite action.

Continue with the design using a W24×76 with 50% composite action.

Steel Anchor Strength

Steel headed stud anchor strengths are tabulated in AISC *Manual* Table 3-21 for typical conditions and may be calculated according to AISC *Specification* Section I8.2a as follows:

$$A_{sa} = \frac{\pi d_{sa}^2}{4} = \frac{\pi\left(\text{¾ in.}\right)^2}{4} = 0.442\text{ in.}^2$$

$$f_c' = 4\text{ ksi}$$

$$E_c = w_c^{1.5}\sqrt{f_c'} = \left(145\text{ lb/ft}^3\right)^{1.5}\sqrt{4\text{ ksi}} = 3{,}490\text{ ksi}$$

$R_g = 1.0$, stud anchors welded directly to the steel shape within the slab haunch

$R_p = 0.75$, stud anchors welded directly to the steel shape

$F_u = 65 \text{ ksi}$

$$\begin{aligned} Q_n &= 0.5A_{sa}\sqrt{f_c' E_c} \le R_g R_p A_{sa} F_u \qquad \text{(}\textit{Spec}\text{. Eq. I8-1)} \\ &= (0.5)\left(0.442 \text{ in.}^2\right)\sqrt{(4 \text{ ksi})(3{,}490 \text{ ksi})} \le (1.0)(0.75)\left(0.442 \text{ in.}^2\right)(65 \text{ ksi}) \\ &= 26.1 \text{ kips} > 21.5 \text{ kips} \end{aligned}$$

Use Q_n = 21.5 kips.

Number and Spacing of Anchors

According to AISC *Specification* Section I8.2c, the number of steel headed stud anchors required between any concentrated load and the nearest point of zero moment shall be sufficient to develop the maximum moment required at the concentrated load point.

From Figure I.2-2 the moment at the concentrated load points, M_{r1} and M_{r3}, is approximately equal to the maximum beam moment, M_{r2}. The number of anchors between the beam ends and the point loads should therefore be adequate to develop the required compressive force associated with the maximum moment, C, previously determined to be 560 kips.

$$\begin{aligned} N_{anchors} &= \frac{\sum Q_n}{Q_n} \\ &= \frac{C}{Q_n} \\ &= \frac{560 \text{ kips}}{21.5 \text{ kips/anchor}} \\ &= 26 \text{ anchors from each end to concentrated load points} \end{aligned}$$

In accordance with AISC *Specification* Section I8.2d, anchors between point loads should be spaced at a maximum of:

$8t_{slab} = 60.0 \text{ in.}$

or 36 in. **controls**

For beams with deck running parallel to the span such as the one under consideration, spacing of the stud anchors is independent of the flute spacing of the deck. Single anchors can therefore be spaced as needed along the beam length provided a minimum longitudinal spacing of six anchor diameters in accordance with AISC *Specification* Section I8.2d is maintained. Anchors can also be placed in aligned or staggered pairs provided a minimum transverse spacing of four stud diameters = 3 in. is maintained. For this design, it was chosen to use pairs of anchors along each end of the girder to meet strength requirements and single anchors along the center section of the girder to meet maximum spacing requirements as illustrated in Figure I.2-4.

AISC *Specification* Section I8.2d requires that the distance from the center of an anchor to a free edge in the direction of the shear force be a minimum of 8 in. for normal weight concrete slabs. For simply-supported composite beams this provision could apply to the distance between the slab edge and the first anchor at each end of the beam. Assuming the slab edge is coincident to the centerline of support, Figure I.2-4 illustrates an acceptable edge distance of 9 in., though in this case the column flange would prevent breakout and negate the need for this check. The slab edge is often uniformly supported by a column flange or pour stop in typical composite construction thus preventing the possibility of a concrete breakout failure and nullifying the edge distance requirement as discussed in AISC *Specification* Commentary Section I8.3.

For this example, the minimum number of headed stud anchors required to meet the maximum spacing limit previously calculated is used within the middle third of the girder span. Note also that AISC *Specification* Section I3.2c.1(d) requires that steel deck be anchored to all supporting members at a maximum spacing of 18 in. Additionally, *Standard for Composite Steel Floor Deck-Slabs*, ANSI/SDI C1.0-2011 (SDI, 2011), requires deck attachment at an average of 12 in. but no more than 18 in.

From the previous discussion and Figure I.2-4, the total number of stud anchors used is equal to $(13)(2)+3+(13)(2)=55$. A plan layout illustrating the final girder design is provided in Figure I.2-5.

Steel Anchor Ductility Check

As discussed in AISC *Specification* Commentary Section I3.2d, beams are not susceptible to connector failure due to insufficient deformation capacity if they meet one or more of the following conditions:

(1) Beams with span not exceeding 30 ft;
(2) Beams with a degree of composite action of at least 50%; or

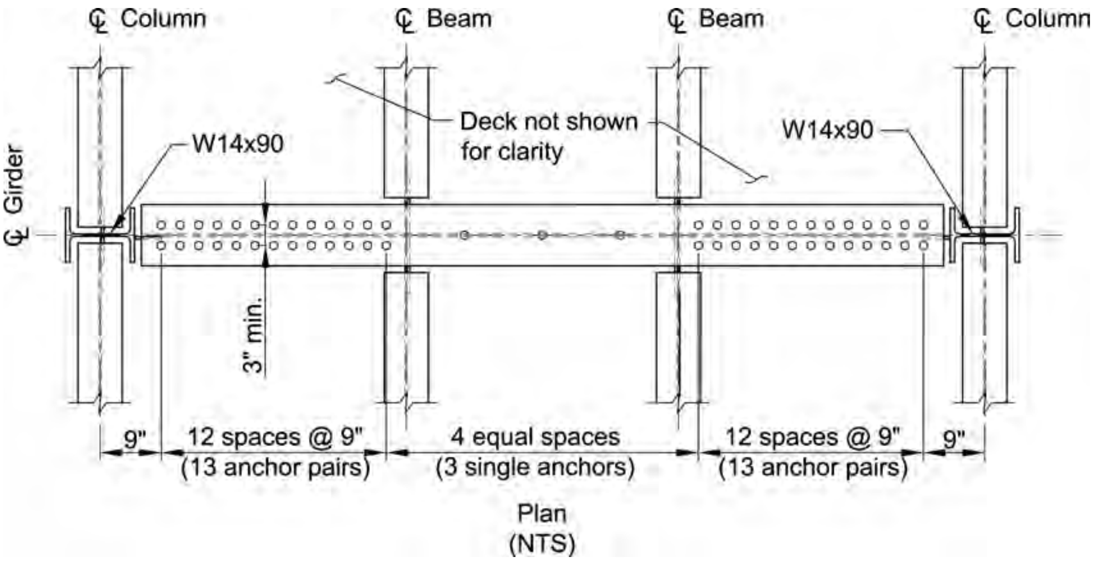

Fig. I.2-4. Steel headed stud anchor layout.

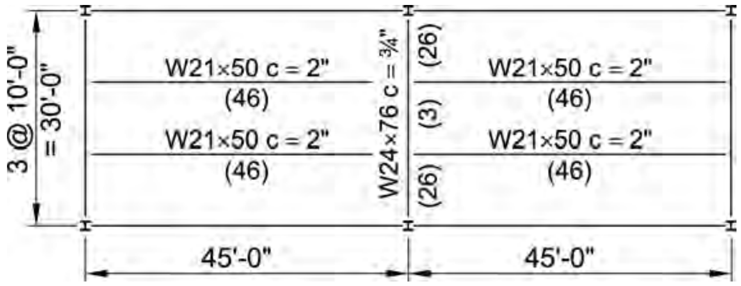

Fig. I.2-5. Revised plan.

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(3) Beams with an average nominal shear connector capacity of at least 16 kips per foot along their span, corresponding to a ¾-in.-diameter steel headed stud anchor placed at 12-in. spacing on average.

The span is 30 ft, which meets the 30 ft limit. The percent composite action is:

$$\begin{aligned}\frac{\sum Q_n}{\min\left\{0.85 f_c' A_c, F_y A_s\right\}} &= \frac{560 \text{ kips}}{\min\left\{0.85(4 \text{ ksi})(540 \text{ in.}^2),\ (50 \text{ ksi})(22.4 \text{ in.}^2)\right\}} \\ &= \frac{560 \text{ kips}}{1{,}120 \text{ kips}}(100) \\ &= 50.0\%\end{aligned}$$

which meets the minimum degree of composite action of 50%. The average shear connector capacity is:

$$\frac{(55 \text{ anchors})(21.5 \text{ kips/anchor})}{30 \text{ ft}} = 39.4 \text{ kip/ft}$$

which exceeds the minimum capacity of 16 kips per foot. Because at least one of the conditions has been met (in fact, all three have been met), the shear connectors meet the ductility requirements.

Live Load Deflection Criteria

Deflections due to live load applied after composite action has been achieved will be limited to $L/360$ under the design live load as required by Table 1604.3 of the *International Building Code* (IBC) (ICC, 2015), or 1 in. using a 50% reduction in design live load as recommended by AISC Design Guide 3.

Deflections for composite members may be determined using the lower bound moment of inertia provided in AISC *Specification* Commentary Equation C-I3-1 and tabulated in AISC *Manual* Table 3-20. The *Specification* Commentary also provides an alternate method for determining deflections through the calculation of an effective moment of inertia. Both methods are acceptable and are illustrated in the following calculations for comparison purposes:

Method 1: Calculation of the lower bound moment of inertia, I_{LB}

$$I_{LB} = I_x + A_s\left(Y_{ENA} - d_3\right)^2 + \left(\frac{\sum Q_n}{F_y}\right)\left(2d_3 + d_1 - Y_{ENA}\right)^2 \qquad \text{(\textit{Spec}. Comm. Eq. C-I3-1)}$$

Variables d_1 and d_3 in AISC *Specification* Commentary Equation C-I3-1 are determined using the same procedure previously illustrated for calculating nominal moment resistance. However, for the determination of I_{LB} the nominal strength of steel anchors is calculated between the point of maximum positive moment and the point of zero moment as opposed to between the concentrated load and point of zero moment used previously. The maximum moment is located at the center of the span and it can be seen from Figure I.2-4 that 27 anchors are located between the midpoint of the beam and each end.

$$\begin{aligned}\Sigma Q_n &= (27\text{ anchors})(21.5\text{ kips/anchor})\\ &= 581\text{ kips}\end{aligned}$$

$$\begin{aligned}a &= \frac{C}{0.85 f_c' b} \qquad \text{(\textit{Spec}. Eq. C-I3-9)}\\ &= \frac{\sum Q_n}{0.85 f_c' b}\\ &= \frac{581\text{ kips}}{0.85(4\text{ ksi})(7.50\text{ ft})(12\text{ in./ft})}\\ &= 1.90\text{ in.}\end{aligned}$$

$$\begin{aligned}d_1 &= t_{slab} - \frac{a}{2}\\ &= 7.50\text{ in.} - \frac{1.90\text{ in.}}{2}\\ &= 6.55\text{ in.}\end{aligned}$$

$$\begin{aligned}x &= \frac{A_s F_y - \sum Q_n}{2 b_f F_y}\\ &= \frac{\left(22.4\text{ in.}^2\right)(50\text{ ksi}) - 581\text{ kips}}{2(8.99\text{ in.})(50\text{ ksi})}\\ &= 0.600\text{ in.} < t_f = 0.680\text{ in.; therefore, the PNA is within the flange}\end{aligned}$$

$$\begin{aligned}d_3 &= \frac{d}{2}\\ &= \frac{23.9\text{ in.}}{2}\\ &= 12.0\text{ in.}\end{aligned}$$

The distance from the top of the steel section to the elastic neutral axis, Y_{ENA}, for use in Equation C-I3-1 is calculated using the procedure provided in AISC *Specification* Commentary Section I3.2 as follows:

$$Y_{ENA} = \frac{A_s d_3 + \left(\frac{\sum Q_n}{F_y}\right)(2d_3 + d_1)}{A_s + \left(\frac{\sum Q_n}{F_y}\right)} \qquad \text{(\textit{Spec}. Comm. Eq. C-I3-2)}$$

$$= \frac{(22.4 \text{ in.}^2)(12.0 \text{ in.}) + \left(\frac{581 \text{ kips}}{50 \text{ ksi}}\right)\left[2(12.0 \text{ in.}) + 6.55 \text{ in.}\right]}{22.4 \text{ in.}^2 + \left(\frac{581 \text{ kips}}{50 \text{ ksi}}\right)}$$

$$= 18.3 \text{ in.}$$

Substituting these values into AISC *Specification* Commentary Equation C-I3-1 yields the following lower bound moment of inertia:

$$I_{LB} = 2{,}100 \text{ in.}^4 + (22.4 \text{ in.}^2)(18.3 \text{ in.} - 12.0 \text{ in.})^2 + \left(\frac{581 \text{ kips}}{50 \text{ ksi}}\right)\left[2(12.0 \text{ in.}) + 6.55 \text{ in.} - 18.3 \text{ in.}\right]^2$$

$$= 4{,}730 \text{ in.}^4$$

Alternately, this value can be determined directly from AISC *Manual* Table 3-20 as illustrated in Design Example I.1.

Method 2: Calculation of the equivalent moment of inertia, I_{equiv}

An alternate procedure for determining a moment of inertia for the deflection calculation of the composite section is presented in AISC *Specification* Commentary Section I3.2 and in the following:

Determine the transformed moment of inertia, I_{tr}

The effective width of the concrete below the top of the deck may be approximated with the deck profile resulting in a 50% effective width as depicted in Figure I.2-6. The effective width, b_{eff} = (7.50 ft)(12 in./ft) = 90.0 in.

Transformed slab widths are calculated as follows:

$$n = \frac{E_s}{E_c}$$

$$= \frac{29{,}000 \text{ ksi}}{3{,}490 \text{ ksi}}$$

$$= 8.31$$

$$b_{tr1} = \frac{b_{eff}}{n}$$

$$= \frac{90.0 \text{ in.}}{8.31}$$

$$= 10.8 \text{ in.}$$

$$
\begin{aligned}
b_{tr2} &= \frac{0.5b_{eff}}{n} \\
&= \frac{0.5(90.0 \text{ in.})}{8.31} \\
&= 5.42 \text{ in.}
\end{aligned}
$$

The transformed model is illustrated in Figure I.2-7.

Determine the elastic neutral axis of the transformed section (assuming fully composite action) and calculate the transformed moment of inertia using the information provided in Table I.2-1 and Figure I.2-7. For this problem, a trial location for the elastic neutral axis (ENA) is assumed to be within the depth of the composite deck.

Table I.2-1. Properties for Elastic Neutral Axis Determination of Transformed Section

Part	A, in.^{2}	y, in.	I, in.^{4}
A_1	48.6	$2.25 + x$	82.0
A_2	$5.42x$	$x/2$	$0.452x^3$
W24×76	22.4	$x - 15.0$	2,100

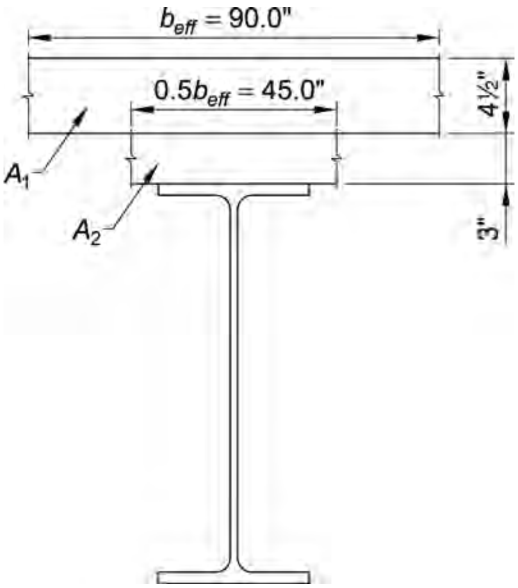


Fig. I.2-6. Effective concrete width.

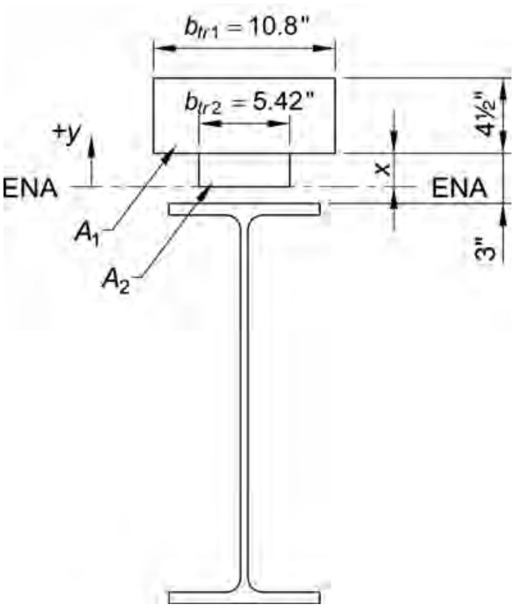


Fig. I.2-7. Transformed area model.

ΣA_y about elastic neutral axis = 0

$$\left(48.6\text{ in.}^2\right)\left(2.25\text{ in.}+x\right)+\left(5.42\text{ in.}\right)\left(\frac{x^2}{2}\right)+\left(22.4\text{ in.}^2\right)\left(x-15.0\text{ in.}\right)=0$$

Solving for *x*:

$$x = 2.88\text{ in.}$$

Verify trial location:

2.88 in. $< h_r = 3$ in.; therefore, the elastic neutral axis is within the composite deck

Utilizing the parallel axis theorem and substituting for *x* yields:

$$\begin{aligned}I_{tr} &= \Sigma I + \Sigma A y^2\\ &= 82.0\text{ in.}^4 + \left(0.452\text{ in.}\right)\left(2.88\text{ in.}\right)^3 + 2{,}100\text{ in.}^4 + \left(48.6\text{ in.}^2\right)\left(2.25\text{ in.}+2.88\text{ in.}\right)^2 + \left(15.6\text{ in.}^2\right)\left(\frac{2.88\text{ in.}}{2}\right)^2\\ &\quad + \left(22.4\text{ in.}^2\right)\left(2.88\text{ in.}-15.0\text{ in.}\right)^2\\ &= 6{,}800\text{ in.}^4\end{aligned}$$

Determine the equivalent moment of inertia, I_{equiv}

$\Sigma Q_n = 581$ kips (previously determined in Method 1)

C_f = compression force for fully composite beam previously determined to be controlled by $A_sF_y = 1{,}120$ kips

$$
\begin{aligned}
I_{equiv} &= I_s + \sqrt{\left(\Sigma Q_n / C_f\right)}\left(I_{tr} - I_s\right) \qquad \text{(}\textit{Spec}\text{. Comm. Eq. C-I3-3)}\\
&= 2{,}100 \text{ in.}^4 + \sqrt{(581 \text{ kips})/(1{,}120 \text{ kips})}\left(6{,}800 \text{ in.}^4 - 2{,}100 \text{ in.}^4\right)\\
&= 5{,}490 \text{ in.}^4
\end{aligned}
$$

Comparison of Methods and Final Deflection Calculation

I_{LB} was determined to be 4,730 in.^{4} and I_{equiv} was determined to be 5,490 in.^{4} I_{LB} will be used for the remainder of this example.

From AISC *Manual* Table 3-23, Case 9:

$$
\begin{aligned}
\Delta_{LL} &= \frac{23P_L L^3}{648EI_{LB}}\\
&= \frac{23(45.0 \text{ kips})\left[(30 \text{ ft})(12 \text{ in./ft})\right]^3}{648(29{,}000 \text{ ksi})\left(4{,}730 \text{ in.}^4\right)}\\
&= 0.543 \text{ in.} < 1.00 \text{ in. (for AISC Design Guide 3 limit)} \quad \textbf{o.k.}
\end{aligned}
$$

(50% reduction in design live load as allowed by Design Guide 3 was not necessary to meet this limit)

$$
= L/662 < L/360 \text{ (for IBC 2015 Table 1604.3 limit)} \quad \textbf{o.k.}
$$

Available Shear Strength

According to AISC *Specification* Section I4.2, the girder should be assessed for available shear strength as a bare steel beam using the provisions of Chapter G.

Applying the loads previously determined for the governing load combination of ASCE/SEI 7 and obtaining available shear strengths from AISC *Manual* Table 3-2 for a W24×76 yields the following:

LRFD	ASD
$V_u = 121 \text{ kips} + (0.0912 \text{ kip/ft})\left(\frac{30 \text{ ft}}{2}\right)$ $= 122 \text{ kips}$ $\phi_v V_n = 315 \text{ kips} > 122 \text{ kips}$ **o.k.**	$V_a = 85.5 \text{ kips} + (0.0760 \text{ kip/ft})\left(\frac{30 \text{ ft}}{2}\right)$ $= 86.6 \text{ kips}$ $\frac{V_n}{\Omega_v} = 210 \text{ kips} > 86.6 \text{ kips}$ **o.k.**

Serviceability

Depending on the intended use of this bay, vibrations might need to be considered. See AISC Design Guide 11 (Murray et al., 2016) for additional information.

It has been observed that cracking of composite slabs can occur over girder lines. The addition of top reinforcing steel transverse to the girder span will aid in mitigating this effect.

Summary

Using LRFD design methodology, it has been determined that a W24×76 with ¾ in. of camber and 55, ¾-in.-diameter by 4⅞-in.-long steel headed stud anchors as depicted in Figure I.2-4, is adequate for the imposed loads and deflection criteria. Using ASD design methodology, a W24×84 with a steel headed stud anchor layout determined using a procedure analogous to the one demonstrated in this example would be required.

EXAMPLE I.3 FILLED COMPOSITE MEMBER FORCE ALLOCATION AND LOAD TRANSFER

Given:

Refer to Figure I.3-1.

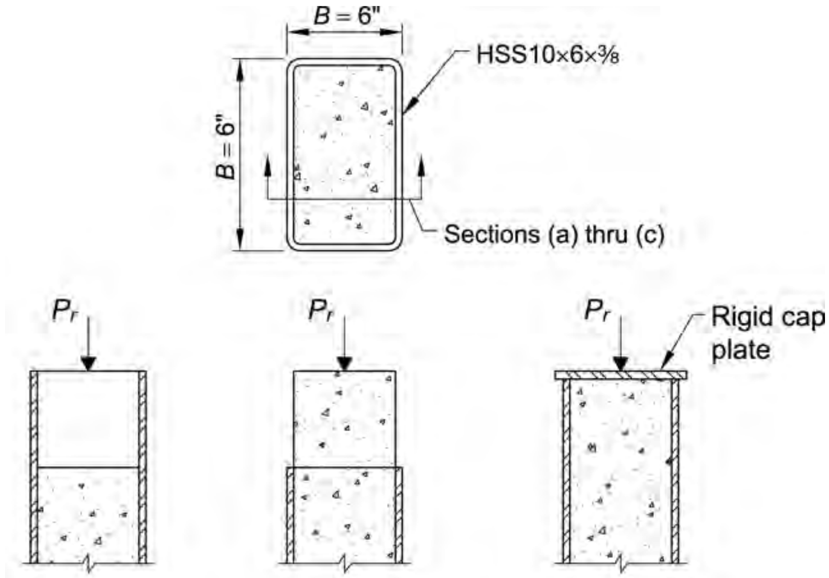
Part I: For each loading condition (a) through (c) determine the required longitudinal shear force, V_r', to be transferred between the steel section and concrete fill.

Part II: For loading condition (a), investigate the force transfer mechanisms of direct bearing, shear connection, and direct bond interaction.

The composite member consists of an ASTM A500, Grade C, HSS with normal weight (145 lb/ft^3) concrete fill having a specified concrete compressive strength, $f_c' = 5$ ksi. Use ASTM A36 material for the bearing plate.

Applied loading, P_r, for each condition illustrated in Figure I.3-1 is composed of the following nominal loads:

$P_D = 32$ kips
$P_L = 84$ kips



(a) External force to steel only

(b) External force to concrete only

(c) External force to both materials concurrently

Fig. I.3-1. Filled composite member in compression.

Solution:

Part I—Force Allocation

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500 Grade C
F_y = 50 ksi
F_u = 62 ksi

From AISC *Manual* Table 1-11 and Figure I.3-1, the geometric properties are as follows:

HSS10×6×⅜
A_s = 10.4 in.^{2}
H = 10.0 in.
B = 6.00 in.
t_{nom} = ⅜ in. (nominal wall thickness)
t = 0.349 in. (design wall thickness in accordance with AISC *Specification* Section B4.2)
h/t = 25.7
b/t = 14.2

Calculate the concrete area using geometry compatible with that used in the calculation of the steel area in AISC *Manual* Table 1-11 (taking into account the design wall thickness and an outside corner radii of two times the design wall thickness in accordance with AISC *Manual* Part 1), as follows:

$$\begin{aligned} h_i &= H - 2t \\ &= 10.0 \text{ in.} - 2(0.349 \text{ in.}) \\ &= 9.30 \text{ in.} \end{aligned}$$

$$\begin{aligned} b_i &= B - 2t \\ &= 6.00 \text{ in.} - 2(0.349 \text{ in.}) \\ &= 5.30 \text{ in.} \end{aligned}$$

$$\begin{aligned} A_c &= b_i h_i - t^2(4 - \pi) \\ &= (5.30 \text{ in.})(9.30 \text{ in.}) - (0.349)^2(4 - \pi) \\ &= 49.2 \text{ in.}^2 \end{aligned}$$

From ASCE/SEI 7, Chapter 2, the required compressive strength is:

LRFD	ASD
$P_r = P_u$ $= 1.2(32 \text{ kips}) + 1.6(84 \text{ kips})$ $= 173 \text{ kips}$	$P_r = P_a$ $= 32 \text{ kips} + 84 \text{ kips}$ $= 116 \text{ kips}$

Composite Section Strength for Force Allocation

In order to determine the composite section strength for force allocation, the member is first classified as compact, noncompact or slender in accordance with AISC *Specification* Table I1.1a.

Governing Width-to-Thickness Ratio

$$\begin{aligned}\lambda &= \frac{h}{t}\\ &= 25.7\end{aligned}$$

The limiting width-to-thickness ratio for a compact compression steel element in a composite member subject to axial compression is:

$$\begin{aligned}\lambda_p &= 2.26\sqrt{\frac{E}{F_y}} \qquad \text{(}\textit{Spec}\text{. Table I1.1a)}\\ &= 2.26\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}\\ &= 54.4 > 25.7;\ \text{therefore the HSS wall is compact}\end{aligned}$$

The nominal axial compressive strength without consideration of length effects, P_{no}, used for force allocation calculations is therefore determined as:

$$P_{no} = P_p \qquad \text{(}\textit{Spec}\text{. Eq. I2-9a)}$$

$$P_p = F_y A_s + C_2 f_c'\left(A_c + A_{sr}\frac{E_s}{E_c}\right) \qquad \text{(}\textit{Spec}\text{. Eq. I2-9b)}$$

where
C_2 = 0.85 for rectangular sections
A_{sr} = 0 in.^{2} when no reinforcing steel is present within the HSS

$$\begin{aligned}P_{no} &= F_y A_s + C_2 f_c'\left(A_c + A_{sr}\frac{E_s}{E_c}\right)\\ &= (50\text{ ksi})(10.4\text{ in.}^2) + 0.85(5\text{ ksi})(49.2\text{ in.}^2 + 0\text{ in.}^2)\\ &= 729\text{ kips}\end{aligned}$$

Transfer Force for Condition (a)

Refer to Figure I.3-1(a). For this condition, the entire external force is applied to the steel section only, and the provisions of AISC *Specification* Section I6.2a apply.

$$\begin{aligned}V_r' &= P_r\left(1 - \frac{F_y A_s}{P_{no}}\right) \qquad \text{(}\textit{Spec}\text{. Eq. I6-1)}\\ &= P_r\left[1 - \frac{(50\text{ ksi})(10.4\text{ in.}^2)}{729\text{ kips}}\right]\\ &= 0.287P_r\end{aligned}$$

LRFD	ASD
$V_r' = 0.287(173\text{ kips})$ $= 49.7\text{ kips}$	$V_r' = 0.287(116\text{ kips})$ $= 33.3\text{ kips}$

Transfer Force for Condition (b)

Refer to Figure I.3-1(b). For this condition, the entire external force is applied to the concrete fill only, and the provisions of AISC *Specification* Section I6.2b apply.

$$V_r' = P_r\left(\frac{F_y A_s}{P_{no}}\right) \qquad (\textit{Spec}.\ \text{Eq. I6-2a})$$

$$= P_r\left[\frac{(50\text{ ksi})(10.4\text{ in.}^2)}{729\text{ kips}}\right]$$

$$= 0.713P_r$$

LRFD	ASD
$V_r' = 0.713(173\text{ kips})$ $= 123\text{ kips}$	$V_r' = 0.713(116\text{ kips})$ $= 82.7\text{ kips}$

Transfer Force for Condition (c)

Refer to Figure I.3-1(c). For this condition, external force is applied to the steel section and concrete fill concurrently, and the provisions of AISC *Specification* Section I6.2c apply.

AISC *Specification* Commentary Section I6.2 states that when loads are applied to both the steel section and concrete fill concurrently, V_r' can be taken as the difference in magnitudes between the portion of the external force applied directly to the steel section and that required by Equation I6-2a and b. Using the plastic distribution approach employed in AISC *Specification* Equations I6-1 and I6-2a, this concept can be written in equation form as follows:

$$V_r' = \left|P_{rs} - P_r\left(\frac{A_s F_y}{P_{no}}\right)\right| \qquad (\text{Eq. 1})$$

where

P_{rs} = portion of external force applied directly to the steel section, kips

Note that this example assumes the external force imparts compression on the composite element as illustrated in Figure I.3-1. If the external force would impart tension on the composite element, consult the AISC *Specification* Commentary for discussion.

Currently the *Specification* provides no specific requirements for determining the distribution of the applied force for the determination of P_{rs}, so it is left to engineering judgment. For a bearing plate condition such as the one represented in Figure I.3-1(c), one possible method for determining the distribution of applied forces is to use an elastic distribution based on the material axial stiffness ratios as follows:

$$E_c = w_c^{1.5}\sqrt{f_c'}$$

$$= \left(145\text{ lb/ft}^3\right)^{1.5}\sqrt{5\text{ ksi}}$$

$$= 3{,}900\text{ ksi}$$

$$
\begin{aligned}
P_{rs} &= \left(\frac{E_s A_s}{E_s A_s + E_c A_c}\right) P_r \\
&= \left[\frac{(29{,}000\text{ ksi})(10.4\text{ in.}^2)}{(29{,}000\text{ ksi})(10.4\text{ in.}^2) + (3{,}900\text{ ksi})(49.2\text{ in.}^2)}\right] P_r \\
&= 0.611 P_r
\end{aligned}
$$

Substituting the results into Equation 1 yields:

$$
\begin{aligned}
V_r' &= \left| 0.611 P_r - P_r \left(\frac{A_s F_y}{P_{no}}\right) \right| \\
&= \left| 0.611 P_r - P_r \left[\frac{(10.4\text{ in.}^2)(50\text{ ksi})}{729\text{ kips}}\right] \right| \\
&= 0.102 P_r
\end{aligned}
$$

LRFD	ASD
$V_r' = 0.102(173\text{ kips})$ $= 17.6\text{ kips}$	$V_r' = 0.102(116\text{ kips})$ $= 11.8\text{ kips}$

An alternate approach would be the use of a plastic distribution method whereby the load is partitioned to each material in accordance with their contribution to the composite section strength given in Equation I2-9b. This method eliminates the need for longitudinal shear transfer provided the local bearing strength of the concrete and steel are adequate to resist the forces resulting from this distribution.

Additional Discussion

- The design and detailing of the connections required to deliver external forces to the composite member should be performed according to the applicable sections of AISC *Specification* Chapters J and K. Note that for checking bearing strength on concrete confined by a steel HSS or box member, the $\sqrt{A_2 / A_1}$ term in Equation J8-2 may be taken as 2.0 according to the User Note in *Specification* Section I6.2.

- The connection cases illustrated by Figure I.3-1 are idealized conditions representative of the mechanics of actual connections. For instance, a standard shear connection welded to the face of an HSS column is an example of a condition where all external force is applied directly to the steel section only. Note that the connection configuration can also impact the strength of the force transfer mechanism as illustrated in Part II of this example.

Solution:

Part II—Load Transfer

The required longitudinal force to be transferred, V_r', determined in Part I condition (a) will be used to investigate the three applicable force transfer mechanisms of AISC *Specification* Section I6.3: direct bearing, shear connection, and direct bond interaction. As indicated in the *Specification*, these force transfer mechanisms may not be superimposed; however, the mechanism providing the greatest nominal strength may be used.

Direct Bearing

Trial Layout of Bearing Plate

For investigating the direct bearing load transfer mechanism, the external force is delivered directly to the HSS section by standard shear connections on each side of the member as illustrated in Figure I.3-2. One method for utilizing direct bearing in this instance is through the use of an internal bearing plate. Given the small clearance within the HSS section under consideration, internal access for welding is limited to the open ends of the HSS; therefore, the HSS section will be spliced at the bearing plate location. Additionally, it is a practical consideration that no more than 50% of the internal width of the HSS section be obstructed by the bearing plate in order to facilitate concrete placement. It is essential that concrete mix proportions and installation of concrete fill produce full bearing above and below the projecting plate. Based on these considerations, the trial bearing plate layout depicted in Figure I.3-2 was selected using an internal plate protrusion, L_p, of 1.0 in.

Location of Bearing Plate

The bearing plate is placed within the load introduction length discussed in AISC *Specification* Section I6.4b. The load introduction length is defined as two times the minimum transverse dimension of the HSS both above and below the load transfer region. The load transfer region is defined in *Specification* Commentary Section I6.4 as the depth of the connection. For the configuration under consideration, the bearing plate should be located within 2(B = 6 in.) = 12 in. of the bottom of the shear connection. From Figure I.3-2, the location of the bearing plate is 6 in. from the bottom of the shear connection and is therefore adequate.

Available Strength for the Limit State of Direct Bearing

The contact area between the bearing plate and concrete, A_1, may be determined as follows:

$$A_1 = A_c - (b_i - 2L_p)(h_i - 2L_p) \qquad \text{(Eq. 2)}$$

where

L_p = typical protrusion of bearing plate inside HSS
= 1.0 in.

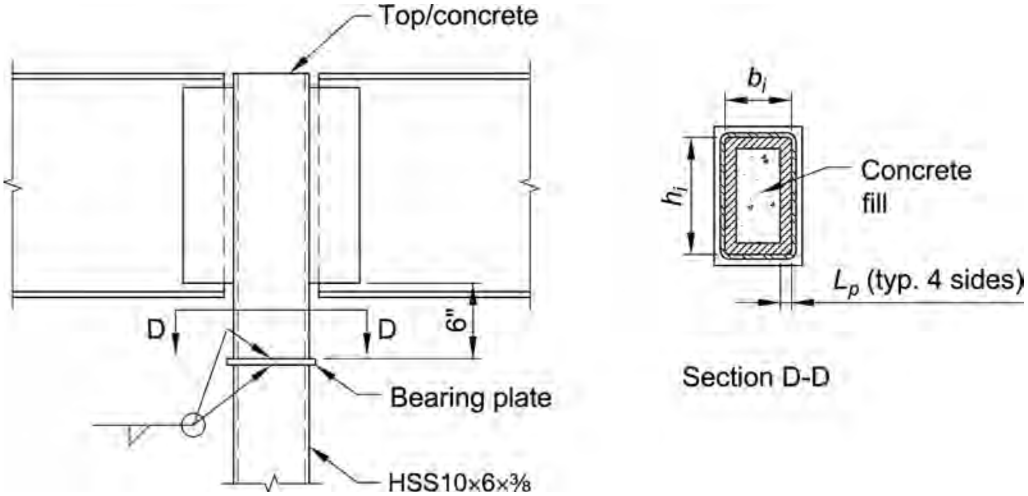


Fig. I.3-2. Internal bearing plate configuration.

Substituting for the appropriate geometric properties previously determined in Part I into Equation 2 yields:

$$\begin{aligned}A_1 &= 49.2\text{ in.}^2 - \left[5.30\text{ in.} - 2(1.0\text{ in.})\right]\left[9.30\text{ in.} - 2(1.0\text{ in.})\right]\\ &= 25.1\text{ in.}^2\end{aligned}$$

The available strength for the direct bearing force transfer mechanism is:

$$R_n = 1.7 f_c' A_1 \qquad (\textit{Spec}.\text{ Eq. I6-3})$$

LRFD	ASD
$\phi_B = 0.65$	$\Omega_B = 2.31$
$\phi_B R_n = 0.65(1.7)(5\text{ ksi})(25.1\text{ in.}^2)$ $= 139\text{ kips} > V_r' = 49.7\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega_B} = \dfrac{1.7(5\text{ ksi})(25.1\text{ in.}^2)}{2.31}$ $= 92.4\text{ kips} > V_r' = 33.3\text{ kips}$ **o.k.**

Required Thickness of Internal Bearing Plate

There are several methods available for determining the bearing plate thickness. For round HSS sections with circular bearing plate openings, a closed-form elastic solution such as those found in *Roark's Formulas for Stress and Strain* (Young and Budynas, 2002) may be used. Alternately, the use of computational methods such as finite element analysis may be employed.

For this example, yield line theory can be employed to determine a plastic collapse mechanism of the plate. In this case, the walls of the HSS lack sufficient stiffness and strength to develop plastic hinges at the perimeter of the bearing plate. Utilizing only the plate material located within the HSS walls, and ignoring the HSS corner radii, the yield line pattern is as depicted in Figure I.3-3.

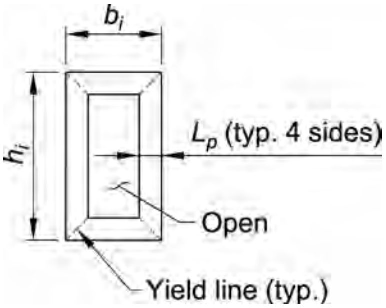


Fig. I.3-3. Yield line pattern.

Utilizing the results of the yield line analysis with $F_y = 36$ ksi plate material, the plate thickness may be determined as follows:

LRFD	ASD
$\phi = 0.90$ $t_p = \sqrt{\frac{w_u}{2\phi F_y}\left[L_p\left(b_i + h_i\right) - \frac{8L_p^2}{3}\right]}$ where w_u = bearing pressure on plate determined using LRFD load combinations $= \frac{V_r'}{A_1}$ $= \frac{49.7\text{ kips}}{25.1\text{ in.}^2}$ $= 1.98\text{ ksi}$ $t_p = \sqrt{\left[\frac{1.98\text{ ksi}}{2(0.90)(36\text{ ksi})}\right] \times \left[(1.0\text{ in.})(5.30\text{ in.} + 9.30\text{ in.}) - \frac{8(1.0\text{ in.})^2}{3}\right]}$ $= 0.604\text{ in.}$	$\Omega = 1.67$ $t_p = \sqrt{\frac{\Omega w_a}{2F_y}\left[L_p\left(b_i + h_i\right) - \frac{8L_p^2}{3}\right]}$ where w_a = bearing pressure on plate determined using ASD load combinations $= \frac{V_r'}{A_1}$ $= \frac{33.3\text{ kips}}{25.1\text{ in.}^2}$ $= 1.33\text{ ksi}$ $t_p = \sqrt{\left[\frac{(1.67)(1.33\text{ ksi})}{2(36\text{ ksi})}\right] \times \left[(1.0\text{ in.})(5.30\text{ in.} + 9.30\text{ in.}) - \frac{8(1.0\text{ in.})^2}{3}\right]}$ $= 0.607\text{ in.}$

Thus, select a ¾-in.-thick bearing plate.

Splice Weld

The HSS is in compression due to the imposed loads, therefore the splice weld indicated in Figure I.3-2 is sized according to the minimum weld size requirements of Chapter J. Should uplift or flexure be applied in other loading conditions, the splice should be designed to resist these forces using the applicable provisions of AISC *Specification* Chapters J and K.

Shear Connection

Shear connection involves the use of steel headed stud or channel anchors placed within the HSS section to transfer the required longitudinal shear force. The use of the shear connection mechanism for force transfer in filled HSS is usually limited to large HSS sections and built-up box shapes, and is not practical for the composite member in question. Consultation with the fabricator regarding their specific capabilities is recommended to determine the feasibility of shear connection for HSS and box members. Should shear connection be a feasible load transfer mechanism, AISC *Specification* Section I6.3b in conjunction with the steel anchors in composite component provisions of Section I8.3 apply.

Direct Bond Interaction

The use of direct bond interaction for load transfer is limited to filled HSS and depends upon the location of the load transfer point within the length of the member being considered (end or interior) as well as the number of faces to which load is being transferred.

From AISC *Specification* Section I6.3c, the nominal bond strength for a rectangular section is:

$$R_n = p_b L_{in} F_{in} \qquad (Spec.\ \text{Eq. I6-5})$$

where

p_b = perimeter of the steel-concrete bond interface within the composite cross section, in.

$$= (2)(10.0\text{ in.} + 6.00\text{ in.}) - (8)\left[(2)(0.349\text{ in.})\right] + (4)\left[\frac{\pi(0.349\text{ in.})}{2}\right]$$

$$= 28.6\text{ in.}$$

L_{in} = load introduction length, determined in accordance with AISC *Specification* Section I6.4

$$= 2\left[\min(B, H)\right]$$
$$= 2(6.00\text{ in.})$$
$$= 12.0\text{ in.}$$

$$F_{in} = \frac{12t}{H^2} \le 0.1\text{, ksi (for a rectangular cross section)}$$

$$= \frac{12(0.349\text{ in.})}{(10.0\text{ in.})^2} \le 0.1\text{ ksi}$$

$$= 0.0419\text{ ksi}$$

For the design of this load transfer mechanism, two possible cases will be considered:

Case 1: End Condition—Load Transferred to Member from Four Sides Simultaneously

For this case the member is loaded at an end condition (the composite member only extends to one side of the point of force transfer). Force is applied to all four sides of the section simultaneously thus allowing the full perimeter of the section to be mobilized for bond strength.

From AISC *Specification* Equation I6-5:

LRFD	ASD
$\phi = 0.50$ $\phi R_n = \phi p_b L_{in} F_{in}$ $= 0.50(28.6\text{ in.})(12.0\text{ in.})(0.0419\text{ ksi})$ $= 7.19\text{ kips} < V_r' = 49.7\text{ kips}$ **n.g.**	$\Omega = 3.00$ $\dfrac{R_n}{\Omega} = \dfrac{p_b L_{in} F_{in}}{\Omega}$ $= \dfrac{(28.6\text{ in.})(12.0\text{ in.})(0.0419\text{ ksi})}{3.00}$ $= 4.79\text{ kips} < V_r' = 33.3\text{ kips}$ **n.g.**

Bond strength is inadequate and another force transfer mechanism such as direct bearing must be used to meet the load transfer provisions of AISC *Specification* Section I6.

Alternately, the detail could be revised so that the external force is applied to both the steel section and concrete fill concurrently as schematically illustrated in Figure I.3-1(c). Comparing bond strength to the load transfer requirements for concurrent loading determined in Part I of this example yields:

LRFD	ASD
$\phi = 0.50$ $\phi R_n = 7.19\text{ kips} < V_r' = 17.6\text{ kips}$ **n.g.**	$\Omega = 3.00$ $\dfrac{R_n}{\Omega} = 4.79\text{ kips} < V_r' = 11.8\text{ kips}$ **n.g.**

Bond strength remains inadequate and another force transfer mechanism such as direct bearing must be used to meet the load transfer provisions of AISC *Specification* Section I6.

Case 2: Interior Condition—Load Transferred to Three Faces

For this case the composite member is loaded from three sides away from the end of the member (the composite member extends to both sides of the point of load transfer) as indicated in Figure I.3-4.

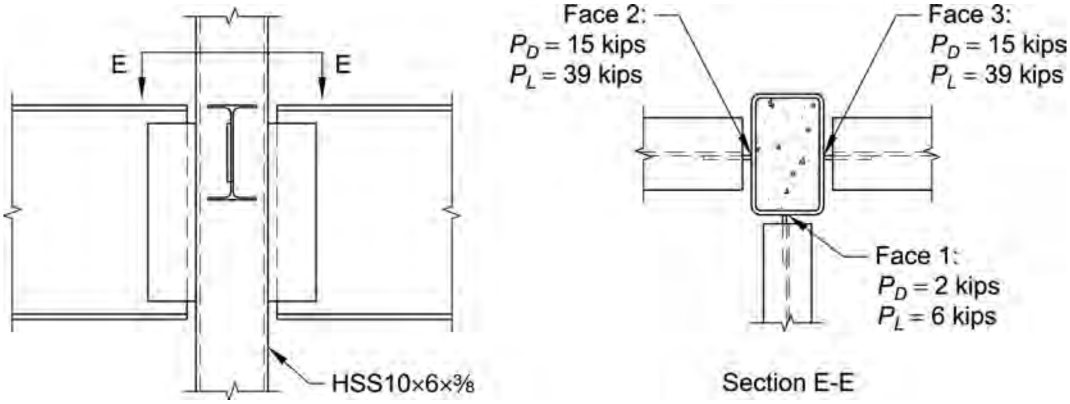


Fig. I.3-4. Case 2 load transfer.

Longitudinal shear forces to be transferred at each face of the HSS are calculated using the relationship to external forces determined in Part I of the example for condition (a) shown in Figure I.3-1, and the applicable ASCE/SEI 7 load combinations as follows:

LRFD	ASD
Face 1: $P_{r1} = P_u$ $= 1.2(2\text{ kips}) + 1.6(6\text{ kips})$ $= 12.0\text{ kips}$ $V_{r1}' = 0.287P_{r1}$ $= 0.287(12.0\text{ kips})$ $= 3.44\text{ kips}$	Face 1: $P_{r1} = P_a$ $= 2\text{ kips} + 6\text{ kips}$ $= 8.00\text{ kips}$ $V_{r1}' = 0.287P_{r1}$ $= 0.287(8.00\text{ kips})$ $= 2.30\text{ kips}$

LRFD	ASD
Faces 2 and 3: $P_{r2-3} = P_u$ $= 1.2(15 \text{ kips}) + 1.6(39 \text{ kips})$ $= 80.4 \text{ kips}$ $V'_{r2-3} = 0.287 P_{r2-3}$ $= 0.287(80.4 \text{ kips})$ $= 23.1 \text{ kips}$	Faces 2 and 3: $P_{r2-3} = P_u$ $= 15 \text{ kips} + 39 \text{ kips}$ $= 54.0 \text{ kips}$ $V'_{r2-3} = 0.287 P_{r2-3}$ $= 0.287(54.0 \text{ kips})$ $= 15.5 \text{ kips}$

Load transfer at each face of the section is checked separately for the longitudinal shear at that face using Equation I6-5 as follows:

LRFD	ASD
$\phi = 0.50$ Face 1: $p_b = 6.00 \text{ in.} - (2 \text{ corners})(2)(0.349 \text{ in.})$ $= 4.60 \text{ in.}$ $\phi R_{n1} = 0.50(4.60 \text{ in.})(12.0 \text{ in.})(0.0419 \text{ ksi})$ $= 1.16 \text{ kips} < V'_{r1} = 3.44 \text{ kips}$ **n.g.** Faces 2 and 3: $p_b = 10.0 \text{ in.} - (2 \text{ corners})(2)(0.349 \text{ in.})$ $= 8.60 \text{ in.}$ $\phi R_{n2-3} = 0.50(8.60 \text{ in.})(12.0 \text{ in.})(0.0419 \text{ ksi})$ $= 2.16 \text{ kips} < V'_{r2-3} = 23.1 \text{ kips}$ **n.g.**	$\Omega = 3.00$ Face 1: $p_b = 6.00 \text{ in.} - (2 \text{ corners})(2)(0.349 \text{ in.})$ $= 4.60 \text{ in.}$ $\dfrac{R_{n1}}{\Omega} = \dfrac{(4.60 \text{ in.})(12.0 \text{ in.})(0.0419 \text{ ksi})}{3.00}$ $= 0.771 \text{ kip} < V'_{r1} = 2.30 \text{ kips}$ **n.g.** Faces 2 and 3: $p_b = 10.0 \text{ in.} - (2 \text{ corners})(2)(0.349 \text{ in.})$ $= 8.60 \text{ in.}$ $\dfrac{R_{n2-3}}{\Omega} = \dfrac{(8.60 \text{ in.})(12.0 \text{ in.})(0.0419 \text{ ksi})}{3.00}$ $= 1.44 \text{ kips} < V'_{r2-3} = 15.5 \text{ kips}$ **n.g.**

The calculations indicate that the bond strength is inadequate for all faces, thus an alternate means of load transfer such as the use of internal bearing plates as demonstrated previously in this example is necessary.

As demonstrated by this example, direct bond interaction provides limited available strength for transfer of longitudinal shears and is generally only acceptable for lightly loaded columns or columns with low shear transfer requirements such as those with loads applied to both concrete fill and steel encasement simultaneously.

EXAMPLE I.4 FILLED COMPOSITE MEMBER IN AXIAL COMPRESSION

Given:

Determine if the filled composite member illustrated in Figure I.4-1 is adequate for the indicated dead and live loads. Table IV-1B in Part IV will be used in this example.

The composite member consists of an ASTM A500 Grade C HSS with normal weight (145 lb/ft^3) concrete fill having a specified concrete compressive strength, f_c' = 5 ksi.

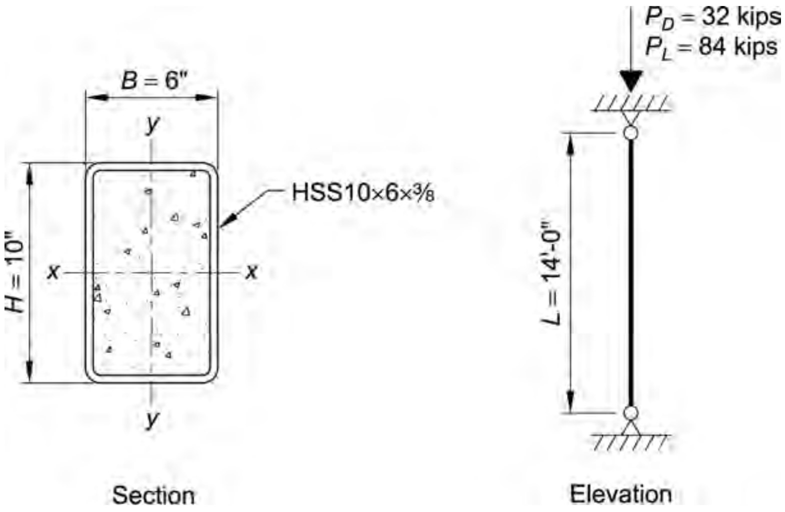


Fig. I.4-1. Filled composite member section and applied loading.

Solution:

From AISC *Manual* Table 2-4, the material properties are:

ASTM A500 Grade C
F_y = 50 ksi
F_u = 62 ksi

From ASCE/SEI 7, Chapter 2, the required compressive strength is:

LRFD	ASD
$P_r = P_u$ $= 1.2(32\text{ kips}) + 1.6(84\text{ kips})$ $= 173\text{ kips}$	$P_r = P_a$ $= 32\text{ kips} + 84\text{ kips}$ $= 116\text{ kips}$

Method 1: AISC Tables

The most direct method of calculating the available compressive strength is through the use of Table IV-1B (Part IV of this document). A K factor of 1.0 is used for a pin-ended member. Because the unbraced length is the same in both the *x-x* and *y-y* directions, and I_x exceeds I_y, *y-y* axis buckling will govern.

Entering Table IV-1B with $L_{cy} = KL_y$ = 14 ft yields:

LRFD	ASD
$\phi_c P_n = 368 \text{ kips} > 173 \text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_c} = 245 \text{ kips} > 116 \text{ kips}$ **o.k.**

Method 2: AISC Specification Calculations

As an alternate to using Table IV-1B, the available compressive strength can be calculated directly using the provisions of AISC *Specification* Chapter I.

From AISC *Manual* Table 1-11 and Figure I.4-1, the geometric properties of an HSS10×6×⅜ are as follows:

A_s = 10.4 in.^{2}
H = 10.0 in.
B = 6.00 in.
t_{nom} = ⅜ in. (nominal wall thickness)
t = 0.349 in. (design wall thickness)
h/t = 25.7
b/t = 14.2
I_{sx} = 137 in.^{4}
I_{sy} = 61.8 in.^{4}

As shown in Figure I.1-1, internal clear distances are determined as:

$$\begin{aligned} h_i &= H - 2t \\ &= 10.0 \text{ in.} - 2(0.349 \text{ in.}) \\ &= 9.30 \text{ in.} \end{aligned}$$

$$\begin{aligned} b_i &= B - 2t \\ &= 6.00 \text{ in.} - 2(0.349 \text{ in.}) \\ &= 5.30 \text{ in.} \end{aligned}$$

From Design Example I.3, the area of concrete, A_c, equals 49.2 in.^{2} The steel and concrete areas can be used to calculate the gross cross-sectional area as follows:

$$\begin{aligned} A_g &= A_s + A_c \\ &= 10.4 \text{ in.}^2 + 49.2 \text{ in.}^2 \\ &= 59.6 \text{ in.}^2 \end{aligned}$$

Calculate the concrete moment of inertia using geometry compatible with that used in the calculation of the steel area in AISC *Manual* Table 1-11 (taking into account the design wall thickness and corner radii of two times the design wall thickness in accordance with AISC *Manual* Part 1), the following equations may be used, based on the terminology given in Figure I-1 in the introduction to these examples:

For bending about the *x-x* axis:

$$
\begin{aligned}
I_{cx} &= \frac{(B-4t)h_i^3}{12} + \frac{t(H-4t)^3}{6} + \frac{(9\pi^2-64)t^4}{36\pi} + \pi t^2\left(\frac{H-4t}{2}+\frac{4t}{3\pi}\right)^2 \\
&= \frac{[6.00\text{ in.}-4(0.349\text{ in.})](9.30\text{ in.})^3}{12} + \frac{(0.349\text{ in.})[10.0\text{ in.}-4(0.349\text{ in.})]^3}{6} + \frac{(9\pi^2-64)(0.349\text{ in.})^4}{36\pi} \\
&\quad + \pi(0.349\text{ in.})^2\left[\frac{10.0\text{ in.}-4(0.349\text{ in.})}{2}+\frac{4(0.349\text{ in.})}{3\pi}\right]^2 \\
&= 353\text{ in.}^4
\end{aligned}
$$

For bending about the *y-y* axis:

$$
\begin{aligned}
I_{cy} &= \frac{(H-4t)b_i^3}{12} + \frac{t(B-4t)^3}{6} + \frac{(9\pi^2-64)t^4}{36\pi} + \pi t^2\left(\frac{B-4t}{2}+\frac{4t}{3\pi}\right)^2 \\
&= \frac{[10.0\text{ in.}-4(0.349\text{ in.})](5.30\text{ in.})^3}{12} + \frac{(0.349\text{ in.})[6.00\text{ in.}-4(0.349\text{ in.})]^3}{6} + \frac{(9\pi^2-64)(0.349\text{ in.})^4}{36\pi} \\
&\quad + \pi(0.349\text{ in.})^2\left[\frac{6.00\text{ in.}-4(0.349\text{ in.})}{2}+\frac{4(0.349\text{ in.})}{3\pi}\right]^2 \\
&= 115\text{ in.}^4
\end{aligned}
$$

Limitations of AISC Specification Sections I1.3 and I2.2a

(1) Concrete Strength: $3\text{ ksi} \le f_c' \le 10\text{ ksi}$

$f_c' = 5\text{ ksi}$ **o.k.**

(2) Specified minimum yield stress of structural steel: $F_y \le 75\text{ ksi}$

$F_y = 50\text{ ksi}$ **o.k.**

(3) Cross-sectional area of steel section: $A_s \ge 0.01A_g$

$$
\begin{aligned}
10.4\text{ in.}^2 &\ge (0.01)(59.6\text{ in.}^2) \\
&> 0.596\text{ in.}^2 \quad \textbf{o.k.}
\end{aligned}
$$

There are no minimum longitudinal reinforcement requirements in the AISC *Specification* within filled composite members; therefore, the area of reinforcing bars, A_{sr}, for this example is zero.

Classify Section for Local Buckling

In order to determine the strength of the composite section subject to axial compression, the member is first classified as compact, noncompact or slender in accordance with AISC *Specification* Table I1.1a.

$$
\begin{aligned}
\lambda_p &= 2.26\sqrt{\frac{E}{F_y}} \\
&= 2.26\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} \\
&= 54.4
\end{aligned}
$$

$$\lambda_{controlling} = \max \begin{Bmatrix} h/t = 25.7 \\ b/t = 14.2 \end{Bmatrix}$$
$$= 25.7$$

$\lambda_{controlling} \le \lambda_p$; therefore, the section is compact

Available Compressive Strength

The nominal axial compressive strength for compact sections without consideration of length effects, P_{no}, is determined from AISC *Specification* Section I2.2b as:

$$P_{no} = P_p \quad \text{(Spec. Eq. I2-9a)}$$

$$P_p = F_y A_s + C_2 f_c' \left(A_c + A_{sr} \frac{E_s}{E_c} \right) \quad \text{(Spec. Eq. I2-9b)}$$

where

$C_2 = 0.85$ for rectangular sections

$$P_{no} = (50 \text{ ksi})(10.4 \text{ in.}^2) + 0.85(5 \text{ ksi})(49.2 \text{ in.}^2 + 0.0 \text{ in.}^2)$$
$$= 729 \text{ kips}$$

Because the unbraced length is the same in both the *x-x* and *y-y* directions, the column will buckle about the weaker *y-y* axis (the axis having the lower moment of inertia). I_{cy} and I_{sy} will therefore be used for calculation of length effects in accordance with AISC *Specification* Sections I2.2b and I2.1b as follows:

$$C_3 = 0.45 + 3\left(\frac{A_s + A_{sr}}{A_g} \right) \le 0.9 \quad \text{(Spec. Eq. I2-13)}$$
$$= 0.45 + 3\left(\frac{10.4 \text{ in.}^2 + 0.0 \text{ in.}^2}{59.6 \text{ in.}^2} \right) \le 0.9$$
$$= 0.973 > 0.9$$
$$= 0.9$$

$$E_c = w_c^{1.5} \sqrt{f_c'}$$
$$= (145 \text{ lb/ft}^3)^{1.5} \sqrt{5 \text{ ksi}}$$
$$= 3{,}900 \text{ ksi}$$

$$EI_{eff} = E_s I_{sy} + E_s I_{sr} + C_3 E_c I_{cy} \quad \text{(from Spec. Eq. I2-12)}$$
$$= (29{,}000 \text{ ksi})(61.8 \text{ in.}^4) + 0 \text{ kip-in.}^2 + 0.9(3{,}900 \text{ ksi})(115 \text{ in.}^4)$$
$$= 2{,}200{,}000 \text{ kip-in.}^2$$

$$P_e = \pi^2 (EI_{eff}) / (L_c)^2 \quad \text{(Spec. Eq. I2-5)}$$

where $L_c = KL$ and $K = 1.0$ for a pin-ended member

$$P_e = \frac{\pi^2\left(2{,}200{,}000 \text{ kip-in.}^2\right)}{\left[(1.0)(14 \text{ ft})(12 \text{ in./ft})\right]^2}$$
$$= 769 \text{ kips}$$

$$\frac{P_{no}}{P_e} = \frac{729 \text{ kips}}{769 \text{ kips}}$$
$$= 0.948 < 2.25$$

Therefore, use AISC *Specification* Equation I2-2.

$$P_n = P_{no}\left(0.658^{\frac{P_{no}}{P_e}}\right) \qquad (\textit{Spec}. \text{ Eq. I2-2})$$
$$= (729 \text{ kips})(0.658)^{0.948}$$
$$= 490 \text{ kips}$$

Check adequacy of the composite column for the required axial compressive strength:

LRFD	ASD
$\phi_c = 0.75$ $\phi_c P_n = 0.75(490 \text{ kips})$ $= 368 \text{ kips} > 173 \text{ kips}$ **o.k.**	$\Omega_c = 2.00$ $\dfrac{P_n}{\Omega_c} = \dfrac{490 \text{ kips}}{2.00}$ $= 245 \text{ kips} > 116 \text{ kips}$ **o.k.**

The values match those tabulated in Table IV-1B.

Available Compressive Strength of Bare Steel Section

Due to the differences in resistance and safety factors between composite and noncomposite column provisions, it is possible to calculate a lower available compressive strength for a composite column than one would calculate for the corresponding bare steel section. However, in accordance with AISC *Specification* Section I2.2b, the available compressive strength need not be less than that calculated for the bare steel member in accordance with Chapter E.

From AISC *Manual* Table 4-3, for an HSS10×6×⅜, KL_y = 14 ft:

LRFD	ASD
$\phi_c P_n = 331 \text{ kips} < 368 \text{ kips}$	$\dfrac{P_n}{\Omega_c} = 220 \text{ kips} < 245 \text{ kips}$

Thus, the composite section strength controls and is adequate for the required axial compressive strength as previously demonstrated.

Force Allocation and Load Transfer

Load transfer calculations for external axial forces should be performed in accordance with AISC *Specification* Section I6. The specific application of the load transfer provisions is dependent upon the configuration and detailing of the connecting elements. Expanded treatment of the application of load transfer provisions is provided in Design Example I.3.

EXAMPLE I.5 FILLED COMPOSITE MEMBER IN AXIAL TENSION

Given:

Determine if the filled composite member illustrated in Figure I.5-1 is adequate for the indicated dead load compression and wind load tension. The entire load is applied to the steel section.

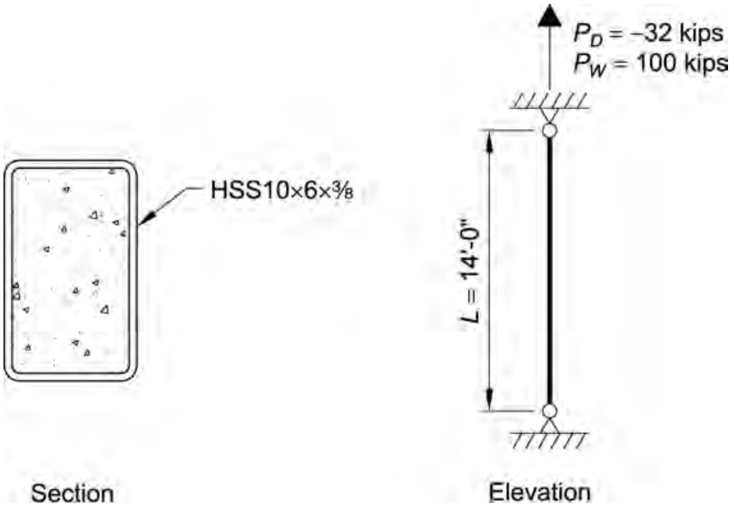


Fig. I.5-1. Filled composite member section and applied loading.

The composite member consists of an ASTM A500, Grade C, HSS with normal weight (145 lb/ft^3) concrete fill having a specified concrete compressive strength, $f_c' = 5$ ksi.

Solution:

From AISC *Manual* Table 2-4, the material properties are:

ASTM A500 Grade C
$F_y = 50$ ksi
$F_u = 62$ ksi

From AISC *Manual* Table 1-11, the geometric properties are as follows:

HSS10×6×⅜
$A_s = 10.4$ in.^{2}

There are no minimum requirements for longitudinal reinforcement in the AISC *Specification*; therefore, it is common industry practice to use filled shapes without longitudinal reinforcement, thus $A_{sr} = 0$.

From ASCE/SEI 7, Chapter 2, the required compressive strength is (taking compression as negative and tension as positive):

LRFD	ASD
Governing Uplift Load Combination $= 0.9D + 1.0W$ $P_r = P_u$ $= 0.9(-32 \text{ kips}) + 1.0(100 \text{ kips})$ $= 71.2$ kips	Governing Uplift Load Combination $= 0.6D + 0.6W$ $P_r = P_a$ $= 0.6(-32 \text{ kips}) + 0.6(100 \text{ kips})$ $= 40.8$ kips

Available Tensile Strength

Available tensile strength for a filled composite member is determined in accordance with AISC *Specification* Section I2.2c.

$$\begin{aligned} P_n &= A_s F_y + A_{sr} F_{ysr} \\ &= \left(10.4 \text{ in.}^2\right)\left(50 \text{ ksi}\right) + \left(0 \text{ in.}^2\right)\left(60 \text{ ksi}\right) \\ &= 520 \text{ kips} \end{aligned}$$ (*Spec*. Eq. I2-14)

LRFD	ASD
$\phi_t = 0.90$	$\Omega_t = 1.67$
$\phi_t P_n = 0.90\left(520 \text{ kips}\right)$ $= 468 \text{ kips} > 71.2 \text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_t} = \dfrac{520 \text{ kips}}{1.67}$ $= 311 \text{ kips} > 40.8 \text{ kips}$ **o.k.**

For filled composite HSS members with no internal longitudinal reinforcing, the values for available tensile strength may also be taken directly from AISC *Manual* Table 5-4. The values calculated here match those for the limit state of yielding shown in Table 5-4.

Force Allocation and Load Transfer

Load transfer calculations are not required for filled composite members in axial tension that do not contain longitudinal reinforcement, such as the one under investigation, as only the steel section resists tension.

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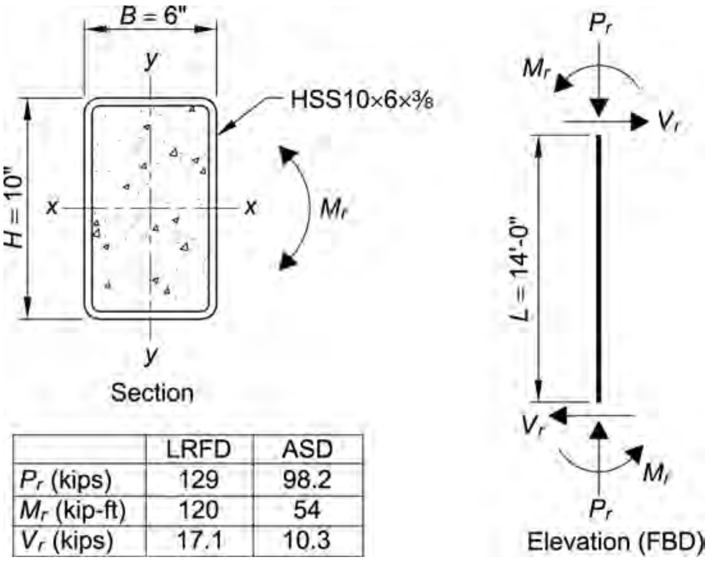
EXAMPLE I.6 FILLED COMPOSITE MEMBER IN COMBINED AXIAL COMPRESSION, FLEXURE AND SHEAR

Given:

Using AISC design tables, determine if the filled composite member illustrated in Figure I.6-1 is adequate for the indicated axial forces, shears and moments that have been determined in accordance with the direct analysis method of AISC *Specification* Chapter C for the controlling ASCE/SEI 7 load combinations.



	LRFD	ASD
P_r (kips)	129	98.2
M_r (kip-ft)	120	54
V_r (kips)	17.1	10.3

Fig. I.6-1. Filled composite member section and member forces.

The composite member consists of an ASTM A500, Grade C, HSS with normal weight (145 lb/ft^3) concrete fill having a specified concrete compressive strength, $f_c' = 5$ ksi.

Solution:

From AISC *Manual* Table 2-4, the material properties are:

ASTM A500 Grade C
$F_y = 50$ ksi
$F_u = 62$ ksi

From AISC *Manual* Table 1-11 and Figure I.6-1, the geometric properties are as follows:

HSS10×6×⅜
H = 10.0 in.
B = 6.00 in.
t_{nom} = ⅜ in. (nominal wall thickness)
t = 0.349 in. (design wall thickness)
h/t = 25.7
b/t = 14.2
A_s = 10.4 in.^{2}
I_{sx} = 137 in.^{4}
I_{sy} = 61.8 in.^{4}
Z_{sx} = 33.8 in.^{3}

Additional geometric properties used for composite design are determined in Design Examples I.3 and I.4 as follows:

h_i = 9.30 in. clear distance between HSS walls (longer side)
b_i = 5.30 in. clear distance between HSS walls (shorter side)
A_c = 49.2 in.^{2} cross-sectional area of concrete fill
A_g = 59.6 in.^{2} gross cross-sectional area of composite member
A_{sr} = 0 in.^{2} area of longitudinal reinforcement
E_c = 3,900 ksi modulus of elasticity of concrete
I_{cx} = 353 in.^{4} moment of inertia of concrete fill about the x-x axis
I_{cy} = 115 in.^{4} moment of inertia of concrete fill about the y-y axis

Limitations of AISC Specification Sections I1.3 and I2.2a

(1) Concrete Strength: $3 \text{ ksi} \le f_c' \le 10 \text{ ksi}$

$$f_c' = 5 \text{ ksi} \quad \textbf{o.k.}$$

(2) Specified minimum yield stress of structural steel: $F_y \le 75 \text{ ksi}$

$$F_y = 50 \text{ ksi} \quad \textbf{o.k.}$$

(3) Cross-sectional area of steel section: $A_s \ge 0.01A_g$

$$10.4 \text{ in.}^2 \ge (0.01)\left(59.6 \text{ in.}^2\right)$$

$$> 0.596 \text{ in.}^2 \quad \textbf{o.k.}$$

Classify Section for Local Buckling

The composite member in question was shown to be compact for pure compression in Example I.4 in accordance with AISC *Specification* Table I1.1a. The section must also be classified for local buckling due to flexure in accordance with *Specification* Table I1.1b; however, since the limits for members subject to flexure are equal to or less stringent than those for members subject to compression, the member is compact for flexure.

Interaction of Axial Force and Flexure

The interaction between axial forces and flexure in composite members is governed by AISC *Specification* Section I5 which, for compact members, permits the use of the methods of Section I1.2 with the option to use the interaction equations of Section H1.1.

The strain compatibility method is a generalized approach that allows for the construction of an interaction diagram based upon the same concepts used for reinforced concrete design. Application of the strain compatibility method is required for irregular/nonsymmetrical sections, and its general application may be found in reinforced concrete design texts and will not be discussed further here.

Plastic stress distribution methods are discussed in AISC *Specification* Commentary Section I5 which provides three acceptable procedures for compact filled members. The first procedure, Method 1, invokes the interaction equations of Section H1. The second procedure, Method 2, involves the construction of a piecewise-linear interaction curve using the plastic strength equations provided in AISC *Manual* Table 6-4. The third procedure, Method 2—Simplified, is a reduction of the piecewise-linear interaction curve that allows for the use of less conservative interaction equations than those presented in Chapter H (refer to AISC *Specification* Commentary Figure C-I5.3).

For this design example, each of the three applicable plastic stress distribution procedures are reviewed and compared.

Method 1: Interaction Equations of Section H1

The most direct and conservative method of assessing interaction effects is through the use of the interaction equations of AISC *Specification* Section H1. For HSS shapes, both the available compressive and flexural strengths can be determined from Table IV-1B (included in Part IV of this document). In accordance with the direct analysis method, a K factor of 1 is used. Because the unbraced length is the same in both the x-x and y-y directions, and I_x exceeds I_y, y-y axis buckling will govern for the compressive strength. Flexural strength is determined for the x-x axis to resist the applied moment about this axis indicated in Figure I.6-1.

Entering Table IV-1B with $L_{cy} = 14$ ft yields:

LRFD	ASD
$\phi_c P_n = 368 \text{ kips}$ $\phi_b M_{nx} = 141 \text{ kip-ft}$	$P_n / \Omega_c = 245 \text{ kips}$ $M_{nx} / \Omega_b = 93.5 \text{ kip-ft}$
$\frac{P_r}{P_c} = \frac{P_u}{\phi_c P_n}$ $= \frac{129 \text{ kips}}{368 \text{ kips}}$ $= 0.351 > 0.2$	$\frac{P_r}{P_c} = \frac{P_a}{P_n / \Omega_c}$ $= \frac{98.2 \text{ kips}}{245 \text{ kips}}$ $= 0.401 > 0.2$
Therefore, use AISC *Specification* Equation H1-1a.	Therefore, use AISC *Specification* Equation H1-1a.
$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_u}{\phi_b M_n}\right) \le 1.0$ (from *Spec.* Eq. H1-1a) $\frac{129 \text{ kips}}{368 \text{ kips}} + \frac{8}{9}\left(\frac{120 \text{ kip-ft}}{141 \text{ kip-ft}}\right) \le 1.0$ $1.11 > 1.0$ **n.g.**	$\frac{P_a}{P_n / \Omega_c} + \frac{8}{9}\left(\frac{M_a}{M_n / \Omega_b}\right) \le 1.0$ (from *Spec.* Eq. H1-1a) $\frac{98.2 \text{ kips}}{245 \text{ kips}} + \frac{8}{9}\left(\frac{54 \text{ kip-ft}}{93.5 \text{ kip-ft}}\right) \le 1.0$ $0.914 < 1.0$ **o.k.**

Using LRFD methodology, Method 1 indicates that the section is inadequate for the applied loads. The designer can elect to choose a new section that passes the interaction check or re-analyze the current section using a less conservative design method such as Method 2. The use of Method 2 is illustrated in the following section. Using ASD methodology, Method 1 indicates that the section is adequate for the applied loads.

Method 2: Interaction Curves from the Plastic Stress Distribution Model

The procedure for creating an interaction curve using the plastic stress distribution model is illustrated graphically in Figure I.6-2.

Referencing Figure I.6-2, the nominal strength interaction surface A, B, C, D, E is first determined using the equations provided in AISC *Manual* Table 6-4. This curve is representative of the short column member strength without consideration of length effects. A slenderness reduction factor, λ, is then calculated and applied to each point to create surface A′, B′, C′, D′, E′. The appropriate resistance or safety factors are then applied to create the design surface A″, B″, C″, D″, E″. Finally, the required axial and flexural strengths from the applicable load combinations of ASCE/SEI 7 are plotted on the design surface, and the member is acceptable for the applied loading if all points fall within the design surface. These steps are illustrated in detail by the following calculations.

Step 1: Construct nominal strength interaction surface A, B, C, D, E without length effects

Using the equations provided in AISC *Manual* Table 6-4 for bending about the x-x axis yields:

Point A (pure axial compression):

$$
\begin{aligned}
P_A &= F_y A_s + 0.85 f_c' A_c \\
&= (50\text{ ksi})\left(10.4\text{ in.}^2\right) + 0.85(5\text{ ksi})\left(49.2\text{ in.}^2\right) \\
&= 729\text{ kips}
\end{aligned}
$$

$$M_A = 0\text{ kip-ft}$$

Point D (maximum nominal moment strength):

$$
\begin{aligned}
P_D &= \frac{0.85 f_c' A_c}{2} \\
&= \frac{0.85(5\text{ ksi})\left(49.2\text{ in.}^2\right)}{2} \\
&= 105\text{ kips}
\end{aligned}
$$

$$Z_{sx} = 33.8\text{ in.}^3$$

$$
\begin{aligned}
r_i &= t \\
&= 0.349\text{ in.}
\end{aligned}
$$

$$
\begin{aligned}
Z_c &= \frac{b_i h_i^2}{4} - 0.429 r_i^2 h_i + 0.192 r_i^3 \\
&= \frac{(5.30\text{ in.})(9.30\text{ in.})^2}{4} - 0.429(0.349\text{ in.})^2(9.30\text{ in.}) + 0.192(0.349\text{ in.})^3 \\
&= 114\text{ in.}^3
\end{aligned}
$$

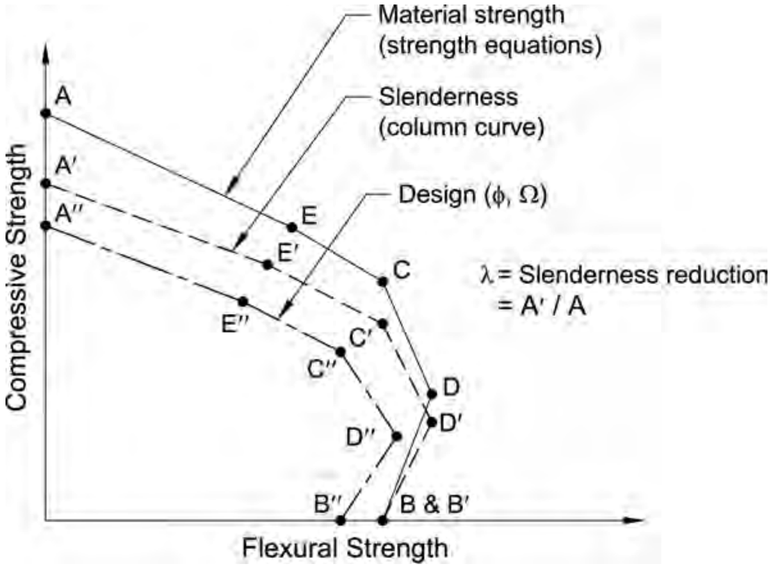


Fig. I.6-2. Interaction diagram for composite beam-column—Method 2.

$$
\begin{aligned}
M_D &= F_y Z_{sx} + \frac{0.85 f_c' Z_c}{2} \\
&= \left[(50 \text{ ksi})(33.8 \text{ in.}^3) + \frac{0.85(5 \text{ ksi})(114 \text{ in.}^3)}{2} \right] \left(\frac{1}{12 \text{ in./ft}} \right) \\
&= 161 \text{ kip-ft}
\end{aligned}
$$

Point B (pure flexure):

$$P_B = 0 \text{ kips}$$

$$
\begin{aligned}
h_n &= \frac{0.85 f_c' A_c}{2(0.85 f_c' b_i + 4 F_y t)} \le \frac{h_i}{2} \\
&= \frac{0.85(5 \text{ ksi})(49.2 \text{ in.}^2)}{2[0.85(5 \text{ ksi})(5.30 \text{ in.}) + 4(50 \text{ ksi})(0.349 \text{ in.})]} \le \frac{9.30 \text{ in.}}{2} \\
&= 1.13 \text{ in.} < 4.65 \text{ in.} \\
&= 1.13 \text{ in.}
\end{aligned}
$$

$$
\begin{aligned}
Z_{sn} &= 2 t h_n^2 \\
&= 2(0.349 \text{ in.})(1.13 \text{ in.})^2 \\
&= 0.891 \text{ in.}^3
\end{aligned}
$$

$$
\begin{aligned}
Z_{cn} &= b_i h_n^2 \\
&= (5.30 \text{ in.})(1.13 \text{ in.})^2 \\
&= 6.77 \text{ in.}^3
\end{aligned}
$$

$$
\begin{aligned}
M_B &= M_D - F_y Z_{sn} - 0.85 f_c' \left(\frac{Z_{cn}}{2} \right) \\
&= 161 \text{ kip-ft} - (50 \text{ ksi})(0.891 \text{ in.}^3)\left(\frac{1}{12 \text{ in./ft}} \right) - 0.85(5 \text{ ksi})\left(\frac{6.77 \text{ in.}^3}{2} \right)\left(\frac{1}{12 \text{ in./ft}} \right) \\
&= 156 \text{ kip-ft}
\end{aligned}
$$

Point C (intermediate point):

$$
\begin{aligned}
P_C &= 0.85 f_c' A_c \\
&= 0.85(5 \text{ ksi})(49.2 \text{ in.}^2) \\
&= 209 \text{ kips}
\end{aligned}
$$

$$
\begin{aligned}
M_C &= M_B \\
&= 156 \text{ kip-ft}
\end{aligned}
$$

Point E (optional):

Point E is an optional point that helps better define the interaction curve.

$$\begin{aligned} h_E &= \frac{h_n}{2} + \frac{H}{4} \quad \text{where } h_n = 1.13 \text{ in. from Point B} \\ &= \frac{1.13 \text{ in.}}{2} + \frac{10.0 \text{ in.}}{4} \\ &= 3.07 \text{ in.} \end{aligned}$$

$$\begin{aligned} P_E &= \frac{0.85 f_c' A_c}{2} + 0.85 f_c' b_i h_E + 4 F_y t h_E \\ &= \frac{0.85(5 \text{ ksi})(49.2 \text{ in.}^2)}{2} + 0.85(5 \text{ ksi})(5.30 \text{ in.})(3.07 \text{ in.}) + 4(50 \text{ ksi})(0.349 \text{ in.})(3.07 \text{ in.}) \\ &= 388 \text{ kips} \end{aligned}$$

$$\begin{aligned} Z_{cE} &= b_i h_E^2 \\ &= (5.30 \text{ in.})(3.07 \text{ in.})^2 \\ &= 50.0 \text{ in.}^3 \end{aligned}$$

$$\begin{aligned} Z_{sE} &= 2 t h_E^2 \\ &= 2(0.349 \text{ in.})(3.07 \text{ in.})^2 \\ &= 6.58 \text{ in.}^3 \end{aligned}$$

$$\begin{aligned} M_E &= M_D - F_y Z_{sE} - \frac{0.85 f_c' Z_{cE}}{2} \\ &= 161 \text{ kip-ft} - (50 \text{ ksi})(6.58 \text{ in.}^3)\left(\frac{1}{12 \text{ in./ft}}\right) - \left[\frac{0.85(5 \text{ ksi})(50.0 \text{ in.}^3)}{2}\right]\left(\frac{1}{12 \text{ in./ft}}\right) \\ &= 125 \text{ kip-ft} \end{aligned}$$

The calculated points are plotted to construct the nominal strength interaction surface without length effects as depicted in Figure I.6-3.

Step 2: Construct nominal strength interaction surface A′, B′, C′, D′, E′ with length effects

The slenderness reduction factor, λ, is calculated for Point A using AISC *Specification* Section I2.2 in accordance with *Specification* Commentary Section I5.

$$\begin{aligned} P_{no} &= P_A \\ &= 729 \text{ kips} \end{aligned}$$

$$C_3 = 0.45 + 3\left(\frac{A_s + A_{sr}}{A_g}\right) \le 0.9 \qquad \text{(\textit{Spec}. Eq. I2-13)}$$

$$\begin{aligned} &= 0.45 + 3\left(\frac{10.4 \text{ in.}^2 + 0 \text{ in.}^2}{59.6 \text{ in.}^2}\right) \le 0.9 \\ &= 0.973 > 0.9 \\ &= 0.9 \end{aligned}$$

$$EI_{eff} = E_s I_{sy} + E_s I_{sr} + C_3 E_c I_{cy} \quad \text{(from \textit{Spec}. Eq. I2-12)}$$
$$= (29{,}000 \text{ ksi})(61.8 \text{ in.}^4) + 0 + 0.9(3{,}900 \text{ ksi})(115 \text{ in.}^4)$$
$$= 2{,}200{,}000 \text{ ksi}$$

$P_e = \pi^2 (EI_{eff}) / (L_c)^2$, where $L_c = KL$ and $K = 1.0$ in accordance with the direct analysis method (*Spec*. Eq. I2-5)

$$= \frac{\pi^2 (2{,}200{,}000 \text{ ksi})}{[(14 \text{ ft})(12 \text{ in./ft})]^2}$$
$$= 769 \text{ kips}$$

$$\frac{P_{no}}{P_e} = \frac{729 \text{ kips}}{769 \text{ kips}}$$
$$= 0.948 < 2.25$$

Use AISC *Specification* Equation I2-2.

$$P_n = P_{no}\left(0.658^{\frac{P_{no}}{P_e}}\right) \quad \text{(\textit{Spec}. Eq. I2-2)}$$
$$= (729 \text{ kips})(0.658)^{0.948}$$
$$= 490 \text{ kips}$$

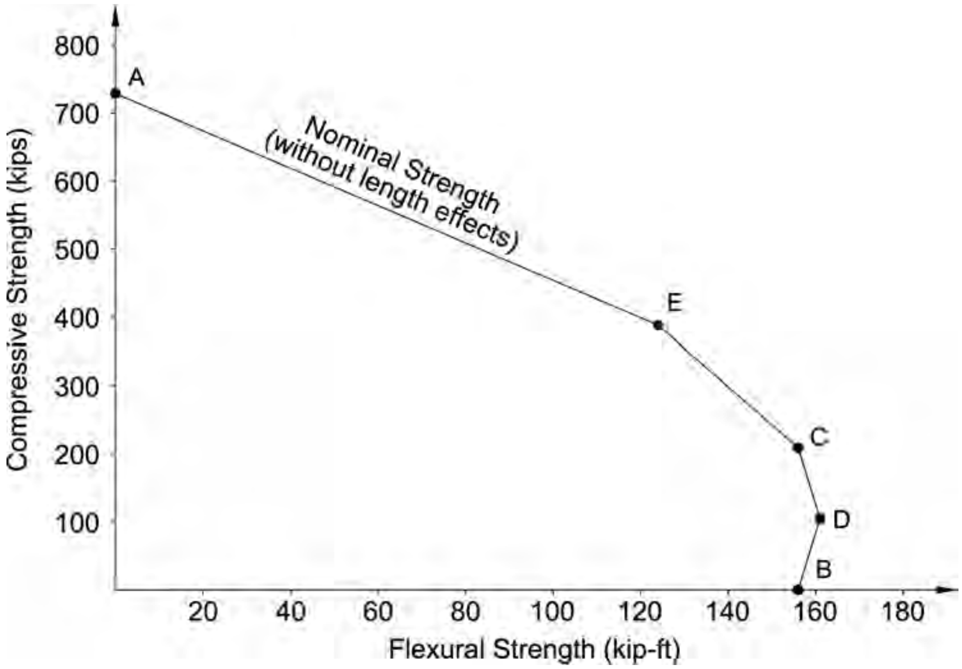


Fig. I.6-3. Nominal strength interaction surface without length effects.

From AISC *Specification* Commentary Section I5:

$$\begin{aligned}\lambda &= \frac{P_n}{P_{no}}\\ &= \frac{490\text{ kips}}{729\text{ kips}}\\ &= 0.672\end{aligned}$$

In accordance with AISC *Specification* Commentary Section I5, the same slenderness reduction is applied to each of the remaining points on the interaction surface as follows:

$$\begin{aligned}P_{A'} &= \lambda P_A\\ &= 0.672\left(729\text{ kips}\right)\\ &= 490\text{ kips}\end{aligned}$$

$$\begin{aligned}P_{B'} &= \lambda P_B\\ &= 0.672\left(0\text{ kip}\right)\\ &= 0\text{ kip}\end{aligned}$$

$$\begin{aligned}P_{C'} &= \lambda P_C\\ &= 0.672\left(209\text{ kips}\right)\\ &= 140\text{ kips}\end{aligned}$$

$$\begin{aligned}P_{D'} &= \lambda P_D\\ &= 0.672\left(105\text{ kips}\right)\\ &= 70.6\text{ kips}\end{aligned}$$

$$\begin{aligned}P_{E'} &= \lambda P_E\\ &= 0.672\left(388\text{ kips}\right)\\ &= 261\text{ kips}\end{aligned}$$

The modified axial strength values are plotted with the flexural strength values previously calculated to construct the nominal strength interaction surface including length effects. These values are superimposed on the nominal strength surface not including length effects for comparison purposes in Figure I.6-4.

Step 3: Construct design interaction surface A″, B″, C″, D″, E″ and verify member adequacy

The final step in the Method 2 procedure is to reduce the interaction surface for design using the appropriate resistance or safety factors.

LRFD	ASD
Design compressive strength: $\phi_c = 0.75$ $P_{X''} = \phi_c P_{X'}$, where X = A, B, C, D or E	Allowable compressive strength: $\Omega_c = 2.00$ $P_{X''} = P_{X'} / \Omega_c$, where X = A, B, C, D or E

LRFD	ASD
$P_{A''} = 0.75(490\text{ kips})$ $= 368\text{ kips}$	$P_{A''} = 490\text{ kips}/2.00$ $= 245\text{ kips}$
$P_{B''} = 0.75(0\text{ kip})$ $= 0\text{ kip}$	$P_{B''} = 0\text{ kip}/2.00$ $= 0\text{ kip}$
$P_{C''} = 0.75(140\text{ kips})$ $= 105\text{ kips}$	$P_{C''} = 140\text{ kips}/2.00$ $= 70.0\text{ kips}$
$P_{D''} = 0.75(70.6\text{ kips})$ $= 53.0\text{ kips}$	$P_{D''} = 70.6\text{ kips}/2.00$ $= 35.3\text{ kips}$
$P_{E''} = 0.75(261\text{ kips})$ $= 196\text{ kips}$	$P_{E''} = 261\text{ kips}/2.00$ $= 131\text{ kips}$

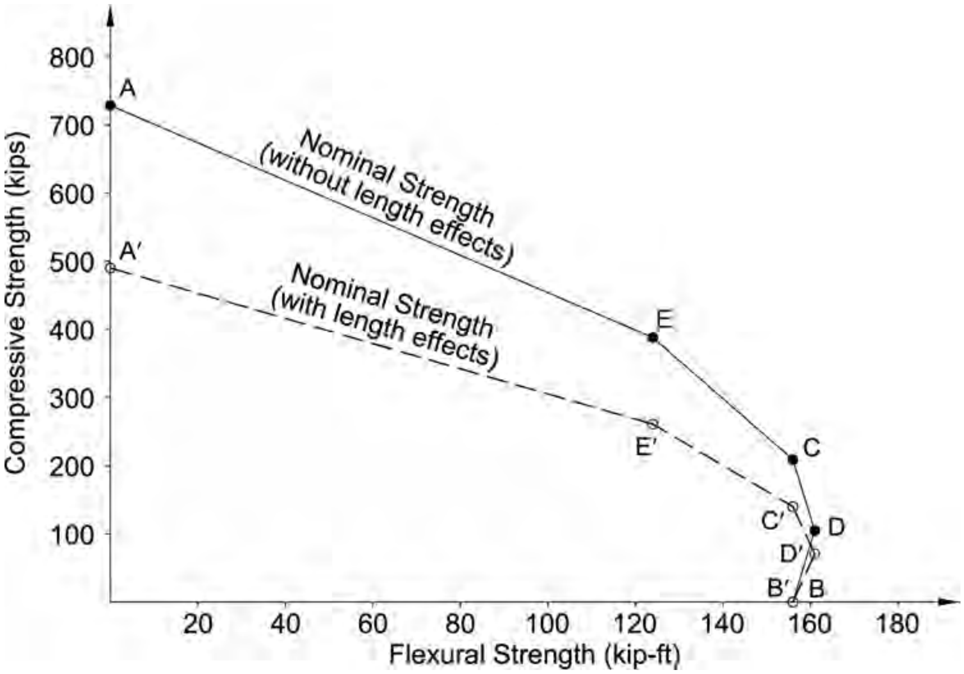


Fig. I.6-4. Nominal strength interaction surfaces (with and without length effects).

LRFD	ASD
Design flexural strength: $\phi_b = 0.90$	Allowable flexural strength: $\Omega_b = 1.67$
$M_{X''} = \phi_b M_X$, where X = A, B, C, D or E	$M_{X''} = M_X / \Omega_b$, where X = A, B, C, D or E
$M_{A''} = 0.90(0 \text{ kip-ft})$ $= 0 \text{ kip-ft}$	$M_{A''} = 0 \text{ kip-ft}/1.67$ $= 0 \text{ kip-ft}$
$M_{B''} = 0.90(156 \text{ kip-ft})$ $= 140 \text{ kip-ft}$	$M_{B''} = 156 \text{ kip-ft}/1.67$ $= 93.4 \text{ kip-ft}$
$M_{C''} = 0.90(156 \text{ kip-ft})$ $= 140 \text{ kip-ft}$	$M_{C''} = 156 \text{ kip-ft}/1.67$ $= 93.4 \text{ kip-ft}$
$M_{D''} = 0.90(161 \text{ kip-ft})$ $= 145 \text{ kip-ft}$	$M_{D''} = 161 \text{ kip-ft}/1.67$ $= 96.4 \text{ kip-ft}$
$M_{E''} = 0.90(124 \text{ kip-ft})$ $= 112 \text{ kip-ft}$	$M_{E''} = 124 \text{ kip-ft}/1.67$ $= 74.3 \text{ kip-ft}$

The available strength values for each design method can now be plotted. These values are superimposed on the nominal strength surfaces (with and without length effects) previously calculated for comparison purposes in Figure I.6-5.

By plotting the required axial and flexural strength values determined for the governing load combinations on the available strength surfaces indicated in Figure I.6-5, it can be seen that both ASD (M_a, P_a) and LRFD (M_u, P_u) points lie within their respective design surfaces. The member in question is therefore adequate for the applied loads.

Designers should carefully review the proximity of the available strength values in relation to point D'' on Figure I.6-5 as it is possible for point D'' to fall outside of the nominal strength curve, thus resulting in an unsafe design. This possibility is discussed further in AISC *Specification* Commentary Section I5 and is avoided through the use of Method 2—Simplified as illustrated in the following section.

Method 2: Simplified

The simplified version of Method 2 involves the removal of points D″ and E″ from the Method 2 interaction surface leaving only points A″, B″ and C″ as illustrated in the comparison of the two methods in Figure I.6-6.

Reducing the number of interaction points allows for a bilinear interaction check defined by AISC *Specification* Commentary Equations C-I5-1a and C-I5-1b to be performed. Using the available strength values previously calculated in conjunction with the Commentary equations, interaction ratios are determined as follows:

LRFD	ASD
$P_r = P_u$ $= 129\text{ kips}$	$P_r = P_a$ $= 98.2\text{ kips}$
$P_r \ge P_{C''}$ $\ge 105\text{ kips}$	$P_r \ge P_{C''}$ $\ge 70.0\text{ kips}$
Therefore, use AISC *Specification* Commentary Equation C-I5-1b.	Therefore, use AISC *Specification* Commentary Equation C-I5-1b.
$\dfrac{P_r - P_C}{P_A - P_C} + \dfrac{M_r}{M_C} \le 1.0$ (from *Spec*. Eq. C-I5-1b)	$\dfrac{P_r - P_C}{P_A - P_C} + \dfrac{M_r}{M_C} \le 1.0$ (from *Spec*. Eq. C-I5-1b)
which for LRFD equals:	which for ASD equals:
$\dfrac{P_u - P_{C''}}{P_{A''} - P_{C''}} + \dfrac{M_u}{M_{C''}} \le 1.0$ $\dfrac{129\text{ kips} - 105\text{ kips}}{368\text{ kips} - 105\text{ kips}} + \dfrac{120\text{ kip-ft}}{140\text{ kip-ft}} \le 1.0$ $0.948 < 1.0$ **o.k.**	$\dfrac{P_a - P_{C''}}{P_{A''} - P_{C''}} + \dfrac{M_a}{M_{C''}} \le 1.0$ $\dfrac{98.2\text{ kips} - 70.0\text{ kips}}{245\text{ kips} - 70.0\text{ kips}} + \dfrac{54\text{ kip-ft}}{93.4\text{ kip-ft}} \le 1.0$ $0.739 < 1.0$ **o.k.**

Thus, the member is adequate for the applied loads.

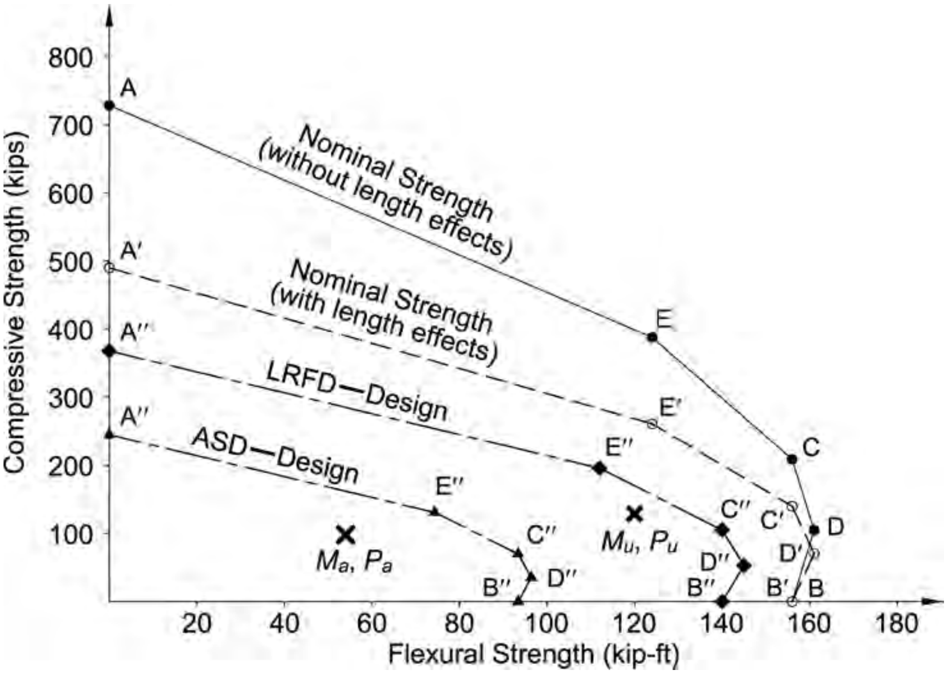


Fig. I.6-5. Available and nominal interaction surfaces.

Comparison of Methods

The composite member was found to be inadequate using Method 1—Chapter H interaction equations, but was found to be adequate using both Method 2 and Method 2—Simplified procedures. A comparison between the methods is most easily made by overlaying the design curves from each method as illustrated in Figure I.6-7 for LRFD design.

From Figure I.6-7, the conservative nature of the Chapter H interaction equations can be seen. Method 2 provides the highest available strength; however, the Method 2—Simplified procedure also provides a good representation of the complete design curve. By using the Part IV design tables to determine the available strength of the composite member in compression and flexure (Points A'' and B'' respectively), the modest additional effort required to calculate the available compressive strength at Point C'' can result in appreciable gains in member strength when using Method 2—Simplified as opposed to Method 1.

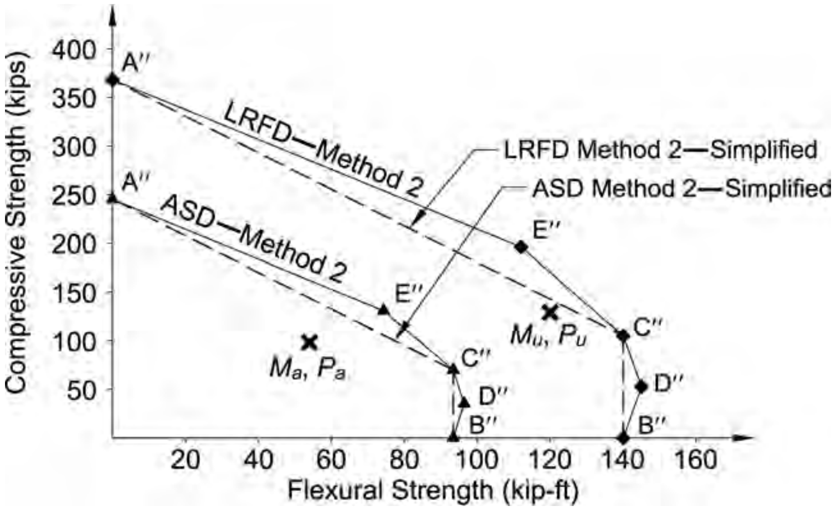


Fig. I.6-6. Comparison of Method 2 and Method 2—Simplified.

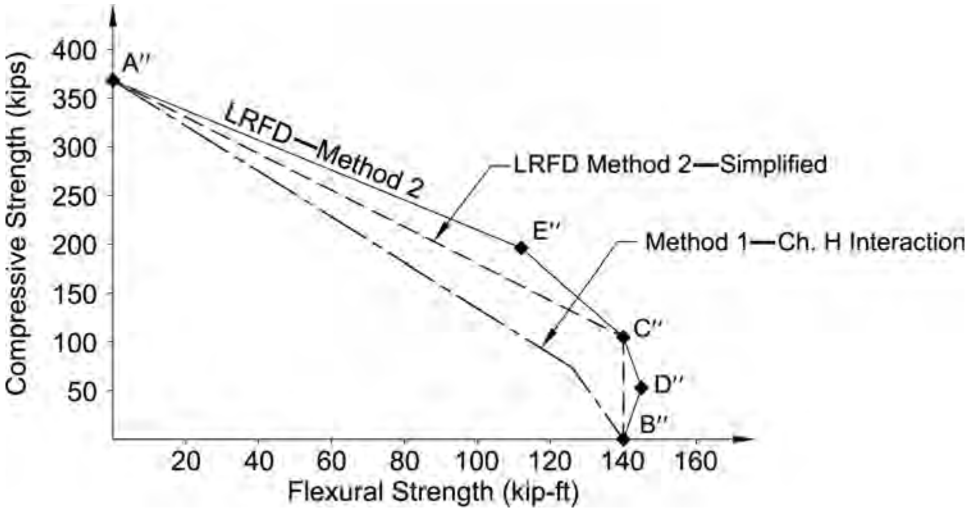


Fig. I.6-7. Comparison of interaction methods (LRFD).

Available Shear Strength

AISC *Specification* Section I4.1 provides three methods for determining the available shear strength of a filled composite member: available shear strength of the steel section alone in accordance with Chapter G; available shear strength of the reinforced concrete portion alone per ACI 318 (ACI 318, 2014); or available shear strength of the steel section plus the reinforcing steel ignoring the contribution of the concrete. The available shear strength will be determined using the first two methods because there is no reinforcing steel provided in this example.

Available Shear Strength of Steel Section

The nominal shear strength, V_n, of rectangular HSS members is determined using the provisions of AISC *Specification* Section G4. The web shear coefficient, C_{v2}, is determined from AISC *Specification* Section G2.2 with, $h/t_w = h/t$ and $k_v = 5$.

$$\begin{aligned}1.10\sqrt{k_v\,E/F_y} &= 1.10\sqrt{\frac{(5)(29{,}000\text{ ksi})}{50\text{ ksi}}}\\ &= 59.2 > h/t = 25.7\end{aligned}$$

Use AISC *Specification* Equation G2-9.

$$C_{v2} = 1.0 \qquad \text{(}Spec\text{. Eq. G2-9)}$$

The nominal shear strength is calculated as:

$$\begin{aligned}h &= H - 3t\\ &= 10.0\text{ in.} - 3(0.349\text{ in.})\\ &= 8.95\text{ in.}\end{aligned}$$

$$\begin{aligned}A_w &= 2ht\\ &= 2(8.95\text{ in.})(0.349\text{ in.})\\ &= 6.25\text{ in.}^2\end{aligned}$$

$$\begin{aligned}V_n &= 0.6F_yA_wC_{v2} \qquad \text{(}Spec\text{. Eq. G4-1)}\\ &= 0.6(50\text{ ksi})(6.25\text{ in.}^2)(1.0)\\ &= 188\text{ kips}\end{aligned}$$

The available shear strength of the steel section is:

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(188\text{ kips})$ $= 169\text{ kips} > 17.1\text{ kips}$ **o.k.**	$\Omega_v = 1.67$ $\dfrac{V_n}{\Omega_v} = \dfrac{188\text{ kips}}{1.67}$ $= 113\text{ kips} > 10.3\text{ kips}$ **o.k.**

Available Shear Strength of the Reinforced Concrete

The available shear strength of the steel section alone has been shown to be sufficient, but the available shear strength of the concrete will be calculated for demonstration purposes. Considering that the member does not have longitudinal reinforcing, the method of shear strength calculation involving reinforced concrete is not valid; however, the design shear strength of the plain concrete using ACI 318, Chapter 14, can be determined as follows:

ϕ = 0.60 for plain concrete design from ACI 318 Section 21.2.1
λ = 1.0 for normal weight concrete from ACI 318 Section 19.2.4.2

$$V_n = \left(\frac{4}{3}\right)\lambda\sqrt{f_c'}b_w h \qquad \text{(ACI 318 Section 14.5.5.1)}$$

$b_w = b_i$
$h = h_i$

$$V_n = \left(\frac{4}{3}\right)(1.0)\sqrt{5{,}000 \text{ psi}}\,(5.30 \text{ in.})(9.30 \text{ in.})\left(\frac{1 \text{ kip}}{1{,}000 \text{ lb}}\right)$$
$$= 4.65 \text{ kips}$$
$$\phi V_n = 0.60(4.65 \text{ kips}) \qquad \text{(ACI 318 Section 14.5.1.1)}$$
$$= 2.79 \text{ kips} < 17.1 \text{ kips} \quad \textbf{n.g.}$$

As can be seen from this calculation, the shear resistance provided by plain concrete is small and the strength of the steel section alone is generally sufficient.

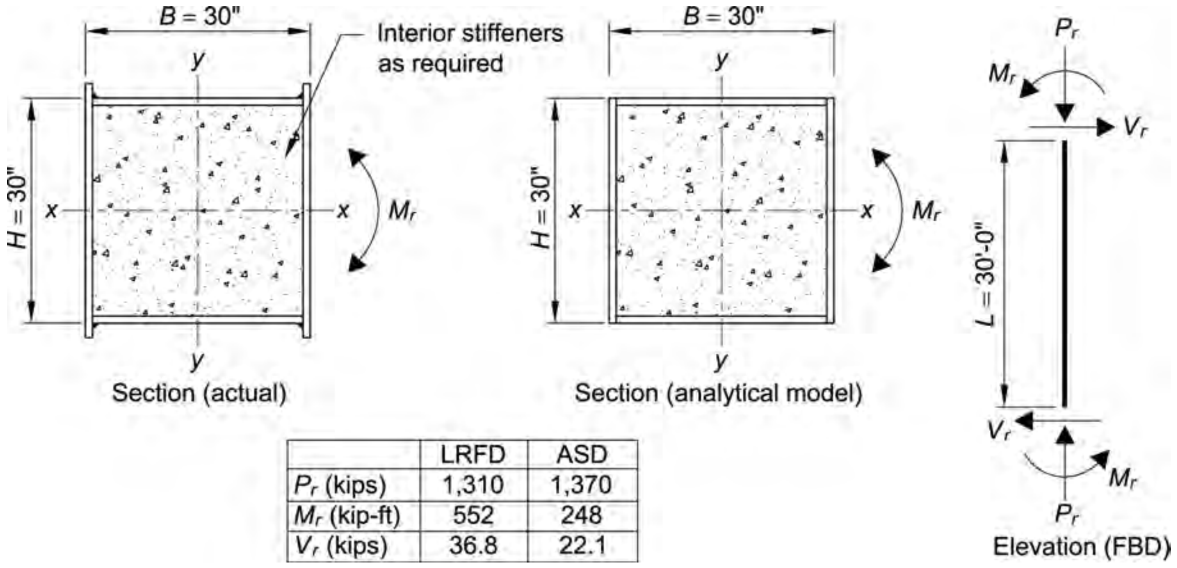
Force Allocation and Load Transfer

Load transfer calculations for applied axial forces should be performed in accordance with AISC *Specification* Section I6. The specific application of the load transfer provisions is dependent upon the configuration and detailing of the connecting elements. Expanded treatment of the application of load transfer provisions is provided in Design Example I.3.

EXAMPLE I.7 FILLED COMPOSITE BOX COLUMN WITH NONCOMPACT/SLENDER ELEMENTS

Given:

Determine the required ASTM A36 plate thickness of the filled composite box column illustrated in Figure I.7-1 to resist the indicated axial forces, shears and moments that have been determined in accordance with the direct analysis method of AISC *Specification* Chapter C for the controlling ASCE/SEI 7 load combinations. The core is composed of normal weight (145 lb/ft^3) concrete fill having a specified concrete compressive strength, $f_c' = 7$ ksi.



	LRFD	ASD
P_r (kips)	1,310	1,370
M_r (kip-ft)	552	248
V_r (kips)	36.8	22.1

Fig. I.7-1. Composite box column section and member forces.

Solution:

From AISC *Manual* Table 2-5, the material properties are:

ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

Trial Size 1 (Noncompact)

For ease of calculation the contribution of the plate extensions to the member strength will be ignored as illustrated by the analytical model in Figure I.7-1.

Trial Plate Thickness and Geometric Section Properties of the Composite Member

Select a trial plate thickness, *t*, of ⅜ in. Note that the design wall thickness reduction of AISC *Specification* Section B4.2 applies only to electric-resistance-welded HSS members and does not apply to built-up sections such as the one under consideration.

The calculated geometric properties of the 30 in. by 30 in. steel box column are:

$$B = 30 \text{ in.}$$
$$H = 30 \text{ in.}$$
$$A_g = 900 \text{ in.}^2$$
$$A_c = 856 \text{ in.}^2$$
$$A_s = 44.4 \text{ in.}^2$$

$$\begin{aligned} b_i &= B - 2t \\ &= 30 \text{ in.} - 2\left(\tfrac{3}{8} \text{ in.}\right) \\ &= 29.2 \text{ in.} \end{aligned}$$

$$\begin{aligned} h_i &= H - 2t \\ &= 30 \text{ in.} - 2\left(\tfrac{3}{8} \text{ in.}\right) \\ &= 29.2 \text{ in.} \end{aligned}$$

$$\begin{aligned} E_c &= w_c^{1.5}\sqrt{f_c'} \\ &= \left(145 \text{ lb/ft}^3\right)^{1.5}\sqrt{7 \text{ ksi}} \\ &= 4{,}620 \text{ ksi} \end{aligned}$$

$$\begin{aligned} I_{gx} &= \frac{BH^3}{12} \\ &= \frac{(30 \text{ in.})(30 \text{ in.})^3}{12} \\ &= 67{,}500 \text{ in.}^4 \end{aligned}$$

$$\begin{aligned} I_{cx} &= \frac{b_i h_i^3}{12} \\ &= \frac{(29.2 \text{ in.})(29.2 \text{ in.})^3}{12} \\ &= 60{,}600 \text{ in.}^4 \end{aligned}$$

$$\begin{aligned} I_{sx} &= I_{gx} - I_{cx} \\ &= 67{,}500 \text{ in.}^4 - 60{,}600 \text{ in.}^4 \\ &= 6{,}900 \text{ in.}^4 \end{aligned}$$

Limitations of AISC Specification Sections I1.3 and I2.2a

(1) Concrete Strength: $3 \text{ ksi} \le f_c' \le 10 \text{ ksi}$

$f_c' = 7 \text{ ksi}$ **o.k.**

(2) Specified minimum yield stress of structural steel: $F_y \le 75 \text{ ksi}$

$F_y = 36 \text{ ksi}$ **o.k.**

(3) Cross-sectional area of steel section: $A_s \geq 0.01A_g$

$$44.4\text{ in.}^2 \geq (0.01)\left(900\text{ in.}^2\right)$$
$$> 9.00\text{ in.}^2 \quad \textbf{o.k.}$$

Classify Section for Local Buckling

Classification of the section for local buckling is performed in accordance with AISC *Specification* Table I1.1a for compression and Table I1.1b for flexure. As noted in *Specification* Section I1.4, the definitions of width, depth and thickness used in the evaluation of slenderness are provided in Section B4.1b.

For box columns, the widths of the stiffened compression elements used for slenderness checks, b and h, are equal to the clear distances between the column walls, b_i and h_i. The slenderness ratios are determined as follows:

$$\begin{aligned}\lambda &= \frac{b_i}{t} = \frac{h_i}{t}\\ &= \frac{29.2\text{ in.}}{3/8\text{ in.}}\\ &= 77.9\end{aligned}$$

Classify section for local buckling in steel elements subject to axial compression from AISC *Specification* Table I1.1a:

$$\begin{aligned}\lambda_p &= 2.26\sqrt{\frac{E}{F_y}}\\ &= 2.26\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}}\\ &= 64.1\end{aligned}$$

$$\begin{aligned}\lambda_r &= 3.00\sqrt{\frac{E}{F_y}}\\ &= 3.00\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}}\\ &= 85.1\end{aligned}$$

$\lambda_p \leq \lambda \leq \lambda_r$; therefore, the section is noncompact for compression

According to AISC *Specification* Section I1.4, if any side of the section in question is noncompact or slender, then the entire section is treated as noncompact or slender. For the square section under investigation; however, this distinction is unnecessary as all sides are equal in length.

Classification of the section for local buckling in elements subject to flexure is performed in accordance with AISC *Specification* Table I1.1b. Note that flanges and webs are treated separately; however, for the case of a square section only the most stringent limitations, those of the flange, need be applied. Noting that the flange limitations for bending are the same as those for compression,

$\lambda_p \leq \lambda \leq \lambda_r$; therefore, the section is noncompact for flexure

Available Compressive Strength

Compressive strength for noncompact filled composite members is determined in accordance with AISC *Specification* Section I2.2b(b).

$$P_p = F_y A_s + C_2 f_c'\left(A_c + A_{sr}\frac{E_s}{E_c}\right), \text{ where } C_2 = 0.85 \text{ for rectangular sections} \qquad (\textit{Spec}. \text{ Eq. I2-9b})$$

$$= (36\text{ ksi})(44.4\text{ in.}^2) + 0.85(7\text{ ksi})(856\text{ in.}^2 + 0\text{ in.}^2)$$

$$= 6{,}690\text{ kips}$$

$$P_y = F_y A_s + 0.7 f_c'\left(A_c + A_{sr}\frac{E_s}{E_c}\right) \qquad (\textit{Spec}. \text{ Eq. I2-9d})$$

$$= (36\text{ ksi})(44.4\text{ in.}^2) + 0.7(7\text{ ksi})(856\text{ in.}^2 + 0\text{ in.}^2)$$

$$= 5{,}790\text{ kips}$$

$$P_{no} = P_p - \frac{P_p - P_y}{(\lambda_r - \lambda_p)^2}(\lambda - \lambda_p)^2 \qquad (\textit{Spec}. \text{ Eq. I2-9c})$$

$$= 6{,}690\text{ kips} - \frac{6{,}690\text{ kips} - 5{,}790\text{ kips}}{(85.1 - 64.1)^2}(77.9 - 64.1)^2$$

$$= 6{,}300\text{ kips}$$

$$C_3 = 0.45 + 3\left(\frac{A_s + A_{sr}}{A_g}\right) \le 0.9 \qquad (\textit{Spec}. \text{ Eq. I2-13})$$

$$= 0.45 + 3\left(\frac{44.4\text{ in.}^2 + 0\text{ in.}^2}{900\text{ in.}^2}\right) \le 0.9$$

$$= 0.598 < 0.9$$

$$= 0.598$$

$$EI_{eff} = E_s I_s + E_s I_{sr} + C_3 E_c I_c \qquad (\textit{Spec}. \text{ Eq. I2-12})$$

$$= (29{,}000\text{ ksi})(6{,}900\text{ in.}^4) + 0.0\text{ kip-in.}^2 + 0.598(4{,}620\text{ ksi})(60{,}600\text{ in.}^4)$$

$$= 368{,}000{,}000\text{ kip-in.}^2$$

$$P_e = \pi^2(EI_{eff})/(L_c)^2, \text{ where } L_c = KL \text{ and } K = 1.0 \text{ in accordance with the direct analysis method} \qquad (\textit{Spec}. \text{ Eq. I2-5})$$

$$= \frac{\pi^2(368{,}000{,}000\text{ kip-in.}^2)}{[(30\text{ ft})(12\text{ in./ft})]^2}$$

$$= 28{,}000\text{ kips}$$

$$\frac{P_{no}}{P_e} = \frac{6{,}300\text{ kips}}{28{,}000\text{ kips}}$$

$$= 0.225 < 2.25$$

Therefore, use AISC *Specification* Equation I2-2.

$$P_n = P_{no}\left(0.658^{\frac{P_{no}}{P_e}}\right) \qquad \text{(\textit{Spec}. Eq. I2-2)}$$

$$= (6{,}300\text{ kips})(0.658)^{0.225}$$

$$= 5{,}730\text{ kips}$$

According to AISC *Specification* Section I2.2b, the compression strength need not be less than that specified for the bare steel member as determined by *Specification* Chapter E. It can be shown that the compression strength of the bare steel for this section is equal to 955 kips, thus the strength of the composite section controls.

The available compressive strength is:

LRFD	ASD
$\phi_c = 0.75$ $\phi_c P_n = 0.75(5{,}730\text{ kips})$ $= 4{,}300\text{ kips}$	$\Omega_c = 2.00$ $\dfrac{P_n}{\Omega_c} = \dfrac{5{,}730\text{ kips}}{2.00}$ $= 2{,}870\text{ kips}$

Available Flexural Strength

Flexural strength of noncompact filled composite members is determined in accordance with AISC *Specification* Section I3.4b(b):

$$M_n = M_p - (M_p - M_y)\frac{(\lambda - \lambda_p)}{(\lambda_r - \lambda_p)} \qquad \text{(\textit{Spec}. Eq. I3-3b)}$$

In order to utilize Equation I3-3b, both the plastic moment strength of the section, M_p, and the yield moment strength of the section, M_y, must be calculated.

Plastic Moment Strength

The first step in determining the available flexural strength of a noncompact section is to calculate the moment corresponding to the plastic stress distribution over the composite cross section, M_p. This concept is illustrated graphically in AISC *Specification* Commentary Figure C-I3.7(a) and follows the force distribution depicted in Figure I.7-2 and detailed in Table I.7-1.

Table I.7-1. Plastic Moment Equations

Component	Force	Moment Arm
Compression in steel flange	$C_1 = b_i t_f F_y$	$y_{C1} = a_p - \dfrac{t_f}{2}$
Compression in concrete	$C_2 = 0.85f'_c\left(a_p - t_f\right)b_i$	$y_{C2} = \dfrac{a_p - t_f}{2}$
Compression in steel web	$C_3 = a_p 2t_w F_y$	$y_{C3} = \dfrac{a_p}{2}$
Tension in steel web	$T_1 = \left(H - a_p\right)2t_w F_y$	$y_{T1} = \dfrac{H - a_p}{2}$
Tension in steel flange	$T_2 = b_i t_f F_y$	$y_{T2} = H - a_p - \dfrac{t_f}{2}$

where:

$$a_p = \frac{2F_y H t_w + 0.85f'_c b_i t_f}{4t_w F_y + 0.85f'_c b_i}$$

$$M_p = \sum(\text{force})(\text{moment arm})$$

Using the equations provided in Table I.7-1 for the section in question results in the following:

$$\begin{aligned} a_p &= \frac{2(36\text{ ksi})(30\text{ in.})(\tfrac{3}{8}\text{ in.}) + 0.85(7\text{ ksi})(29.2\text{ in.})(\tfrac{3}{8}\text{ in.})}{4(\tfrac{3}{8}\text{ in.})(36\text{ ksi}) + 0.85(7\text{ ksi})(29.2\text{ in.})} \\ &= 3.84\text{ in.} \end{aligned}$$

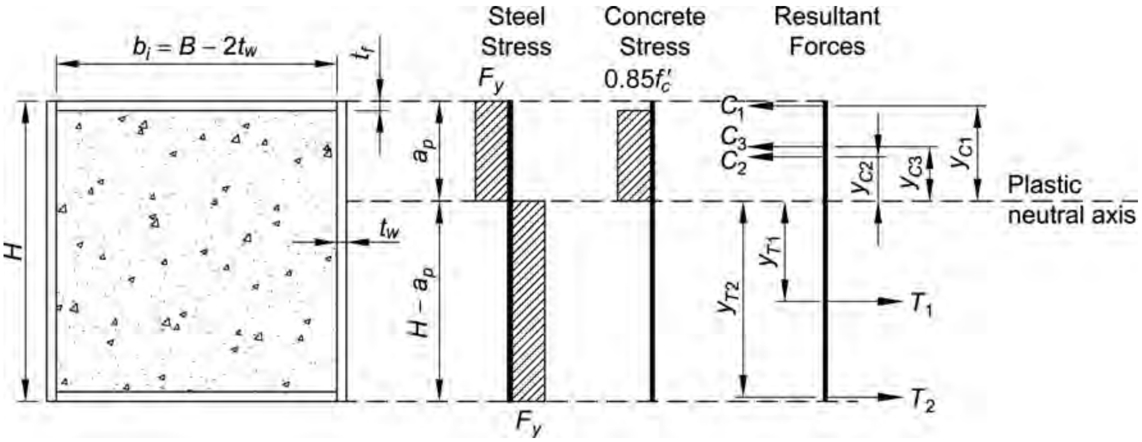


Figure I.7-2. Plastic moment stress blocks and force distribution.

Force	Moment Arm	Force × Moment Arm
$C_1 = (29.2 \text{ in.})(\tfrac{3}{8} \text{ in.})(36 \text{ ksi})$ $= 394 \text{ kips}$	$y_{C1} = 3.84 \text{ in.} - \dfrac{\tfrac{3}{8} \text{ in.}}{2}$ $= 3.65 \text{ in.}$	$C_1 y_{C1} = 1{,}440 \text{ kip-in.}$
$C_2 = 0.85(7 \text{ ksi})(3.84 \text{ in.} - \tfrac{3}{8} \text{ in.})(29.2 \text{ in.})$ $= 602 \text{ kips}$	$y_{C2} = \dfrac{3.84 \text{ in.} - \tfrac{3}{8} \text{ in.}}{2}$ $= 1.73 \text{ in.}$	$C_2 y_{C2} = 1{,}040 \text{ kip-in.}$
$C_3 = (3.84 \text{ in.})(2)(\tfrac{3}{8} \text{ in.})(36 \text{ ksi})$ $= 104 \text{ kips}$	$y_{C3} = \dfrac{3.84 \text{ in.}}{2}$ $= 1.92 \text{ in.}$	$C_3 y_{C3} = 200 \text{ kip-in.}$
$T_1 = (30 \text{ in.} - 3.84 \text{ in.})(2)(\tfrac{3}{8} \text{ in.})(36 \text{ ksi})$ $= 706 \text{ kips}$	$y_{T1} = \dfrac{30 \text{ in.} - 3.84 \text{ in.}}{2}$ $= 13.1 \text{ in.}$	$T_1 y_{T1} = 9{,}250 \text{ kip-in.}$
$T_2 = (29.2 \text{ in.})(\tfrac{3}{8} \text{ in.})(36 \text{ ksi})$ $= 394 \text{ kips}$	$y_{T2} = 30 \text{ in.} - 3.84 \text{ in.} - \dfrac{\tfrac{3}{8} \text{ in.}}{2}$ $= 26.0 \text{ in.}$	$T_2 y_{T2} = 10{,}200 \text{ kip-in.}$
$M_p = \sum (\text{force})(\text{moment arm})$ $= \dfrac{1{,}440 \text{ kip-in.} + 1{,}040 \text{ kip-in.} + 200 \text{ kip-in.} + 9{,}250 \text{ kip-in.} + 10{,}200 \text{ kip-in.}}{12 \text{ in./ft}}$ $= 1{,}840 \text{ kip-ft}$		

Yield Moment Strength

The next step in determining the available flexural strength of a noncompact filled member is to determine the yield moment strength. The yield moment is defined in AISC *Specification* Section I3.4b(b) as the moment corresponding to first yield of the compression flange calculated using a linear elastic stress distribution with a maximum concrete compressive stress of $0.7 f_c'$. This concept is illustrated diagrammatically in *Specification* Commentary Figure C-I3.7(b) and follows the force distribution depicted in Figure I.7-3 and detailed in Table I.7-2.

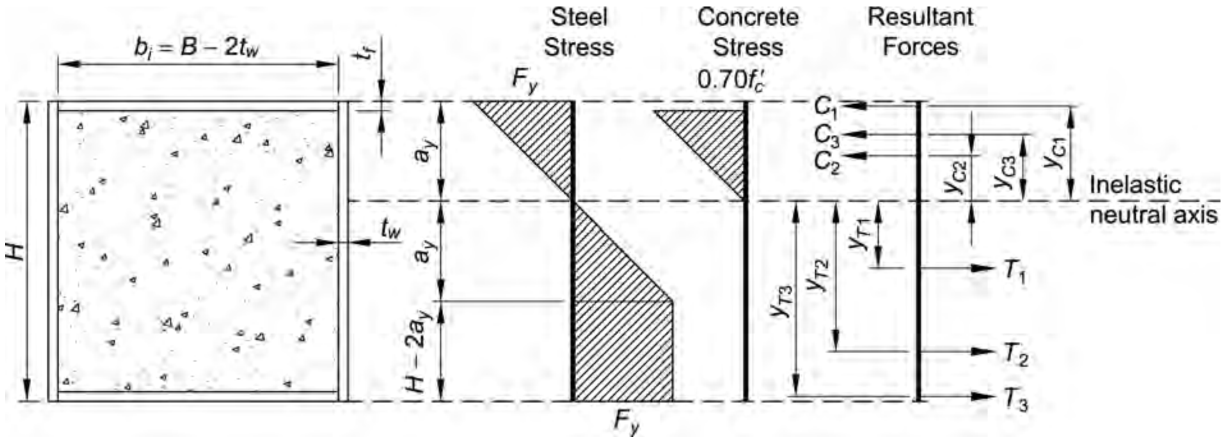


Figure I.7-3. Yield moment stress blocks and force distribution.

Table I.7-2. Yield Moment Equations

Component	Force	Moment Arm
Compression in steel flange	$C_1 = b_i t_f F_y$	$y_{C1} = a_y - \dfrac{t_f}{2}$
Compression in concrete	$C_2 = 0.35 f'_c \left(a_y - t_f\right) b_i$	$y_{C2} = \dfrac{2\left(a_y - t_f\right)}{3}$
Compression in steel web	$C_3 = a_y 2t_w 0.5F_y$	$y_{C3} = \dfrac{2a_y}{3}$
Tension in steel web	$T_1 = a_y 2t_w 0.5F_y$ $T_2 = \left(H - 2a_y\right) 2t_w F_y$	$y_{T1} = \dfrac{2a_y}{3}$ $y_{T2} = \dfrac{H}{2}$
Tension in steel flange	$T_3 = b_i t_f F_y$	$y_{T3} = H - a_y - \dfrac{t_f}{2}$

where

$$a_y = \frac{2F_y H t_w + 0.35 f'_c b_i t_f}{4t_w F_y + 0.35 f'_c b_i}$$

$$M_y = \sum \left(\text{force}\right)\left(\text{moment arm}\right)$$

Using the equations provided in Table I.7-2 for the section in question results in the following:

$$\begin{aligned} a_y &= \frac{2\left(36 \text{ ksi}\right)\left(30 \text{ in.}\right)\left(\tfrac{3}{8} \text{ in.}\right) + 0.35\left(7 \text{ ksi}\right)\left(29.2 \text{ in.}\right)\left(\tfrac{3}{8} \text{ in.}\right)}{4\left(\tfrac{3}{8} \text{ in.}\right)\left(36 \text{ ksi}\right) + 0.35\left(7 \text{ ksi}\right)\left(29.2 \text{ in.}\right)} \\ &= 6.66 \text{ in.} \end{aligned}$$

Force	Moment Arm	Force × Moment Arm
$C_1 = \left(29.2 \text{ in.}\right)\left(\tfrac{3}{8} \text{ in.}\right)\left(36 \text{ ksi}\right)$ $= 394 \text{ kips}$	$y_{C1} = 6.66 \text{ in.} - \dfrac{\tfrac{3}{8} \text{ in.}}{2}$ $= 6.47 \text{ in.}$	$C_1 y_{C1} = 2{,}550 \text{ kip-in.}$
$C_2 = 0.35\left(7 \text{ ksi}\right)\left(6.66 \text{ in.} - \tfrac{3}{8} \text{ in.}\right)\left(29.2 \text{ in.}\right)$ $= 450 \text{ kips}$	$y_{C2} = \dfrac{2\left(6.66 \text{ in.} - \tfrac{3}{8} \text{ in.}\right)}{3}$ $= 4.19 \text{ in.}$	$C_2 y_{C2} = 1{,}890 \text{ kip-in.}$
$C_3 = \left(6.66 \text{ in.}\right)\left(2\right)\left(\tfrac{3}{8} \text{ in.}\right)\left(0.5\right)\left(36 \text{ ksi}\right)$ $= 89.9 \text{ kips}$	$y_{C3} = \dfrac{2\left(6.66 \text{ in.}\right)}{3}$ $= 4.44 \text{ in.}$	$C_3 y_{C3} = 399 \text{ kip-in.}$
$T_1 = \left(6.66 \text{ in.}\right)\left(2\right)\left(\tfrac{3}{8} \text{ in.}\right)\left(0.5\right)\left(36 \text{ ksi}\right)$ $= 89.9 \text{ kips}$	$y_{T1} = \dfrac{2\left(6.66 \text{ in.}\right)}{3}$ $= 4.44 \text{ in.}$	$T_1 y_{T1} = 399 \text{ kip-in.}$
$T_2 = \left[30 \text{ in.} - 2\left(6.66 \text{ in.}\right)\right]\left(2\right)\left(\tfrac{3}{8} \text{ in.}\right)\left(36 \text{ ksi}\right)$ $= 450 \text{ kips}$	$y_{T2} = \dfrac{30 \text{ in.}}{2}$ $= 15.0 \text{ in.}$	$T_2 y_{T2} = 6{,}750 \text{ kip-in.}$
$T_3 = \left(29.2 \text{ in.}\right)\left(\tfrac{3}{8} \text{ in.}\right)\left(36 \text{ ksi}\right)$ $= 394 \text{ kips}$	$y_{T3} = 30 \text{ in.} - 6.66 \text{ in.} - \dfrac{\tfrac{3}{8} \text{ in.}}{2}$ $= 23.2 \text{ in.}$	$T_3 y_{T3} = 9{,}140 \text{ kip-in.}$

$$\begin{aligned} M_y &= \sum \left(\text{force}\right)\left(\text{moment arm}\right) \\ &= \frac{2{,}550 \text{ kip-in.} + 1{,}890 \text{ kip-in.} + 399 \text{ kip-in.} + 399 \text{ kip-in.} + 6{,}750 \text{ kip-in.} + 9{,}140 \text{ kip-in.}}{12 \text{ in./ft}} \\ &= 1{,}760 \text{ kip-ft} \end{aligned}$$

Now that both M_p and M_y have been determined, Equation I3-3b may be used in conjunction with the flexural slenderness values previously calculated to determine the nominal flexural strength of the composite section as follows:

$$M_n = M_p - \left(M_p - M_y\right)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \qquad \text{(}Spec\text{. Eq. I3-3b)}$$

$$= 1{,}840 \text{ kip-ft} - \left(1{,}840 \text{ kip-ft} - 1{,}760 \text{ kip-ft}\right)\left(\frac{77.9 - 64.1}{85.1 - 64.1}\right)$$

$$= 1{,}790 \text{ kip-ft}$$

The available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90\left(1{,}790 \text{ kip-ft}\right)$ $= 1{,}610 \text{ kip-ft}$	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{1{,}790 \text{ kip-ft}}{1.67}$ $= 1{,}070 \text{ kip-ft}$

Interaction of Flexure and Compression

Design of members for combined forces is performed in accordance with AISC *Specification* Section I5. For filled composite members with noncompact or slender sections, interaction may be determined in accordance with Section H1.1 as follows:

LRFD	ASD
$P_u = 1{,}310 \text{ kips}$ $M_u = 552 \text{ kip-ft}$ $\dfrac{P_r}{P_c} = \dfrac{P_u}{\phi_c P_n}$ $= \dfrac{1{,}310 \text{ kips}}{4{,}300 \text{ kips}}$ $= 0.305 > 0.2$ Therefore, use AISC *Specification* Equation H1-1a. $\dfrac{P_u}{\phi_c P_n} + \dfrac{8}{9}\left(\dfrac{M_u}{\phi_b M_n}\right) \le 1.0$ (from *Spec*. Eq. H1-1a) $0.305 + \dfrac{8}{9}\left(\dfrac{552 \text{ kip-ft}}{1{,}610 \text{ kip-ft}}\right) \le 1.0$ $0.610 < 1.0$ **o.k.**	$P_a = 1{,}370 \text{ kips}$ $M_a = 248 \text{ kip-ft}$ $\dfrac{P_r}{P_c} = \dfrac{P_a}{P_n / \Omega_c}$ $= \dfrac{1{,}370 \text{ kips}}{2{,}870 \text{ kips}}$ $= 0.477 > 0.2$ Therefore, use AISC *Specification* Equation H1-1a. $\dfrac{P_a}{P_n / \Omega_c} + \dfrac{8}{9}\left(\dfrac{M_a}{M_n / \Omega_b}\right) \le 1.0$ (from *Spec*. Eq. H1-1a) $0.477 + \dfrac{8}{9}\left(\dfrac{248 \text{ kip-ft}}{1{,}070 \text{ kip-ft}}\right) \le 1.0$ $0.683 < 1.0$ **o.k.**

The composite section is adequate; however, as there is available strength remaining for the trial plate thickness chosen, re-analyze the section to determine the adequacy of a reduced plate thickness.

Trial Size 2 (Slender)

The calculated geometric section properties using a reduced plate thickness of t = ¼ in. are:

$$
\begin{aligned}
B &= 30 \text{ in.}\\
H &= 30 \text{ in.}\\
A_g &= 900 \text{ in.}^2\\
A_c &= 870 \text{ in.}^2\\
A_s &= 29.8 \text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
b_i &= B - 2t\\
&= 30 \text{ in.} - 2\left(\tfrac{1}{4} \text{ in.}\right)\\
&= 29.5 \text{ in.}
\end{aligned}
$$

$$
\begin{aligned}
h_i &= H - 2t\\
&= 30 \text{ in.} - 2\left(\tfrac{1}{4} \text{ in.}\right)\\
&= 29.5 \text{ in.}
\end{aligned}
$$

$$
\begin{aligned}
E_c &= w_c^{1.5}\sqrt{f_c'}\\
&= \left(145 \text{ lb/ft}^3\right)^{1.5}\sqrt{7 \text{ ksi}}\\
&= 4{,}620 \text{ ksi}
\end{aligned}
$$

$$
\begin{aligned}
I_{gx} &= \frac{BH^3}{12}\\
&= \frac{(30 \text{ in.})(30 \text{ in.})^3}{12}\\
&= 67{,}500 \text{ in.}^4
\end{aligned}
$$

$$
\begin{aligned}
I_{cx} &= \frac{b_i h_i^3}{12}\\
&= \frac{(29.5 \text{ in.})(29.5 \text{ in.})^3}{12}\\
&= 63{,}100 \text{ in.}^4
\end{aligned}
$$

$$
\begin{aligned}
I_{sx} &= I_{gx} - I_{cx}\\
&= 67{,}500 \text{ in.}^4 - 63{,}100 \text{ in.}^4\\
&= 4{,}400 \text{ in.}^4
\end{aligned}
$$

Limitations of AISC Specification Sections I1.3 and I2.2a

(1) Concrete Strength: $3 \text{ ksi} \le f_c' \le 10 \text{ ksi}$

$f_c' = 7 \text{ ksi}$ **o.k.**

(2) Specified minimum yield stress of structural steel: $F_y \le 75 \text{ ksi}$

$F_y = 36 \text{ ksi}$ **o.k.**

(3) Cross sectional area of steel section: $A_s \geq 0.01A_g$

$$29.8 \text{ in.}^2 \geq (0.01)\left(900 \text{ in.}^2\right)$$

$$> 9.00 \text{ in.}^2 \quad \textbf{o.k.}$$

Classify Section for Local Buckling

As noted previously, the definitions of width, depth and thickness used in the evaluation of slenderness are provided in AISC *Specification* Section B4.1b.

For a box column, the slenderness ratio is determined as the ratio of clear distance-to-wall thickness:

$$\begin{aligned}\lambda &= \frac{b_i}{t} = \frac{h_i}{t}\\ &= \frac{29.5 \text{ in.}}{\frac{1}{4} \text{ in.}}\\ &= 118\end{aligned}$$

Classify section for local buckling in steel elements subject to axial compression from AISC *Specification* Table I1.1a. As determined previously, $\lambda_r = 85.1$.

$$\begin{aligned}\lambda_{max} &= 5.00\sqrt{\frac{E}{F_y}}\\ &= 5.00\sqrt{\frac{29{,}000 \text{ ksi}}{36 \text{ ksi}}}\\ &= 142\end{aligned}$$

$\lambda_r \leq \lambda \leq \lambda_{max}$; therefore, the section is slender for compression

Classification of the section for local buckling in elements subject to flexure occurs separately per AISC *Specification* Table I1.1b. Because the flange limitations for bending are the same as those for compression,

$\lambda_r \leq \lambda \leq \lambda_{max}$; therefore, the section is slender for flexure

Available Compressive Strength

Compressive strength for a slender filled member is determined in accordance with AISC *Specification* Section I2.2b(c).

$$F_{cr} = \frac{9E_s}{\left(\frac{b}{t}\right)^2} \qquad (\textit{Spec}.\ \text{Eq. I2-10})$$

$$\begin{aligned}&= \frac{9(29{,}000 \text{ ksi})}{(118)^2}\\ &= 18.7 \text{ ksi}\end{aligned}$$

$$P_{no} = F_{cr}A_s + 0.7f'_c\left(A_c + A_{sr}\frac{E_s}{E_c}\right) \qquad (Spec.\ \text{Eq. I2-9e})$$

$$= (18.7\text{ ksi})\left(29.8\text{ in.}^2\right) + 0.7(7\text{ ksi})\left(870\text{ in.}^2 + 0\text{ in.}^2\right)$$

$$= 4{,}820\text{ kips}$$

$$C_3 = 0.45 + 3\left(\frac{A_s + A_{sr}}{A_g}\right) \le 0.9 \qquad (Spec.\ \text{Eq. I2-13})$$

$$= 0.45 + 3\left(\frac{29.8\text{ in.}^2 + 0\text{ in.}^2}{900\text{ in.}^2}\right) \le 0.9$$

$$= 0.549 < 0.9$$

$$= 0.549$$

$$EI_{eff} = E_sI_s + E_sI_{sr} + C_3E_cI_c \qquad (Spec.\ \text{Eq. I2-12})$$

$$= (29{,}000\text{ ksi})\left(4{,}400\text{ in.}^4\right) + 0\text{ kip-in.}^2 + 0.549(4{,}620\text{ ksi})\left(63{,}100\text{ in.}^4\right)$$

$$= 288{,}000{,}000\text{ kip-in.}^2$$

$P_e = \pi^2\left(EI_{eff}\right)/\left(L_c\right)^2$, where $L_c = KL$ and $K = 1.0$ in accordance with the direct analysis method (*Spec*. Eq. I2-5)

$$= \frac{\pi^2\left(288{,}000{,}000\text{ kip-in.}^2\right)}{\left[(30\text{ ft})(12\text{ in./ft})\right]^2}$$

$$= 21{,}900\text{ kips}$$

$$\frac{P_{no}}{P_e} = \frac{4{,}820\text{ kips}}{21{,}900\text{ kips}}$$

$$= 0.220 < 2.25$$

Therefore, use AISC *Specification* Equation I2-2.

$$P_n = P_{no}\left(0.658^{\frac{P_{no}}{P_e}}\right) \qquad (Spec.\ \text{Eq. I2-2})$$

$$= (4{,}820\text{ kips})(0.658)^{0.220}$$

$$= 4{,}400\text{ kips}$$

According to AISC *Specification* Section I2.2b the compression strength need not be less than that determined for the bare steel member using *Specification* Chapter E. It can be shown that the compression strength of the bare steel for this section is equal to 450 kips, thus the strength of the composite section controls.

The available compressive strength is:

LRFD	ASD
$\phi_c = 0.75$ $\phi_c P_n = 0.75(4{,}400\text{ kips})$ $= 3{,}300\text{ kips}$	$\Omega_c = 2.00$ $\dfrac{P_n}{\Omega_c} = \dfrac{4{,}400\text{ kips}}{2.00}$ $= 2{,}200\text{ kips}$

Available Flexural Strength

Flexural strength of slender filled composite members is determined in accordance with AISC *Specification* Section I3.4b(c). The nominal flexural strength is determined as the first yield moment, M_{cr}, corresponding to a flange compression stress of F_{cr} using a linear elastic stress distribution with a maximum concrete compressive stress of $0.7f_c'$. This concept is illustrated diagrammatically in *Specification* Commentary Figure C-I3.7(c) and follows the force distribution depicted in Figure I.7-4 and detailed in Table I.7-3.

Table I.7-3. First Yield Moment Equations

Component	Force	Moment Arm
Compression in steel flange	$C_1 = b_i t_f F_{cr}$	$y_{C1} = a_{cr} - \dfrac{t_f}{2}$
Compression in concrete	$C_2 = 0.35 f_c'\left(a_{cr} - t_f\right) b_i$	$y_{C2} = \dfrac{2\left(a_{cr} - t_f\right)}{3}$
Compression in steel web	$C_3 = a_{cr} 2t_w 0.5F_{cr}$	$y_{C3} = \dfrac{2a_{cr}}{3}$
Tension in steel web	$T_1 = \left(H - a_{cr}\right) 2t_w 0.5F_y$	$y_{T1} = \dfrac{2\left(H - a_{cr}\right)}{3}$
Tension in steel flange	$T_2 = b_i t_f F_y$	$y_{T2} = H - a_{cr} - \dfrac{t_f}{2}$

where:

$$a_{cr} = \frac{F_y H t_w + \left(0.35 f_c' + F_y - F_{cr}\right) b_i t_f}{t_w\left(F_{cr} + F_y\right) + 0.35 f_c' b_i}$$

$$M_{cr} = \sum(\text{force})(\text{moment arm})$$

Using the equations provided in Table I.7-3 for the section in question results in the following:

$$\begin{aligned} a_{cr} &= \frac{(36\text{ ksi})(30\text{ in.})(\text{¼ in.}) + \left[0.35(7\text{ ksi}) + 36\text{ ksi} - 18.7\text{ ksi}\right](29.5\text{ in.})(\text{¼ in.})}{(\text{¼ in.})(18.7\text{ ksi} + 36\text{ ksi}) + 0.35(7\text{ ksi})(29.5\text{ in.})} \\ &= 4.84\text{ in.} \end{aligned}$$

Force	Moment Arm	Force × Moment Arm
$C_1 = (29.5\text{ in.})(\text{¼ in.})(18.7\text{ ksi})$ $= 138\text{ kips}$	$y_{C1} = 4.84\text{ in.} - \dfrac{\text{¼ in.}}{2}$ $= 4.72\text{ in.}$	$C_1 y_{C1} = 651\text{ kip-in.}$
$C_2 = 0.35(7\text{ ksi})(4.84\text{ in.} - \text{¼ in.})(29.5\text{ in.})$ $= 332\text{ kips}$	$y_{C2} = \dfrac{2(4.84\text{ in.} - \text{¼ in.})}{3}$ $= 3.06\text{ in.}$	$C_2 y_{C2} = 1{,}020\text{ kip-in.}$
$C_3 = (4.84\text{ in.})(2)(\text{¼ in.})(0.5)(18.7\text{ ksi})$ $= 22.6\text{ kips}$	$y_{C3} = \dfrac{2(4.84\text{ in.})}{3}$ $= 3.23\text{ in.}$	$C_3 y_{C3} = 73.0\text{ kip-in.}$
$T_1 = (30\text{ in.} - 4.84\text{ in.})(2)(\text{¼ in.})(0.5)(36\text{ ksi})$ $= 226\text{ kips}$	$y_{T1} = \dfrac{2(30\text{ in.} - 4.84\text{ in.})}{3}$ $= 16.8\text{ in.}$	$T_1 y_{T1} = 3{,}800\text{ kip-in.}$
$T_2 = (29.5\text{ in.})(\text{¼ in.})(36\text{ ksi})$ $= 266\text{ kips}$	$y_{T2} = 30\text{ in.} - 4.84\text{ in.} - \dfrac{\text{¼ in.}}{2}$ $= 25.0\text{ in.}$	$T_2 y_{T2} = 6{,}650\text{ kip-in.}$
$M_{cr} = \sum(\text{force component })(\text{moment arm})$ $= \dfrac{651\text{ kip-in.} + 1{,}020\text{ kip-in.} + 73.0\text{ kip-in.} + 3{,}800\text{ kip-in.} + 6{,}650\text{ kip-in.}}{12\text{ in./ft}}$ $= 1{,}020\text{ kip-ft}$		

The available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $M_n = 0.90(1{,}020\text{ kip-ft})$ $= 918\text{ kip-ft}$	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{1{,}020\text{ kip-ft}}{1.67}$ $= 611\text{ kip-ft}$

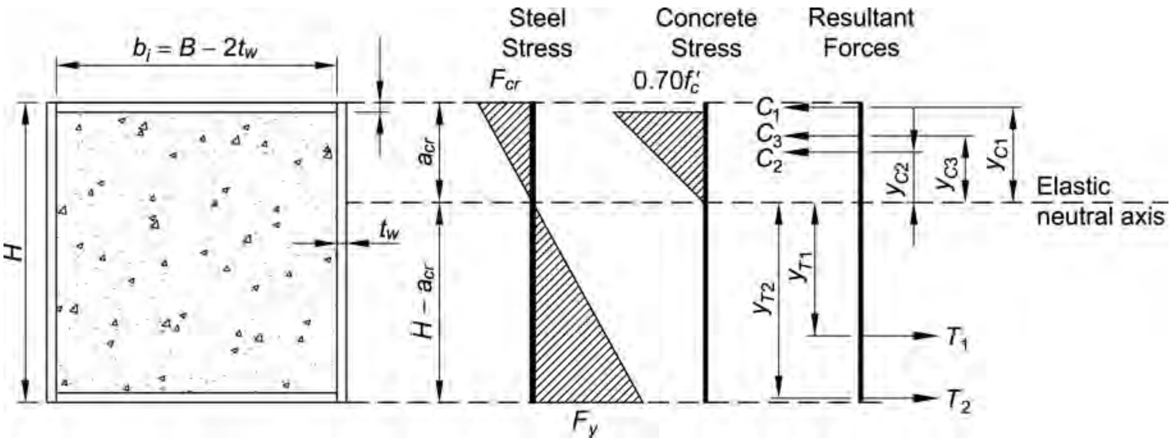


Figure I.7-4. First yield moment stress blocks and force distribution.

Interaction of Flexure and Compression

The interaction of flexure and compression may be determined in accordance with AISC *Specification* Section H1.1 as follows:

LRFD	ASD
$P_u = 1{,}310 \text{ kips}$ $M_u = 552 \text{ kip-ft}$	$P_a = 1{,}370 \text{ kips}$ $M_a = 248 \text{ kip-ft}$
$\frac{P_r}{P_c} = \frac{P_u}{\phi_c P_n}$ $= \frac{1{,}310 \text{ kips}}{3{,}300 \text{ kips}}$ $= 0.397 > 0.2$	$\frac{P_r}{P_c} = \frac{P_a}{P_n / \Omega_c}$ $= \frac{1{,}370 \text{ kips}}{2{,}200 \text{ kips}}$ $= 0.622 > 0.2$
Therefore, use AISC *Specification* Equation H1-1a.	Therefore, use AISC *Specification* Equation H1-1a.
$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_u}{\phi_b M_n}\right) \le 1.0$ (from *Spec.* Eq. H1-1a) $0.397 + \frac{8}{9}\left(\frac{552 \text{ kip-ft}}{918 \text{ kip-ft}}\right) \le 1.0$ $0.931 < 1.0$ **o.k.**	$\frac{P_a}{P_n / \Omega_c} + \frac{8}{9}\left(\frac{M_a}{M_n / \Omega_c}\right) \le 1.0$ (from *Spec.* Eq. H1-1a) $0.622 + \frac{8}{9}\left(\frac{248 \text{ kip-ft}}{611 \text{ kip-ft}}\right) \le 1.0$ $0.983 < 1.0$ **o.k.**

Thus, a plate thickness of ¼ in. is adequate.

Note that in addition to the design checks performed for the composite condition, design checks for other load stages should be performed as required by AISC *Specification* Section I1. These checks should take into account the effect of hydrostatic loads from concrete placement as well as the strength of the steel section alone prior to composite action.

Available Shear Strength

According to AISC *Specification* Section I4.1, there are three acceptable methods for determining the available shear strength of the member: available shear strength of the steel section alone in accordance with Chapter G; available shear strength of the reinforced concrete portion alone per ACI 318; or available shear strength of the steel section in addition to the reinforcing steel ignoring the contribution of the concrete. Considering that the member in question does not have longitudinal reinforcing, it is determined by inspection that the shear strength will be controlled by the steel section alone using the provisions of Chapter G.

From AISC *Specification* Section G4, the nominal shear strength, V_n, of box members is determined using AISC *Specification* Equation G4-1 with C_{v2} determined from AISC *Specification* Section G2.2 with $k_v = 5$. As opposed to HSS sections that require the use of a reduced web area to take into account the corner radii, the web area of a box section may be used as follows:

$$\begin{aligned} A_w &= 2ht_w, \text{ where } h = \text{clear distance between flanges} \\ &= 2(29.5 \text{ in.})(\text{¼ in.}) \\ &= 14.8 \text{ in.}^2 \end{aligned}$$

The slenderness value, $h/t_w = h/t$, which is the same as that calculated previously for use in local buckling classification, $\lambda = 118$.

$$1.37\sqrt{k_v E/F_y} = 1.37\sqrt{5\left(\frac{29{,}000 \text{ ksi}}{36 \text{ ksi}}\right)}$$
$$= 86.9 < h/t = 118$$

Therefore, use AISC *Specification* Equation G2-11 to calculate C_{v2}.

The web shear coefficient and nominal shear strength are calculated as:

$$C_{v2} = \frac{1.51k_v E}{(h/t_w)^2 F_y} \qquad \text{(}\textit{Spec}\text{. Eq. G2-11)}$$
$$= \frac{1.51(5)(29{,}000 \text{ ksi})}{(118)^2(36 \text{ ksi})}$$
$$= 0.437$$

$$V_n = 0.6F_y A_w C_{v2}$$
$$= 0.6(36 \text{ ksi})(14.8 \text{ in.}^2)(0.437) \qquad \text{(}\textit{Spec}\text{. Eq. G4-1)}$$
$$= 140 \text{ kips}$$

The available shear strength is checked as follows:

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(140 \text{ kips})$ $= 126 \text{ kips} > 36.8 \text{ kips}$ **o.k.**	$\Omega_v = 1.67$ $\frac{V_n}{\Omega_v} = \frac{140 \text{ kips}}{1.67}$ $= 83.8 \text{ kips} > 22.1 \text{ kips}$ **o.k.**

Force Allocation and Load Transfer

Load transfer calculations for applied axial forces should be performed in accordance with AISC *Specification* Section I6. The specific application of the load transfer provisions is dependent upon the configuration and detailing of the connecting elements. Expanded treatment of the application of load transfer provisions is provided in Example I.3.

Summary

It has been determined that a 30 in. × 30 in. composite box column composed of ¼-in.-thick plate is adequate for the imposed loads.

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EXAMPLE I.8 ENCASED COMPOSITE MEMBER FORCE ALLOCATION AND LOAD TRANSFER

Given:

Refer to Figure I.8-1.

Part I: For each loading condition (a) through (c), determine the required longitudinal shear force, V_r', to be transferred between the embedded steel section and concrete encasement.

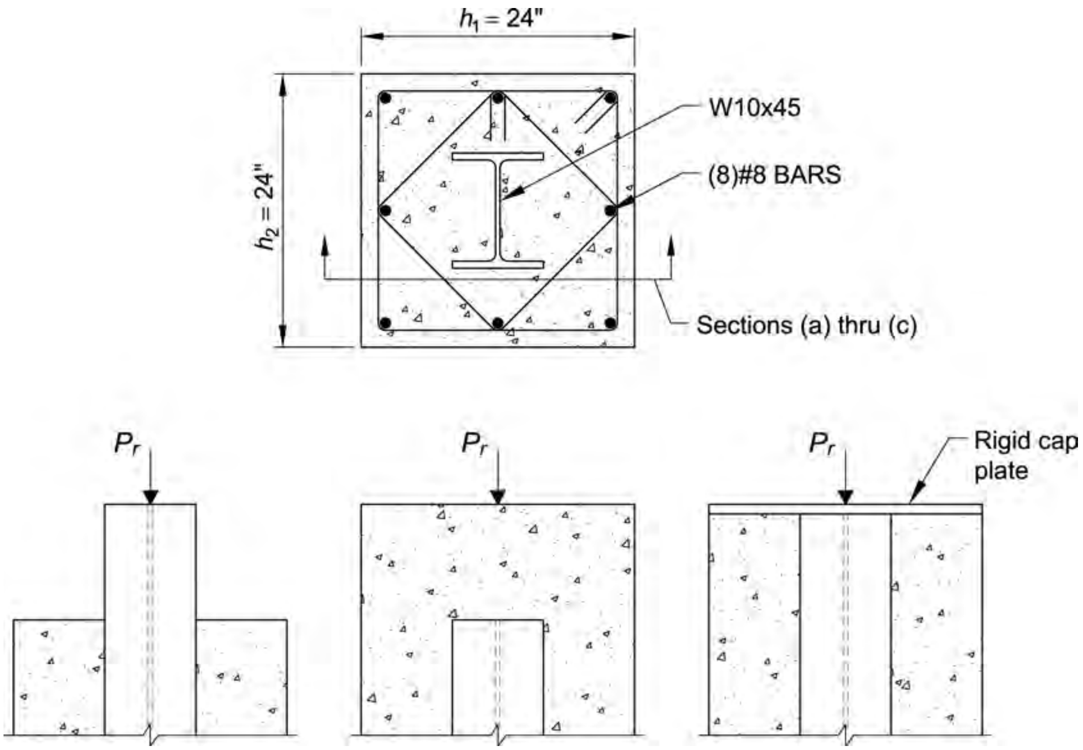
Part II: For loading condition (b), investigate the force transfer mechanisms of direct bearing and shear connection.

The composite member consists of an ASTM A992 W-shape encased by normal weight (145 lb/ft^3) reinforced concrete having a specified concrete compressive strength, $f_c' = 5$ ksi.

Deformed reinforcing bars conform to ASTM A615 with a minimum yield stress, F_{yr}, of 60 ksi.

Applied loading, P_r, for each condition illustrated in Figure I.8-1 is composed of the following loads:

$P_D = 260$ kips
$P_L = 780$ kips



(a) External force to steel only

(b) External force to concrete only

(c) External force to both materials concurrently

Fig. I.8-1. Encased composite member in compression.

Solution:

Part I—Force Allocation

From AISC *Manual* Table 2-4, the steel material properties are:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 1-1 and Figure I.8-1, the geometric properties of the encased W10×45 are as follows:

$$\begin{aligned}A_s &= 13.3\text{ in.}^2\\ b_f &= 8.02\text{ in.}\\ t_f &= 0.620\text{ in.}\\ t_w &= 0.350\text{ in.}\\ d &= 10.1\text{ in.}\\ h_1 &= 24\text{ in.}\\ h_2 &= 24\text{ in.}\end{aligned}$$

Additional geometric properties of the composite section used for force allocation and load transfer are calculated as follows:

$$\begin{aligned}A_g &= h_1 h_2\\ &= (24\text{ in.})(24\text{ in.})\\ &= 576\text{ in.}^2\end{aligned}$$

$$A_{sri} = 0.79\text{ in.}^2 \text{ for a No. 8 bar}$$

$$\begin{aligned}A_{sr} &= \sum_{i=1}^{n} A_{sri}\\ &= 8\left(0.79\text{ in.}^2\right)\\ &= 6.32\text{ in.}^2\end{aligned}$$

$$\begin{aligned}A_c &= A_g - A_s - A_{sr}\\ &= 576\text{ in.}^2 - 13.3\text{ in.}^2 - 6.32\text{ in.}^2\\ &= 556\text{ in.}^2\end{aligned}$$

where
A_c = cross-sectional area of concrete encasement, in.^{2}
A_g = gross cross-sectional area of composite section, in.^{2}
A_{sri} = cross-sectional area of reinforcing bar i, in.^{2}
A_{sr} = cross-sectional area of continuous reinforcing bars, in.^{2}
n = number of continuous reinforcing bars in composite section

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$P_r = P_u$ $= 1.2(260\text{ kips}) + 1.6(780\text{ kips})$ $= 1{,}560\text{ kips}$	$P_r = P_a$ $= 260\text{ kips} + 780\text{ kips}$ $= 1{,}040\text{ kips}$

Composite Section Strength for Force Allocation

In accordance with AISC *Specification* Section I6, force allocation calculations are based on the nominal axial compressive strength of the encased composite member without length effects, P_{no}. This section strength is defined in Section I2.1b as:

$$\begin{aligned} P_{no} &= F_y A_s + F_{ysr} A_{sr} + 0.85 f_c' A_c \\ &= (50\text{ ksi})(13.3\text{ in.}^2) + (60\text{ ksi})(6.32\text{ in.}^2) + 0.85(5\text{ ksi})(556\text{ in.}^2) \\ &= 3{,}410\text{ kips} \end{aligned} \qquad (Spec.\text{ Eq. I2-4})$$

Transfer Force for Condition (a)

Refer to Figure I.8-1(a). For this condition, the entire external force is applied to the steel section only, and the provisions of AISC *Specification* Section I6.2a apply.

$$\begin{aligned} V_r' &= P_r\left(1 - \frac{F_y A_s}{P_{no}}\right) \\ &= P_r\left[1 - \frac{(50\text{ ksi})(13.3\text{ in.}^2)}{3{,}410\text{ kips}}\right] \\ &= 0.805P_r \end{aligned} \qquad (Spec.\text{ Eq. I6-1})$$

LRFD	ASD
$V_r' = 0.805(1{,}560\text{ kips})$ $= 1{,}260\text{ kips}$	$V_r' = 0.805(1{,}040\text{ kips})$ $= 837\text{ kips}$

Transfer Force for Condition (b)

Refer to Figure I.8-1(b). For this condition, the entire external force is applied to the concrete encasement only, and the provisions of AISC *Specification* Section I6.2b apply.

$$\begin{aligned} V_r' &= P_r\left(\frac{F_y A_s}{P_{no}}\right) \\ &= P_r\left[\frac{(50\text{ ksi})(13.3\text{ in.}^2)}{3{,}410\text{ kips}}\right] \\ &= 0.195P_r \end{aligned} \qquad (Spec.\text{ Eq. I6-2a})$$

LRFD	ASD
$V_r' = 0.195(1{,}560\text{ kips})$ $= 304\text{ kips}$	$V_r' = 0.195(1{,}040\text{ kips})$ $= 203\text{ kips}$

Transfer Force for Condition (c)

Refer to Figure I.8-1(c). For this condition, external force is applied to the steel section and concrete encasement concurrently, and the provisions of AISC *Specification* Section I6.2c apply.

AISC *Specification* Commentary Section I6.2 states that when loads are applied to both the steel section and concrete encasement concurrently, V_r' can be taken as the difference in magnitudes between the portion of the external force applied directly to the steel section and that required by Equation I6-2a. This concept can be written in equation form as follows:

$$V_r' = \left| P_{rs} - P_r \left(\frac{F_y A_s}{P_{no}} \right) \right| \qquad \text{(Eq. 1)}$$

where

P_{rs} = portion of external force applied directly to the steel section, kips

Currently, the *Specification* provides no specific requirements for determining the distribution of the applied force for the determination of P_{rs}, so it is left to engineering judgment. For a bearing plate condition such as the one represented in Figure I.8-1(c), one possible method for determining the distribution of applied forces is to use an elastic distribution based on the material axial stiffness ratios as follows:

$$\begin{aligned} E_c &= w_c^{1.5}\sqrt{f_c'} \\ &= \left(145 \text{ lb/ft}^3\right)^{1.5}\sqrt{5 \text{ ksi}} \\ &= 3{,}900 \text{ ksi} \end{aligned}$$

$$\begin{aligned} P_{rs} &= \left(\frac{E_s A_s}{E_s A_s + E_c A_c + E_{sr} A_{sr}} \right) P_r \\ &= \left[\frac{(29{,}000 \text{ ksi})\left(13.3 \text{ in.}^2\right)}{(29{,}000 \text{ ksi})\left(13.3 \text{ in.}^2\right) + (3{,}900 \text{ ksi})\left(556 \text{ in.}^2\right) + (29{,}000 \text{ ksi})\left(6.32 \text{ in.}^2\right)} \right] P_r \\ &= 0.141 P_r \end{aligned}$$

Substituting the results into Equation 1 yields:

$$\begin{aligned} V_r' &= \left| 0.141 P_r - P_r \left(\frac{F_y A_s}{P_{no}} \right) \right| \\ &= \left| 0.141 P_r - P_r \left[\frac{(50 \text{ ksi})\left(13.3 \text{ in.}^2\right)}{3{,}410 \text{ kips}} \right] \right| \\ &= 0.0540 P_r \end{aligned}$$

LRFD	ASD
$V_r' = 0.0540(1{,}560 \text{ kips})$ $= 84.2 \text{ kips}$	$V_r' = 0.0540(1{,}040 \text{ kips})$ $= 56.2 \text{ kips}$

An alternate approach would be use of a plastic distribution method whereby the load is partitioned to each material in accordance with their contribution to the composite section strength given in Equation I2-4. This method eliminates the need for longitudinal shear transfer provided the local bearing strength of the concrete and steel are adequate to resist the forces resulting from this distribution.

Additional Discussion

- The design and detailing of the connections required to deliver external forces to the composite member should be performed according to the applicable sections of AISC *Specification* Chapters J and K.

- The connection cases illustrated by Figure I.8-1 are idealized conditions representative of the mechanics of actual connections. For instance, an extended single plate connection welded to the flange of the W10 and extending out beyond the face of concrete to attach to a steel beam is an example of a condition where it may be assumed that all external force is applied directly to the steel section only.

Solution:

Part II—Load Transfer

The required longitudinal force to be transferred, V_r', determined in Part I condition (b) is used to investigate the applicable force transfer mechanisms of AISC *Specification* Section I6.3: direct bearing and shear connection. As indicated in the *Specification*, these force transfer mechanisms may not be superimposed; however, the mechanism providing the greatest nominal strength may be used. Note that direct bond interaction is not applicable to encased composite members as the variability of column sections and connection configurations makes confinement and bond strength more difficult to quantify than in filled HSS.

Direct Bearing

Determine Layout of Bearing Plates

One method of utilizing direct bearing as a load transfer mechanism is through the use of internal bearing plates welded between the flanges of the encased W-shape as indicated in Figure I.8-2.

When using bearing plates in this manner, it is essential that concrete mix proportions and installation techniques produce full bearing at the plates. Where multiple sets of bearing plates are used as illustrated in Figure I.8-2, it is recommended that the minimum spacing between plates be equal to the depth of the encased steel member to enhance constructability and concrete consolidation. For the configuration under consideration, this guideline is met with a plate spacing of $24 \text{ in.} \geq d = 10.1 \text{ in.}$

Bearing plates should be located within the load introduction length given in AISC *Specification* Section I6.4a. The load introduction length is defined as two times the minimum transverse dimension of the composite member both above and below the load transfer region. The load transfer region is defined in *Specification* Commentary Section I6.4 as the depth of the connection. For the connection configuration under consideration, where the majority of the required force is being applied from the concrete column above, the depth of connection is conservatively taken as zero. Because the composite member only extends to one side of the point of force transfer, the bearing plates should be located within $2h_2$ = 48 in. of the top of the composite member as indicated in Figure I.8-2.

Available Strength for the Limit State of Direct Bearing

Assuming two sets of bearing plates are to be used as indicated in Figure I.8-2, the total contact area between the bearing plates and the concrete, A_1, may be determined as follows:

$$a = \frac{b_f - t_w}{2}$$

$$= \frac{8.02\text{ in.} - 0.350\text{ in.}}{2}$$

$$= 3.84\text{ in.}$$

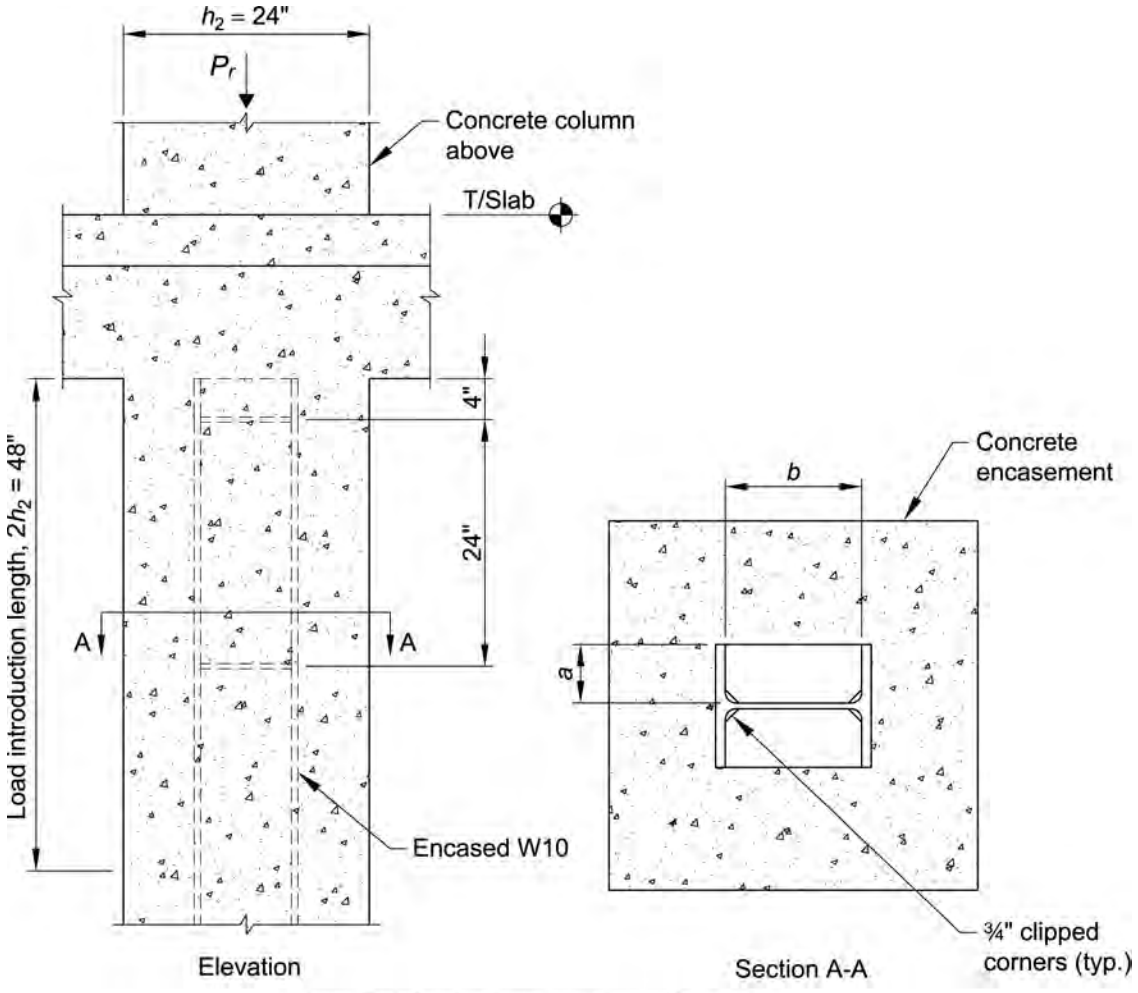
$$b = d - 2t_f$$

$$= 10.1\text{ in.} - 2(0.620\text{ in.})$$

$$= 8.86\text{ in.}$$

c = width of clipped corners

= ¾ in.



Note: Reinforcing bars not shown for clarity.

Fig. I.8-2. Composite member with internal bearing plates.

$$
\begin{aligned}
A_1 &= \left(2ab - 2c^2\right)\left(\text{number of bearing plate sets}\right) \\
&= \left[2\left(3.84 \text{ in.}\right)\left(8.86 \text{ in.}\right) - 2\left(\tfrac{3}{4} \text{ in.}\right)^2\right]\left(2\right) \\
&= 134 \text{ in.}^2
\end{aligned}
$$

The available strength for the direct bearing force transfer mechanism is:

$$
\begin{aligned}
R_n &= 1.7 f_c' A_1 \\
&= 1.7\left(5 \text{ ksi}\right)\left(134 \text{ in.}^2\right) \qquad (\textit{Spec}.\text{ Eq. I6-3}) \\
&= 1{,}140 \text{ kips}
\end{aligned}
$$

LRFD	ASD
$\phi_B = 0.65$ $\phi_B R_n = 0.65\left(1{,}140 \text{ kips}\right)$ $= 741 \text{ kips} > V_r' = 304 \text{ kips}$ **o.k.**	$\Omega_B = 2.31$ $\dfrac{R_n}{\Omega_B} = \dfrac{1{,}140 \text{ kips}}{2.31}$ $= 494 \text{ kips} > V_r' = 203 \text{ kips}$ **o.k.**

Thus, two sets of bearing plates are adequate. From these calculations, it can be seen that one set of bearing plates are adequate for force transfer purposes; however, the use of two sets of bearing plates serves to reduce the bearing plate thickness calculated in the following section.

Required Bearing Plate Thickness

There are several methods available for determining the bearing plate thickness. For rectangular plates supported on three sides, elastic solutions for plate stresses, such as those found in *Roark's Formulas for Stress and Strain* (Young and Budynas, 2002), may be used in conjunction with AISC *Specification* Section F12 for thickness calculations. Alternately, yield line theory or computational methods such as finite element analysis may be employed.

For this example, yield line theory is employed. Results of the yield line analysis depend on an assumption of column flange strength versus bearing plate strength in order to estimate the fixity of the bearing plate to column flange connection. In general, if the thickness of the bearing plate is less than the column flange thickness, fixity and plastic hinging can occur at this interface; otherwise, the use of a pinned condition is conservative. Ignoring the fillets of the W-shape and clipped corners of the bearing plate, the yield line pattern chosen for the fixed condition is depicted in Figure I.8-3. Note that the simplifying assumption of 45° yield lines illustrated in Figure I.8-3 has been shown to provide reasonably accurate results (Park and Gamble, 2000), and that this yield line pattern is only valid where $b \geq 2a$.

The plate thickness using $F_y = 36$ ksi material may be determined as:

LRFD	ASD
$\phi = 0.90$ If $t_p \geq t_f$: $t_p = \sqrt{\dfrac{2a^2 w_u \left(3b - 2a\right)}{3\phi F_y \left(4a + b\right)}}$	$\Omega = 1.67$ If $t_p \geq t_f$: $t_p = \sqrt{\left(\dfrac{2\Omega}{3F_y}\right)\left[\dfrac{a^2 w_a \left(3b - 2a\right)}{\left(4a + b\right)}\right]}$

LRFD	ASD
If $t_p < t_f$: $t_p = \sqrt{\dfrac{2a^2 w_u (3b-2a)}{3\phi F_y (6a+b)}}$ where w_u = bearing pressure on plate determined using LRFD load combinations $= \dfrac{V_r'}{A_1}$ $= \dfrac{304\text{ kips}}{134\text{ in.}^2}$ $= 2.27\text{ ksi}$ Assuming $t_p \geq t_f$ $t_p = \sqrt{\dfrac{2(3.84\text{ in.})^2(2.27\text{ ksi})\times[3(8.86\text{ in.})-2(3.84\text{ in.})]}{3(0.90)(36\text{ ksi})[4(3.84\text{ in.})+8.86\text{ in.}]}}$ $= 0.733\text{ in.}$ Select ¾-in. plate. t_p = ¾ in. > t_f = 0.620 in. **assumption o.k.**	If $t_p < t_f$: $t_p = \sqrt{\left(\dfrac{2\Omega}{3F_y}\right)\left[\dfrac{a^2 w_a (3b-2a)}{(6a+b)}\right]}$ where w_a = bearing pressure on plate determined using ASD load combinations $= \dfrac{V_r'}{A_1}$ $= \dfrac{203\text{ kips}}{134\text{ in.}^2}$ $= 1.51\text{ ksi}$ Assuming $t_p \geq t_f$ $t_p = \sqrt{\dfrac{2(1.67)(3.84\text{ in.})^2(1.51\text{ ksi})\times[3(8.86\text{ in.})-2(3.84\text{ in.})]}{3(36\text{ ksi})[4(3.84\text{ in.})+8.86\text{ in.}]}}$ $= 0.733\text{ in.}$ Select ¾-in. plate t_p = ¾ in. > t_f = 0.620 in. **assumption o.k.**

Thus, select ¾-in.-thick bearing plates.

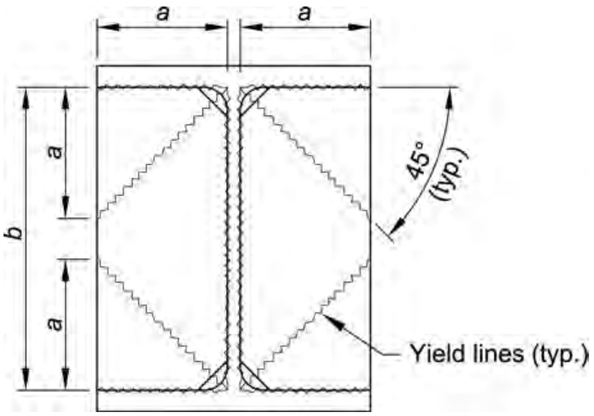


Fig. I.8-3. Internal bearing plate yield line pattern (fixed condition).

Bearing Plate to Encased Steel Member Weld

The bearing plates should be connected to the encased steel member using welds designed in accordance with AISC *Specification* Chapter J to develop the full strength of the plate. For fillet welds, a weld size of $\frac{5}{8}t_p$ will serve to develop the strength of either a 36- or 50-ksi plate as discussed in AISC *Manual* Part 10.

Shear Connection

Shear connection involves the use of steel headed stud or channel anchors placed on at least two faces of the steel shape in a generally symmetric configuration to transfer the required longitudinal shear force. For this example, ¾-in.-diameter × 4³⁄₁₆-in.-long steel headed stud anchors composed of ASTM A108 material are selected. The specified minimum tensile strength, F_u, of ASTM A108 material is 65 ksi.

Available Shear Strength of Steel Headed Stud Anchors

The available shear strength of an individual steel headed stud anchor is determined in accordance with the composite component provisions of AISC *Specification* Section I8.3 as directed by Section I6.3b.

$$Q_{nv} = F_u A_{sa} \qquad \text{(}\textit{Spec}\text{. Eq. I8-3)}$$

$$\begin{aligned} A_{sa} &= \frac{\pi(\text{¾ in.})^2}{4} \\ &= 0.442 \text{ in.}^2 \end{aligned}$$

LRFD	ASD
$\phi_v = 0.65$ $\phi_v Q_{nv} = 0.65(65 \text{ ksi})(0.442 \text{ in.}^2)$ $= 18.7 \text{ kips per steel headed stud anchor}$	$\Omega_v = 2.31$ $\dfrac{Q_{nv}}{\Omega_v} = \dfrac{(65 \text{ ksi})(0.442 \text{ in.}^2)}{2.31}$ $= 12.4 \text{ kips per steel headed stud anchor}$

Required Number of Steel Headed Stud Anchors

The number of steel headed stud anchors required to transfer the longitudinal shear is calculated as follows:

LRFD	ASD
$n_{anchors} = \dfrac{V_r'}{\phi_v Q_{nv}}$ $= \dfrac{304 \text{ kips}}{18.7 \text{ kips}}$ $= 16.3 \text{ steel headed stud anchors}$	$n_{anchors} = \dfrac{V_r'}{Q_{nv}/\Omega_v}$ $= \dfrac{203 \text{ kips}}{12.4 \text{ kips}}$ $= 16.4 \text{ steel headed stud anchors}$

With anchors placed in pairs on each flange, select 20 anchors to satisfy the symmetry provisions of AISC *Specification* Section I6.4a.

Placement of Steel Headed Stud Anchors

Steel headed stud anchors are placed within the load introduction length in accordance with AISC *Specification* Section I6.4a. Because the composite member only extends to one side of the point of force transfer, the steel anchors are located within $2h_2 = 48$ in. of the top of the composite member.

Placing two anchors on each flange provides four anchors per group, and maximum stud spacing within the load introduction length is determined as:

$$s_{max} = \frac{\text{load introduction length} - \text{distance to first anchor group from upper end of encased shape}}{\left[\dfrac{\text{total number of anchors}}{\text{number of anchors per group}}\right] - 1}$$

$$= \frac{48\text{ in.} - 6\text{ in.}}{\left[\dfrac{20\text{ anchors}}{4\text{ anchors per group}}\right] - 1}$$

$$= 10.5\text{ in.}$$

Use 10 in. spacing beginning 6 in. from top of encased member.

In addition to anchors placed within the load introduction length, anchors must also be placed along the remainder of the composite member at a maximum spacing of 32 times the anchor shank diameter = 24 in. in accordance with AISC *Specification* Sections I6.4a and I8.3e.

The chosen anchor layout and spacing is illustrated in Figure I.8-4.

Steel Headed Stud Anchor Detailing Limitations of AISC Specification *Sections I6.4a, I8.1 and I8.3*

Steel headed stud anchor detailing limitations are reviewed in this section with reference to the anchor configuration provided in Figure I.8-4 for anchors having a shank diameter, d_{sa}, of ¾ in. Note that these provisions are specific to the detailing of the anchors themselves and that additional limitations for the structural steel, concrete and reinforcing components of composite members should be reviewed as demonstrated in Design Example I.9.

(1) Anchors must be placed on at least two faces of the steel shape in a generally symmetric configuration:

Anchors are located in pairs on both faces. **o.k.**

(2) Maximum anchor diameter: $d_{sa} \leq 2.5\left(t_f\right)$

$$\tfrac{3}{4}\text{ in.} < 2.5\left(0.620\text{ in.}\right) = 1.55\text{ in.} \quad \textbf{o.k.}$$

(3) Minimum steel headed stud anchor height-to-diameter ratio: $h/d_{sa} \geq 5$

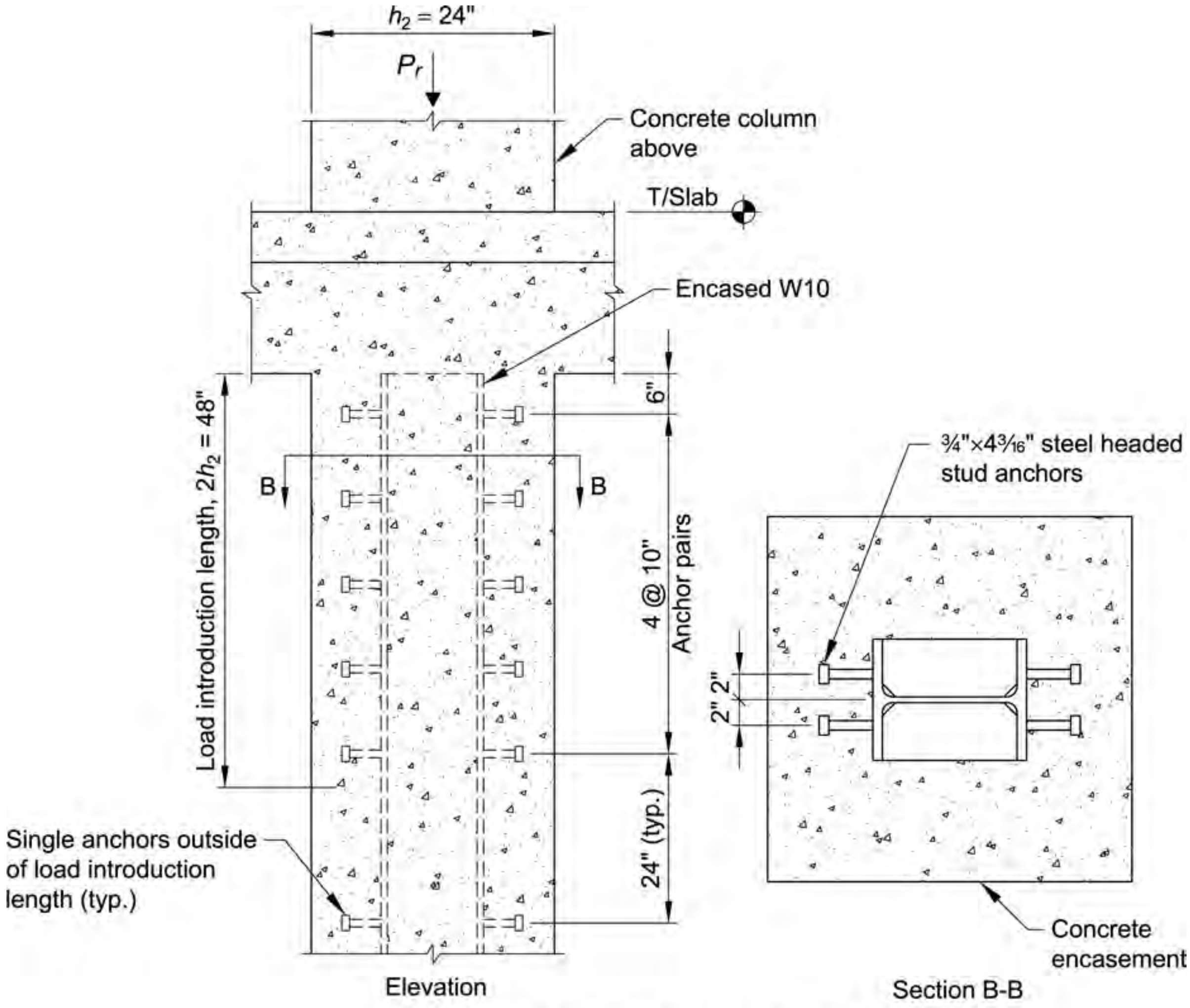
The minimum ratio of installed anchor height (base to top of head), h, to shank diameter, d_{sa}, must meet the provisions of AISC *Specification* Section I8.3 as summarized in the User Note table at the end of the section. For shear in normal weight concrete the limiting ratio is five. As previously discussed, a $4\frac{3}{16}$-in.-long anchor was selected from anchor manufacturer's data. As the h/d_{sa} ratio is based on the installed length, a length reduction for burn off during installation of $\frac{3}{16}$ in. is taken to yield the final installed length of 4 in.

$$\frac{h}{d_{sa}} = \frac{4\text{ in.}}{\tfrac{3}{4}\text{ in.}} = 5.33 > 5 \quad \textbf{o.k.}$$

(4) Minimum lateral clear concrete cover = 1½ in.

From AWS D1.1 (AWS, 2015) Figure 7.1, the head diameter of a ¾-in.-diameter stud anchor is equal to 1.25 in.

$$\begin{aligned}\text{lateral clear cover} &= \left(\frac{h_1}{2}\right)-\left(\frac{\text{lateral spacing between anchor centerlines}}{2}\right)-\left(\frac{\text{anchor head diameter}}{2}\right)\\ &= \left(\frac{24\text{ in.}}{2}\right)-\left(\frac{4\text{ in.}}{2}\right)-\left(\frac{1.25\text{ in.}}{2}\right)\\ &= 9.38\text{ in.} > 1\tfrac{1}{2}\text{ in.} \quad \textbf{o.k.}\end{aligned}$$



Note: Reinforcing bars not shown for clarity.

Fig. I.8-4. Composite member with steel anchors.

(5) Minimum anchor spacing:

$$\begin{aligned} s_{min} &= 4d_{sa} \\ &= 4(\tfrac{3}{4}\text{ in.}) \\ &= 3.00\text{ in.} \end{aligned}$$

In accordance with AISC *Specification* Section I8.3e, this spacing limit applies in any direction.

$$s_{transverse} = 4\text{ in.} > s_{min} \quad \textbf{o.k.}$$
$$s_{longitudinal} = 10\text{ in.} > s_{min} \quad \textbf{o.k.}$$

(6) Maximum anchor spacing:

$$\begin{aligned} s_{max} &= 32d_{sa} \\ &= 32(\tfrac{3}{4}\text{ in.}) \\ &= 24.0\text{ in.} \end{aligned}$$

In accordance with AISC *Specification* Section I6.4a, the spacing limits of Section I8.3e apply to steel anchor spacing both within and outside of the load introduction region.

$$s = 24.0\text{ in.} \le s_{max} \quad \textbf{o.k.}$$

(7) Clear cover above the top of the steel headed stud anchors:

Minimum clear cover over the top of the steel headed stud anchors is not explicitly specified for steel anchors in composite components; however, in keeping with the intent of AISC *Specification* Section I1.1, it is recommended that the clear cover over the top of the anchor head follow the cover requirements of ACI 318 (ACI 318, 2014) Section 20.6.1. For concrete columns, ACI 318 specifies a clear cover of 1½ in.

$$\begin{aligned} \text{clear cover above anchor} &= \frac{h_2}{2} - \frac{d}{2} - \text{installed anchor length} \\ &= \frac{24\text{ in.}}{2} - \frac{10.1\text{ in.}}{2} - 4\text{ in.} \\ &= 2.95\text{ in.} > 1\tfrac{1}{2}\text{ in.} \quad \textbf{o.k.} \end{aligned}$$

Concrete Breakout

AISC *Specification* Section I8.3a states that in order to use Equation I8-3 for shear strength calculations as previously demonstrated, concrete breakout strength in shear must not be an applicable limit state. If concrete breakout is deemed to be an applicable limit state, the *Specification* provides two alternatives: either the concrete breakout strength can be determined explicitly using ACI 318, Chapter 17, in accordance with *Specification* Section I8.3a(b), or anchor reinforcement can be provided to resist the breakout force as discussed in *Specification* Section I8.3a(a).

Determining whether concrete breakout is a viable failure mode is left to the engineer. According to AISC *Specification* Commentary Section I8.3, "it is important that it be deemed by the engineer that a concrete breakout failure mode in shear is directly avoided through having the edges perpendicular to the line of force supported, and the edges parallel to the line of force sufficiently distant that concrete breakout through a side edge is not deemed viable."

For the composite member being designed, no free edge exists in the direction of shear transfer along the length of the column, and concrete breakout in this direction is not an applicable limit state. However, it is still incumbent upon the engineer to review the possibility of concrete breakout through a side edge parallel to the line of force.

One method for explicitly performing this check is through the use of the provisions of ACI 318, Chapter 17, as follows:

ACI 318, Section 17.5.2.1(c), specifies that concrete breakout shall be checked for shear force parallel to the edge of a group of anchors using twice the value for the nominal breakout strength provided by ACI 318, Equation 17.5.2.1b, when the shear force in question acts perpendicular to the edge.

For the composite member being designed, symmetrical concrete breakout planes form to each side of the encased shape, one of which is illustrated in Figure I.8-5.

$\phi = 0.75$ for anchors governed by concrete breakout with supplemental reinforcement (provided by tie reinforcement) in accordance with ACI 318, Section 17.3.3

$$V_{cbg} = 2\left[\frac{A_{Vc}}{A_{Vco}}\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_b\right], \text{ for shear force parallel to an edge} \qquad \text{(ACI 318, Eq. 17.5.2.1b)}$$

$$\begin{aligned} A_{Vco} &= 4.5(c_{a1})^2 \qquad \text{(ACI 318, Eq. 17.5.2.1c)} \\ &= 4.5(10\text{ in.})^2 \\ &= 450\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{Vc} &= (15\text{ in.} + 40\text{ in.} + 15\text{ in.})(24\text{ in.}), \text{ from Figure I.8-5} \\ &= 1{,}680\text{ in.}^2 \end{aligned}$$

$\Psi_{ec,V} = 1.0$ no eccentricity

$\Psi_{ed,V} = 1.0$ in accordance with ACI 318, Section 17.5.2.1(c)

$\Psi_{c,V} = 1.4$ compression-only member assumed uncracked

$\Psi_{h,V} = 1.0$

$$V_b = \left[8\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right]\lambda_a\sqrt{f_c'}\,(c_{a1})^{1.5} \qquad \text{(ACI 318, Eq. 17.5.2.3)}$$

where

$l_e = 4\text{ in.} - \text{⅜-in.}$ anchor head thickness from AWS D1.1, Figure 7.1

$= 3.63\text{ in.}$

$d_a =$ ¾-in. anchor diameter

$\lambda_a = 1.0\lambda$ from ACI 318, Section 17.2.6, for normal weight concrete

$\lambda = 1.0$ from ACI 318, Table 19.2.4.2, for normal weight concrete

$$V_b = \left[8\left(\frac{3.63\text{ in.}}{\text{¾ in.}}\right)^{0.2}\sqrt{\text{¾ in.}}\right](1.0)\frac{\sqrt{5{,}000\text{ psi}}}{1{,}000\text{ lb/kip}}(10\text{ in.})^{1.5}$$
$$= 21.2\text{ kips}$$

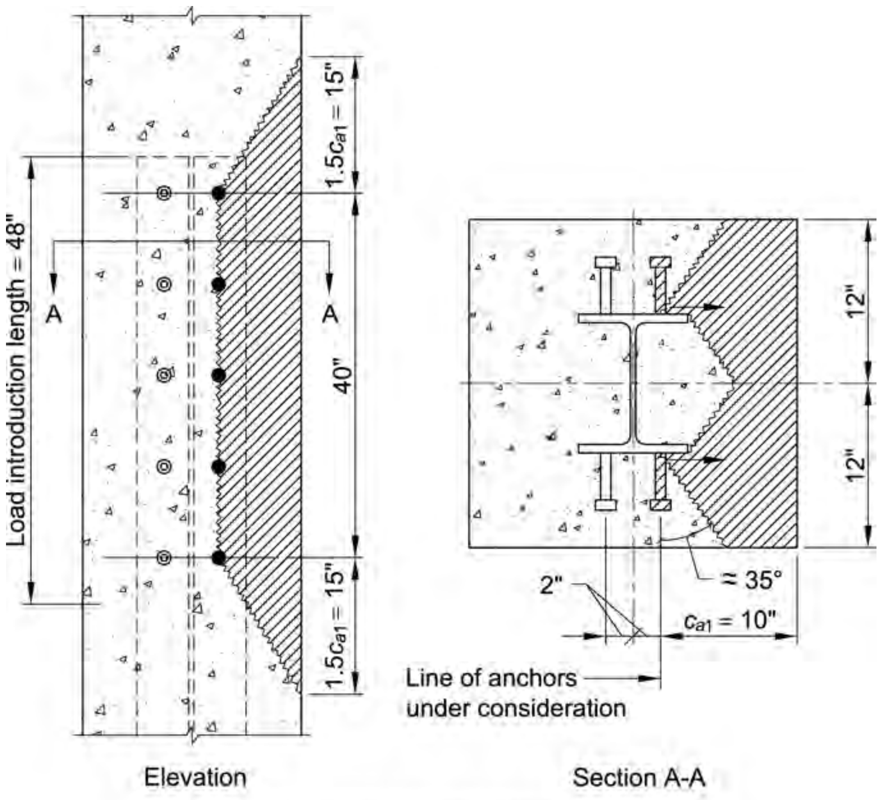
$$V_{cbg} = 2\left[\frac{1{,}680\text{ in.}^2}{450\text{ in.}^2}(1.0)(1.0)(1.4)(1.0)(21.2\text{ kips})\right]$$
$$= 222\text{ kips}$$

$$\phi V_{cbg} = 0.75(222\text{ kips})$$
$$= 167\text{ kips per breakout plane}$$

$$\phi V_{cbg} = (2\text{ breakout planes})(167\text{ kips/plane})$$
$$= 334\text{ kips}$$

$\phi V_{cbg} > V_r' = 304$ kips **o.k.**

Thus, concrete breakout along an edge parallel to the direction of the longitudinal shear transfer is not a controlling limit state, and Equation I8-3 is appropriate for determining available anchor strength.



Note: Reinforcing bars not shown for clarity.

Fig. I.8-5. Concrete breakout check for shear force parallel to an edge.

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Encased beam-column members with reinforcing detailed in accordance with the AISC *Specification* have demonstrated adequate confinement in tests to prevent concrete breakout along a parallel edge from occurring; however, it is still incumbent upon the engineer to review the project-specific detailing used for susceptibility to this limit state.

If concrete breakout was determined to be a controlling limit state, transverse reinforcing ties could be analyzed as anchor reinforcement in accordance with AISC *Specification* Section I8.3a(a), and tie spacing through the load introduction length adjusted as required to prevent breakout. Alternately, the steel headed stud anchors could be relocated to the web of the encased member where breakout is prevented by confinement between the column flanges.

EXAMPLE I.9 ENCASED COMPOSITE MEMBER IN AXIAL COMPRESSION

Given:

Determine if the encased composite member illustrated in Figure I.9-1 is adequate for the indicated dead and live loads.

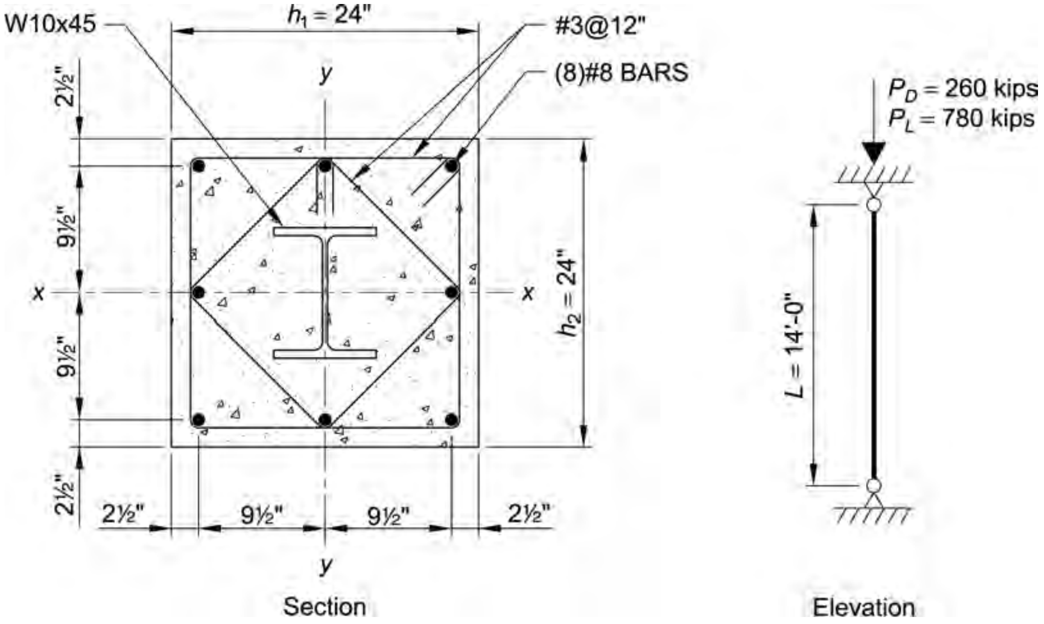

Fig. I.9-1. Encased composite member section and applied loading.

The composite member consists of an ASTM A992 W-shape encased by normal weight (145 lb/ft^3) reinforced concrete having a specified concrete compressive strength, $f_c' = 5$ ksi.

Deformed reinforcing bars conform to ASTM A615 with a minimum yield stress, F_{yr}, of 60 ksi.

Solution:

From AISC *Manual* Table 2-4, the steel material properties are:

ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

From AISC *Manual* Table 1-1, Figure I.9-1, and Design Example I.8, geometric and material properties of the composite section are:

$A_s = 13.3$ in.^{2}
$h_1 = 24$ in.
$A_g = 576$ in.^{2}
$E_c = 3{,}900$ ksi

$b_f = 8.02$ in.
$h_2 = 24$ in.
$A_{sri} = 0.790$ in.^{2}

$t_f = 0.620$ in.
$I_{sx} = 248$ in.^{4}
$A_{sr} = 6.32$ in.^{2}

$d = 10.1$ in.
$I_{sy} = 53.4$ in.^{4}
$A_c = 556$ in.^{2}

The moment of inertia of the reinforcing bars about the elastic neutral axis of the composite section, I_{sr}, is required for composite member design and is calculated as follows:

$d_b = 1$ in. for the diameter of a No. 8 bar

$$\begin{aligned}
I_{sri} &= \frac{\pi d_b^4}{64} \\
&= \frac{\pi(1\text{ in.})^4}{64} \\
&= 0.0491\text{ in.}^4
\end{aligned}$$

$$\begin{aligned}
I_{sr} &= \sum_{i=1}^{n} I_{sri} + \sum_{i=1}^{n} A_{sri} e_i^{\,2} \\
&= 8\left(0.0491\text{ in.}^4\right) + 6\left(0.79\text{ in.}^2\right)(9.50\text{ in.})^2 + 2\left(0.79\text{ in.}^2\right)(0\text{ in.})^2 \\
&= 428\text{ in.}^4
\end{aligned}$$

where

A_{sri} = cross-sectional area of reinforcing bar *i*, in.^{2}
I_{sri} = moment of inertia of reinforcing bar *i* about its elastic neutral axis, in.^{4}
I_{sr} = moment of inertia of the reinforcing bars about the elastic neutral axis of the composite section, in.^{4}
d_b = nominal diameter of reinforcing bar, in.
e_i = eccentricity of reinforcing bar *i* with respect to the elastic neutral axis of the composite section, in.
n = number of reinforcing bars in composite section

Note that the elastic neutral axis for each direction of the section in question is located at the *x-x* and *y-y* axes illustrated in Figure I.9-1, and that the moment of inertia calculated for the longitudinal reinforcement is valid about either axis due to symmetry.

The moment of inertia values for the concrete about each axis are determined as:

$$\begin{aligned}
I_{cx} &= I_{gx} - I_{sx} - I_{srx} \\
&= \frac{(24\text{ in.})^4}{12} - 248\text{ in.}^4 - 428\text{ in.}^4 \\
&= 27{,}000\text{ in.}^4
\end{aligned}$$

$$\begin{aligned}
I_{cy} &= I_{gy} - I_{sy} - I_{sry} \\
&= \frac{(24\text{ in.})^4}{12} - 53.4\text{ in.}^4 - 428\text{ in.}^4 \\
&= 27{,}200\text{ in.}^4
\end{aligned}$$

Classify Section for Local Buckling

In accordance with AISC *Specification* Section I1.2, local buckling effects need not be considered for encased composite members, thus all encased sections are treated as compact sections for strength calculations.

Material and Detailing Limitations

According to the User Note at the end of AISC *Specification* Section I1.1, the intent of the *Specification* is to implement the noncomposite detailing provisions of ACI 318 in conjunction with the composite-specific provisions of *Specification* Chapter I. Detailing provisions may be grouped into material related limits, transverse reinforcement provisions, and longitudinal and structural steel reinforcement provisions as illustrated in the following discussion.

Material limits are provided in AISC *Specification* Sections I1.1(b) and I1.3 as follows:

(1) Concrete strength: $3 \text{ ksi} \le f_c' \le 10 \text{ ksi}$

$f_c' = 5 \text{ ksi}$ **o.k.**

(2) Specified minimum yield stress of structural steel: $F_y \le 75 \text{ ksi}$

$F_y = 50 \text{ ksi}$ **o.k.**

(3) Specified minimum yield stress of reinforcing bars: $F_{yr} \le 75 \text{ ksi}$

$F_{yr} = 60 \text{ ksi}$ **o.k.**

Transverse reinforcement limitations are provided in AISC *Specification* Section I1.1(c), I2.1a(b) and ACI 318 as follows:

(1) Tie size and spacing limitations:

The AISC *Specification* requires that either lateral ties or spirals be used for transverse reinforcement. Where lateral ties are used, a minimum of either No. 3 bars spaced at a maximum of 12 in. on center or No. 4 bars or larger spaced at a maximum of 16 in. on center are required.

No. 3 lateral ties at 12 in. o.c. are provided. **o.k.**

Note that AISC *Specification* Section I1.1(a) specifically excludes the composite column provisions of ACI 318, so it is unnecessary to meet the tie reinforcement provisions of ACI 318 when designing composite columns using the provisions of AISC *Specification* Chapter I.

If spirals are used, the requirements of ACI 318 should be met according to the User Note at the end of AISC *Specification* Section I2.1a.

(2) Additional tie size limitation:

No. 4 ties or larger are required where No. 11 or larger bars are used as longitudinal reinforcement in accordance with ACI 318, Section 9.7.6.4.2.

No. 3 lateral ties are provided for No. 8 longitudinal bars. **o.k.**

(3) Maximum tie spacing should not exceed 0.5 times the least column dimension:

$$s_{max} = 0.5 \min \begin{Bmatrix} h_1 = 24 \text{ in.} \\ h_2 = 24 \text{ in.} \end{Bmatrix}$$

$$= 12.0 \text{ in.}$$

$$s = 12.0 \text{ in.} \le s_{max} \quad \textbf{o.k.}$$

(4) Concrete cover:

ACI 318, Section 20.6.1.3 contains concrete cover requirements. For concrete not exposed to weather or in contact with ground, the required cover for column ties is 1½ in.

$$\begin{aligned} \text{cover} &= 2.5\text{ in.} - \frac{d_b}{2} - \text{diameter of No. 3 tie} \\ &= 2.5\text{ in.} - \tfrac{1}{2}\text{ in.} - \tfrac{3}{8}\text{ in.} \\ &= 1.63\text{ in.} > 1\tfrac{1}{2}\text{ in.} \quad \textbf{o.k.} \end{aligned}$$

(5) Provide ties as required for lateral support of longitudinal bars:

AISC *Specification* Commentary Section I2.1a references ACI 318 for additional transverse tie requirements. In accordance with ACI 318, Section 25.7.2.3 and Figure R25.7.2.3a, ties are required to support longitudinal bars located farther than 6 in. clear on each side from a laterally supported bar. For corner bars, support is typically provided by the main perimeter ties. For intermediate bars, Figure I.9-1 illustrates one method for providing support through the use of a diamond-shaped tie.

Longitudinal and structural steel reinforcement limits are provided in AISC *Specification* Sections I1.1, I2.1 and ACI 318 as follows:

(1) Structural steel minimum reinforcement ratio: $A_s/A_g \geq 0.01$

$$\begin{aligned} \frac{A_s}{A_g} &= \frac{13.3\text{ in.}^2}{576\text{ in.}^2} \geq 0.01 \\ &= 0.0231 > 0.01 \quad \textbf{o.k.} \end{aligned}$$

An explicit maximum reinforcement ratio for the encased steel shape is not provided in the AISC *Specification*; however, a range of 8 to 12% has been noted in the literature to result in economic composite members for the resistance of gravity loads (Leon and Hajjar, 2008).

(2) Minimum longitudinal reinforcement ratio: $A_{sr}/A_g \geq 0.004$

$$\begin{aligned} \frac{A_{sr}}{A_g} &= \frac{6.32\text{ in.}^2}{576\text{ in.}^2} \geq 0.004 \\ &= 0.0110 > 0.004 \quad \textbf{o.k.} \end{aligned}$$

As discussed in AISC *Specification Commentary* Section I2.1a(c), only continuously developed longitudinal reinforcement is included in the minimum reinforcement ratio, so longitudinal restraining bars and other discontinuous longitudinal reinforcement is excluded. Note that this limitation is used in lieu of the minimum ratio provided in ACI 318 as discussed in *Specification* Commentary Section I1.1.

(3) Maximum longitudinal reinforcement ratio: $A_{sr}/A_g \leq 0.08$

$$\begin{aligned} \frac{A_{sr}}{A_g} &= \frac{6.32\text{ in.}^2}{576\text{ in.}^2} \leq 0.08 \\ &= 0.0110 < 0.08 \quad \textbf{o.k.} \end{aligned}$$

This longitudinal reinforcement limitation is provided in ACI 318, Section 10.6.1.1. It is recommended that all longitudinal reinforcement, including discontinuous reinforcement not used in strength calculations, be included in this ratio as it is considered a practical limitation to mitigate congestion of reinforcement. If longitudinal reinforcement is lap spliced as opposed to mechanically coupled, this limit is effectively reduced to 4% in areas away from the splice location.

(4) Minimum number of longitudinal bars:

ACI 318, Section 10.7.3.1, requires a minimum of four longitudinal bars within rectangular or circular members with ties and six bars for columns utilizing spiral ties. The intent for rectangular sections is to provide a minimum of one bar in each corner, so irregular geometries with multiple corners require additional longitudinal bars.

8 bars provided. **o.k.**

(5) Clear spacing between longitudinal bars:

ACI 318 Section 25.2.3 requires a clear distance between bars of 1.5d_b or 1½ in.

$$s_{min} = \max\begin{Bmatrix} 1.5d_b = 1\tfrac{1}{2}\text{ in.} \\ 1\tfrac{1}{2}\text{ in.} \end{Bmatrix}$$
$$= 1\tfrac{1}{2}\text{ in. clear}$$

$$s = 9.50\text{ in.} - 1.00\text{ in.}$$
$$= 8.50\text{ in.} > 1\tfrac{1}{2}\text{ in.} \quad \textbf{o.k.}$$

(6) Clear spacing between longitudinal bars and the steel core:

AISC *Specification* Section I2.1e requires a minimum clear spacing between the steel core and longitudinal reinforcement of 1.5 reinforcing bar diameters, but not less than 1½ in.

$$s_{min} = \max\begin{Bmatrix} 1.5d_b = 1\tfrac{1}{2}\text{ in.} \\ 1\tfrac{1}{2}\text{ in.} \end{Bmatrix}$$
$$= 1\tfrac{1}{2}\text{ in. clear}$$

Closest reinforcing bars to the encased section are the center bars adjacent to each flange:

$$s = \frac{h_2}{2} - \frac{d}{2} - 2.50\text{ in.} - \frac{d_b}{2}$$
$$= \frac{24.0\text{ in.}}{2} - \frac{10.1\text{ in.}}{2} - 2.50\text{ in.} - \frac{1.00\text{ in.}}{2}$$
$$= 3.95\text{ in.} > s_{min} = 1\tfrac{1}{2}\text{ in.} \quad \textbf{o.k.}$$

(7) Concrete cover for longitudinal reinforcement:

ACI 318, Section 20.6.1.3, provides concrete cover requirements for reinforcement. The cover requirements for column ties and primary reinforcement are the same, and the tie cover was previously determined to be acceptable, thus the longitudinal reinforcement cover is acceptable by inspection.

From ASCE/SEI, Chapter 2, the required compressive strength is:

LRFD	ASD
$P_r = P_u$ $= 1.2(260\text{ kips}) + 1.6(780\text{ kips})$ $= 1{,}560\text{ kips}$	$P_r = P_a$ $= 260\text{ kips} + 780\text{ kips}$ $= 1{,}040\text{ kips}$

Available Compressive Strength

The nominal axial compressive strength without consideration of length effects, P_{no}, is determined from AISC *Specification* Section I2.1b as:

$$\begin{aligned} P_{no} &= F_y A_s + F_{ysr} A_{sr} + 0.85 f_c' A_c \\ &= (50\text{ ksi})(13.3\text{ in.}^2) + (60\text{ ksi})(6.32\text{ in.}^2) + 0.85(5\text{ ksi})(556\text{ in.}^2) \\ &= 3{,}410\text{ kips} \end{aligned}$$ (*Spec*. Eq. I2-4)

Because the unbraced length is the same in both the *x-x* and *y-y* directions, the column will buckle about the axis having the smaller effective composite section stiffness, EI_{eff}. Noting the moment of inertia values determined previously for the concrete and reinforcing steel are similar about each axis, the column will buckle about the weak axis of the steel shape by inspection. I_{cy}, I_{sy} and I_{sry} are therefore used for calculation of length effects in accordance with AISC *Specification* Section I2.1b as follows:

$$\begin{aligned} C_1 &= 0.25 + 3\left(\frac{A_s + A_{sr}}{A_g}\right) \le 0.7 \\ &= 0.25 + 3\left(\frac{13.3\text{ in.}^2 + 6.32\text{ in.}^2}{576\text{ in.}^2}\right) \le 0.7 \\ &= 0.352 < 0.7;\ \text{ therefore } C_1 = 0.352 \end{aligned}$$ (*Spec*. Eq. I2-7)

$$\begin{aligned} EI_{eff} &= E_s I_{sy} + E_s I_{sry} + C_1 E_c I_{cy} \\ &= (29{,}000\text{ ksi})(53.4\text{ in.}^4) + (29{,}000\text{ ksi})(428\text{ in.}^4) \\ &\quad + 0.352(3{,}900\text{ ksi})(27{,}200\text{ in.}^4) \\ &= 51{,}300{,}000\text{ kip-in.}^2 \end{aligned}$$ (from *Spec*. Eq. I2-6)

$$\begin{aligned} P_e &= \pi^2 (EI_{eff})/(L_c)^2 \text{, where } L_c = KL \text{ and } K = 1.0 \text{ for a pin-ended member} \\ &= \frac{\pi^2 (51{,}300{,}000\text{ kip-in.}^2)}{\left[(1.0)(14\text{ ft})(12\text{ in./ft})\right]^2} \\ &= 17{,}900\text{ kips} \end{aligned}$$ (*Spec*. Eq. I2-5)

$$\begin{aligned} \frac{P_{no}}{P_e} &= \frac{3{,}410\text{ kips}}{17{,}900\text{ kips}} \\ &= 0.191 < 2.25 \end{aligned}$$

Therefore, use AISC *Specification* Equation I2-2.

$$\begin{aligned} P_n &= P_{no}\left(0.658^{\frac{P_{no}}{P_e}}\right) \\ &= (3{,}410\text{ kips})(0.658)^{0.191} \\ &= 3{,}150\text{ kips} \end{aligned}$$ (*Spec*. Eq. I2-2)

Check adequacy of the composite column for the required axial compressive strength:

LRFD	ASD
$\phi_c = 0.75$ $\phi_c P_n = 0.75(3{,}150 \text{ kips})$ $= 2{,}360 \text{ kips} > 1{,}560 \text{ kips}$ **o.k.**	$\Omega_c = 2.00$ $\dfrac{P_n}{\Omega_c} = \dfrac{3{,}150 \text{ kips}}{2.00}$ $= 1{,}580 \text{ kips} > 1{,}040 \text{ kips}$ **o.k.**

Available Compressive Strength of Composite Section Versus Bare Steel Section

Due to the differences in resistance and safety factors between composite and noncomposite column provisions, it is possible in rare instances to calculate a lower available compressive strength for an encased composite column than one would calculate for the corresponding bare steel section. However, in accordance with AISC *Specification* Section I2.1b, the available compressive strength need not be less than that calculated for the bare steel member in accordance with Chapter E.

From AISC *Manual* Table 4-1a:

LRFD	ASD
$\phi_c P_n = 359 \text{ kips} < 2{,}360 \text{ kips}$	$\dfrac{P_n}{\Omega_c} = 239 \text{ kips} < 1{,}580 \text{ kips}$

Thus, the composite section strength controls and is adequate for the required axial compressive strength as previously demonstrated.

Force Allocation and Load Transfer

Load transfer calculations for external axial forces should be performed in accordance with AISC *Specification* Section I6. The specific application of the load transfer provisions is dependent upon the configuration and detailing of the connecting elements. Expanded treatment of the application of load transfer provisions for encased composite members is provided in Design Example I.8.

Typical Detailing Convention

Designers are directed to AISC Design Guide 6 (Griffis, 1992) for additional discussion and typical details of encased composite columns not explicitly covered in this example.

EXAMPLE I.10 ENCASED COMPOSITE MEMBER IN AXIAL TENSION

Given:

Determine if the encased composite member illustrated in Figure I.10-1 is adequate for the indicated dead load compression and wind load tension. The entire load is applied to the encased steel section.

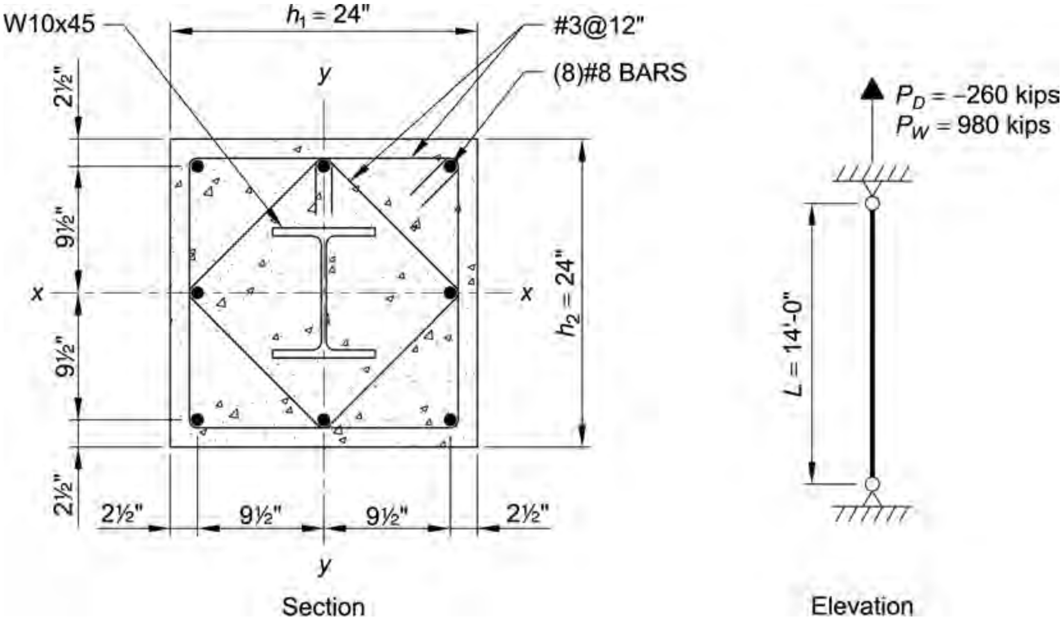

Fig. I.10-1. Encased composite member section and applied loading.

The composite member consists of an ASTM A992 W-shape encased by normal weight (145 lb/ft^3) reinforced concrete having a specified concrete compressive strength, $f_c' = 5$ ksi.

Deformed reinforcing bars conform to ASTM A615 with a minimum yield stress, F_{yr}, of 60 ksi.

Solution:

From AISC *Manual* Table 2-4, the steel material properties are:

ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

From AISC *Manual* Table 1-1 and Figure I.10-1, the relevant properties of the composite section are:

$A_s = 13.3$ in.^{2}
$A_{sr} = 6.32$ in.^{2} (area of eight No. 8 bars)

Material and Detailing Limitations

Refer to Design Example I.9 for a check of material and detailing limitations specified in AISC *Specification* Chapter I for encased composite members.

Taking compression as negative and tension as positive, from ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
Governing uplift load combination $= 0.9\text{D} + 1.0\text{W}$ $P_r = P_u$ $= 0.9(-260 \text{ kips}) + 1.0(980 \text{ kips})$ $= 746 \text{ kips}$	Governing uplift load combination $= 0.6\text{D} + 0.6\text{W}$ $P_r = P_a$ $= 0.6(-260 \text{ kips}) + 0.6(980 \text{ kips})$ $= 432 \text{ kips}$

Available Tensile Strength

Available tensile strength for an encased composite member is determined in accordance with AISC *Specification* Section I2.1c.

$$\begin{aligned} P_n &= F_y A_s + F_{ysr} A_{sr} \\ &= (50 \text{ ksi})(13.3 \text{ in.}^2) + (60 \text{ ksi})(6.32 \text{ in.}^2) \\ &= 1,040 \text{ kips} \end{aligned}$$ (*Spec*. Eq. I2-8)

LRFD	ASD
$\phi_t = 0.90$ $\phi_t P_n = 0.90(1,040 \text{ kips})$ $= 936 \text{ kips} > 746 \text{ kips}$ **o.k.**	$\Omega_t = 1.67$ $\dfrac{P_n}{\Omega_t} = \dfrac{1,040 \text{ kips}}{1.67}$ $= 623 \text{ kips} > 432 \text{ kips}$ **o.k.**

Force Allocation and Load Transfer

In cases where all of the tension is applied to either the reinforcing steel or the encased steel shape, and the available strength of the reinforcing steel or encased steel shape by itself is adequate, no additional load transfer calculations are required.

In cases, such as the one under consideration, where the available strength of both the reinforcing steel and the encased steel shape are needed to provide adequate tension resistance, AISC *Specification* Section I6 can be modified for tensile load transfer requirements by replacing the P_{no} term in Equations I6-1 and I6-2 with the nominal tensile strength, P_n, determined from Equation I2-8.

For external tensile force applied to the encased steel section:

$$V_r' = P_r\left(1 - \frac{F_y A_s}{P_n}\right)$$ (*Spec*. Eq. C-I6-1)

For external tensile force applied to the longitudinal reinforcement of the concrete encasement:

$$V_r' = P_r\left(\frac{F_y A_s}{P_n}\right)$$ (*Spec*. Eq. C-I6-2)

where

P_n = nominal tensile strength of encased composite member from Equation I2-8, kips

P_r = required external tensile force applied to the composite member, kips

Per the problem statement, the entire external force is applied to the encased steel section, thus, AISC *Specification* Equation C-I6-1 is used as follows:

$$
\begin{aligned}
V_r' &= P_r\left[1-\frac{\left(50\text{ ksi}\right)\left(13.3\text{ in.}^2\right)}{1{,}040\text{ kips}}\right] \\
&= 0.361P_r
\end{aligned}
$$

LRFD	ASD
$V_r' = 0.361\left(746\text{ kips}\right)$ $= 269\text{ kips}$	$V_r' = 0.361\left(432\text{ kips}\right)$ $= 156\text{ kips}$

The longitudinal shear force must be transferred between the encased steel shape and longitudinal reinforcing using the force transfer mechanisms of direct bearing or shear connection in accordance with AISC *Specification* Section I6.3 as illustrated in Example I.8.

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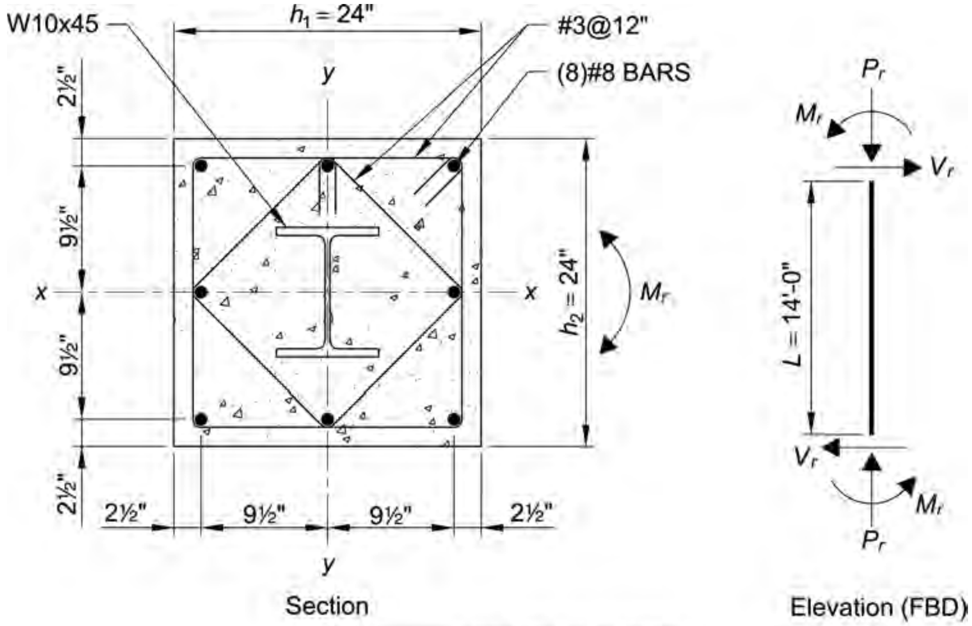
EXAMPLE I.11 ENCASED COMPOSITE MEMBER IN COMBINED AXIAL COMPRESSION, FLEXURE AND SHEAR

Given:

Determine if the encased composite member illustrated in Figure I.11-1 is adequate for the indicated axial forces, shears and moments that have been determined in accordance with the direct analysis method of AISC *Specification* Chapter C for the controlling ASCE/SEI 7 load combinations.



	LRFD	ASD
P_r (kips)	1,170	879
M_r (kip-ft)	670	302
V_r (kips)	95.7	57.4

Fig. I.11-1. Encased composite member section and member forces.

The composite member consists of an ASTM A992 W-shape encased by normal weight (145 lb/ft^3) reinforced concrete having a specified concrete compressive strength, f'_c = 5 ksi.

Deformed reinforcing bars conform to ASTM A615 with a minimum yield stress, F_{yr}, of 60 ksi.

Solution:

From AISC *Manual* Table 2-4, the steel material properties are:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 1-1, Figure I.11-1, and Examples I.8 and I.9, the geometric and material properties of the composite section are:

$A_s = 13.3 \text{ in.}^2$
$A_g = 576 \text{ in.}^2$
$A_c = 556 \text{ in.}^2$
$A_{sr} = 6.32 \text{ in.}^2$
$c = 2\tfrac{1}{2} \text{ in.}$

$d = 10.1 \text{ in.}$
$b_f = 8.02 \text{ in.}$
$t_f = 0.620 \text{ in.}$
$t_w = 0.350 \text{ in.}$
$S_{sx} = 49.1 \text{ in.}^3$

$h_1 = 24 \text{ in.}$
$h_2 = 24 \text{ in.}$
$E_c = 3{,}900 \text{ ksi}$
$Z_{sx} = 54.9 \text{ in.}^3$

$I_{sy} = 53.4 \text{ in.}^4$
$I_{cx} = 27{,}000 \text{ in.}^4$
$I_{cy} = 27{,}200 \text{ in.}^4$
$I_{sr} = 428 \text{ in.}^4$

The area of continuous reinforcing located at the centerline of the composite section, A_{srs}, is determined from Figure I.11-1 as follows:

$$\begin{aligned} A_{srs} &= 2\left(A_{srsi}\right) \\ &= 2\left(0.79 \text{ in.}^2\right) \\ &= 1.58 \text{ in.}^2 \end{aligned}$$

where

A_{srsi} = area of reinforcing bar i at centerline of composite section
$= 0.79 \text{ in.}^2$ for a No. 8 bar

For the section under consideration, A_{srs} is equal about both the x-x and y-y axis.

Classify Section for Local Buckling

In accordance with AISC *Specification* Section I1.2, local buckling effects need not be considered for encased composite members, thus all encased sections are treated as compact sections for strength calculations.

Material and Detailing Limitations

Refer to Design Example I.9 for a check of material and detailing limitations.

Interaction of Axial Force and Flexure

Interaction between flexure and axial forces in composite members is governed by AISC *Specification* Section I5, which permits the use of the methods outlined in Section I1.2.

The strain compatibility method is a generalized approach that allows for the construction of an interaction diagram based upon the same concepts used for reinforced concrete design. Application of the strain compatibility method is required for irregular/nonsymmetrical sections, and its general implementation may be found in reinforced concrete design texts and will not be discussed further here.

Plastic stress distribution methods are discussed in AISC *Specification* Commentary Section I5, which provides four procedures applicable to encased composite members. The first procedure, Method 1, invokes the interaction equations of Section H1. The second procedure, Method 2, involves the construction of a piecewise-linear interaction curve using the plastic strength equations provided in AISC *Manual* Table 6-3a. The third procedure, Method 2—Simplified, is a reduction of the piecewise-linear interaction curve that allows for the use of less conservative interaction equations than those presented in Chapter H. The fourth and final procedure, Method 3, utilizes AISC *Design Guide 6* (Griffis, 1992).

For this design example, three of the available plastic stress distribution procedures are reviewed and compared. Method 3 is not demonstrated as it is not applicable to the section under consideration due to the area of the encased steel section being smaller than the minimum limit of 4% of the gross area of the composite section provided in the earlier *Specification* upon which Design Guide 6 is based.

Method 1—Interaction Equations of Section H1

The most direct and conservative method of assessing interaction effects is through the use of the interaction equations of AISC *Specification* Section H1. Unlike concrete filled HSS shapes, the available compressive and flexural strengths of encased members are not tabulated in the AISC *Manual* due to the large variety of possible combinations. Calculations must therefore be performed explicitly using the provisions of Chapter I.

Available Compressive Strength

The available compressive strength is calculated as illustrated in Example I.9.

LRFD	ASD
$\phi_c P_n = 2{,}360 \text{ kips}$	$\dfrac{P_n}{\Omega_c} = 1{,}580 \text{ kips}$

Nominal Flexural Strength

The applied moment illustrated in Figure I.11-1 is resisted by the flexural strength of the composite section about its strong (*x-x*) axis. The strength of the section in pure flexure is calculated using the equations of AISC *Manual* Table 6-3a for Point B. Note that the calculation of the flexural strength at Point B first requires calculation of the flexural strength at Point D as follows:

$$\begin{aligned} Z_r &= \left(A_{sr} - A_{srs}\right)\left(\frac{h_2}{2} - c\right) \\ &= \left(6.32 \text{ in.}^2 - 1.58 \text{ in.}^2\right)\left(\frac{24 \text{ in.}}{2} - 2\tfrac{1}{2} \text{ in.}\right) \\ &= 45.0 \text{ in.}^3 \end{aligned}$$

$$\begin{aligned} Z_c &= \frac{h_1 h_2^2}{4} - Z_s - Z_r \\ &= \frac{(24 \text{ in.})(24 \text{ in.})^2}{4} - 54.9 \text{ in.}^3 - 45.0 \text{ in.}^3 \\ &= 3{,}360 \text{ in.}^3 \end{aligned}$$

$$\begin{aligned} M_D &= F_y Z_s + F_{yr} Z_r + 0.85 f_c'\left(\frac{Z_c}{2}\right) \\ &= \left[(50 \text{ ksi})\left(54.9 \text{ in.}^3\right) + (60 \text{ ksi})\left(45.0 \text{ in.}^3\right) + 0.85(5 \text{ ksi})\left(\frac{3{,}360 \text{ in.}^3}{2}\right)\right]\left(\frac{1}{12 \text{ in./ft}}\right) \\ &= 1{,}050 \text{ kip-ft} \end{aligned}$$

Assuming h_n is within the flange $\left(\dfrac{d}{2} - t_f < h_n \le \dfrac{d}{2}\right)$:

$$
\begin{aligned}
h_n &= \frac{0.85 f'_c \left(A_c + A_s - d b_f + A_{srs}\right) - 2F_y\left(A_s - d b_f\right) - 2F_{yr} A_{srs}}{2\left[0.85 f'_c \left(h_1 - b_f\right) + 2F_y b_f\right]} \\
&= \frac{\left\{\begin{array}{l} 0.85(5\text{ ksi})\left[556\text{ in.}^2 + 13.3\text{ in.}^2 - (10.1\text{ in.})(8.02\text{ in.}) + 1.58\text{ in.}^2\right] \\ \qquad -2(50\text{ ksi})\left[13.3\text{ in.}^2 - (10.1\text{ in.})(8.02\text{ in.})\right] - 2(60\text{ ksi})\left(1.58\text{ in.}^2\right) \end{array}\right\}}{2\left[0.85(5\text{ ksi})(24\text{ in.} - 8.02\text{ in.}) + 2(50\text{ ksi})(8.02\text{ in.})\right]} \\
&= 4.98\text{ in.}
\end{aligned}
$$

Check assumption:

$$
\left(\frac{10.1\text{ in.}}{2} - 0.620\text{ in.}\right) \le h_n \le \frac{10.1\text{ in.}}{2}
$$

$4.43\text{ in.} < h_n = 4.98\text{ in.} < 5.05\text{ in.}$ assumption **o.k.**

$$
\begin{aligned}
Z_{sn} &= Z_s - b_f\left(\frac{d}{2} - h_n\right)\left(\frac{d}{2} + h_n\right) \\
&= 54.9\text{ in.}^3 - (8.02\text{ in.})\left(\frac{10.1\text{ in.}}{2} - 4.98\text{ in.}\right)\left(\frac{10.1\text{ in.}}{2} + 4.98\text{ in.}\right) \\
&= 49.3\text{ in.}^3
\end{aligned}
$$

$$
\begin{aligned}
Z_{cn} &= h_1 h_n^2 - Z_{sn} \\
&= (24\text{ in.})(4.98\text{ in.})^2 - 49.3\text{ in.}^3 \\
&= 546\text{ in.}^3
\end{aligned}
$$

$$
\begin{aligned}
M_B &= M_D - F_y Z_{sn} - 0.85 f'_c\left(\frac{Z_{cn}}{2}\right) \\
&= \left[12{,}600\text{ kip-in.} - (50\text{ ksi})\left(49.3\text{ in.}^3\right) - 0.85(5\text{ ksi})\left(\frac{546\text{ in.}^3}{2}\right)\right]\left(\frac{1}{12\text{ in./ft}}\right) \\
&= 748\text{ kip-ft}
\end{aligned}
$$

Available Flexural Strength

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(748\text{ kip-ft})$ $= 673\text{ kip-ft}$	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{748\text{ kip-ft}}{1.67}$ $= 448\text{ kip-ft}$

Interaction of Axial Compression and Flexure

LRFD	ASD
$\phi_c P_n = 2{,}360 \text{ kips}$ $\phi_b M_n = 673 \text{ kip-ft}$ $\dfrac{P_r}{P_c} = \dfrac{P_u}{\phi_c P_n}$ $= \dfrac{1{,}170 \text{ kips}}{2{,}360 \text{ kips}}$ $= 0.496 > 0.2$ Therefore, use AISC *Specification* Equation H1-1a. $\dfrac{P_u}{\phi_c P_n} + \dfrac{8}{9}\left(\dfrac{M_u}{\phi_b M_n}\right) \le 1.0$ (from *Spec*. Eq. H1-1a) $0.496 + \dfrac{8}{9}\left(\dfrac{670 \text{ kip-ft}}{673 \text{ kip-ft}}\right) \le 1.0$ $1.38 > 1.0$ **n.g.**	$P_n / \Omega_c = 1{,}580 \text{ kips}$ $M_n / \Omega_c = 448 \text{ kip-ft}$ $\dfrac{P_r}{P_c} = \dfrac{P_a}{P_n / \Omega_c}$ $= \dfrac{879 \text{ kips}}{1{,}580 \text{ kips}}$ $= 0.556 > 0.2$ Therefore, use AISC *Specification* Equation H1-1a. $\dfrac{P_a}{P_n / \Omega_c} + \dfrac{8}{9}\left(\dfrac{M_a}{M_n / \Omega_b}\right) \le 1.0$ (from *Spec*. Eq. H1-1a) $0.556 + \dfrac{8}{9}\left(\dfrac{302 \text{ kip-ft}}{448 \text{ kip-ft}}\right) \le 1.0$ $1.16 > 1.0$ **n.g.**

Method 1 indicates that the section is inadequate for the applied loads. The designer can elect to choose a new section that passes the interaction check or re-analyze the current section using a less conservative design method such as Method 2. The use of Method 2 is illustrated in the following section.

Method 2—Interaction Curves from the Plastic Stress Distribution Model

The procedure for creating an interaction curve using the plastic stress distribution model is illustrated graphically in AISC *Specification* Commentary Figure C-I5.2, and repeated here.

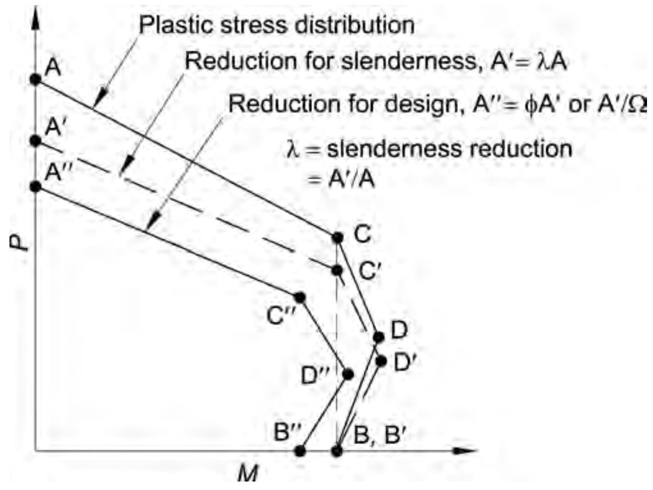


Fig. C-I5.2. Interaction diagram for composite beam-columns—Method 2.

Referencing Figure C.I5.2, the nominal strength interaction surface A, B, C, D is first determined using the equations of AISC *Manual* Table 6-3a. This curve is representative of the short column member strength without consideration of length effects. A slenderness reduction factor, λ, is then calculated and applied to each point to create surface $\text{A}', \text{B}', \text{C}', \text{D}'$. The appropriate resistance or safety factors are then applied to create the design surface $\text{A}'', \text{B}'', \text{C}'', \text{D}''$. Finally, the required axial and flexural strengths from the applicable load combinations of ASCE/SEI 7 are plotted on the design surface. The member is then deemed acceptable for the applied loading if all points fall within the design surface. These steps are illustrated in detail by the following calculations.

Step 1: Construct nominal strength interaction surface A, B, C, D without length effects

Using the equations provided in Figure I-1a for bending about the *x*-*x* axis yields:

Point A (pure axial compression):

$$\begin{aligned} P_A &= F_y A_s + F_{yr} A_{sr} + 0.85 f_c' A_c \\ &= (50\text{ ksi})(13.3\text{ in.}^2) + (60\text{ ksi})(6.32\text{ in.}^2) + 0.85(5\text{ ksi})(556\text{ in.}^2) \\ &= 3{,}410\text{ kips} \end{aligned}$$

$$M_A = 0\text{ kip-ft}$$

Point D (maximum nominal moment strength):

$$\begin{aligned} P_D &= \frac{0.85 f_c' A_c}{2} \\ &= \frac{0.85(5\text{ ksi})(556\text{ in.}^2)}{2} \\ &= 1{,}180\text{ kips} \end{aligned}$$

Calculation of M_D was demonstrated previously in Method 1.

$$M_D = 1{,}050\text{ kip-ft}$$

Point B (pure flexure):

$$P_B = 0\text{ kips}$$

Calculation of M_B was demonstrated previously in Method 1.

$$M_B = 748\text{ kip-ft}$$

Point C (intermediate point):

$$\begin{aligned} P_C &= 0.85 f_c' A_c \\ &= 0.85(5\text{ ksi})(556\text{ in.}^2) \\ &= 2{,}360\text{ kips} \end{aligned}$$

$$\begin{aligned} M_C &= M_B \\ &= 748\text{ kip-ft} \end{aligned}$$

The calculated points are plotted to construct the nominal strength interaction surface without length effects as depicted in Figure I.11-2.

Step 2: Construct nominal strength interaction surface A′, B′, C′, D′ with length effects

The slenderness reduction factor, λ, is calculated for Point A using AISC *Specification* Section I2.1 in accordance with AISC *Specification* Commentary Section I5.

Because the unbraced length is the same in both the *x-x* and *y-y* directions, the column will buckle about the axis having the smaller effective composite section stiffness, EI_{eff}. Noting the moment of inertia values for the concrete and reinforcing steel are similar about each axis, the column will buckle about the weak axis of the steel shape by inspection. I_{cy}, I_{sy} and I_{sry} are therefore used for calculation of length effects in accordance with AISC *Specification* Section I2.1b.

$$\begin{aligned} P_{no} &= P_A \\ &= 3{,}410 \text{ kips} \end{aligned}$$

$$\begin{aligned} C_1 &= 0.25 + 3\left(\frac{A_s + A_{sr}}{A_g}\right) \le 0.7 \qquad \text{(\textit{Spec}. Eq. I2-7)} \\ &= 0.25 + 3\left(\frac{13.3 \text{ in.}^2 + 6.32 \text{ in.}^2}{576 \text{ in.}^2}\right) \le 0.7 \\ &= 0.352 < 0.7; \text{ therefore } C_1 = 0.352. \end{aligned}$$

$$\begin{aligned} EI_{eff} &= E_s I_{sy} + E_s I_{sry} + C_1 E_c I_{cy} \qquad \text{(from \textit{Spec}. Eq. I2-6)} \\ &= (29{,}000 \text{ ksi})\left(53.4 \text{ in.}^4\right) + (29{,}000 \text{ ksi})\left(428 \text{ in.}^4\right) + 0.352(3{,}900 \text{ ksi})\left(27{,}200 \text{ in.}^4\right) \\ &= 51{,}300{,}000 \text{ kip-in.}^2 \end{aligned}$$

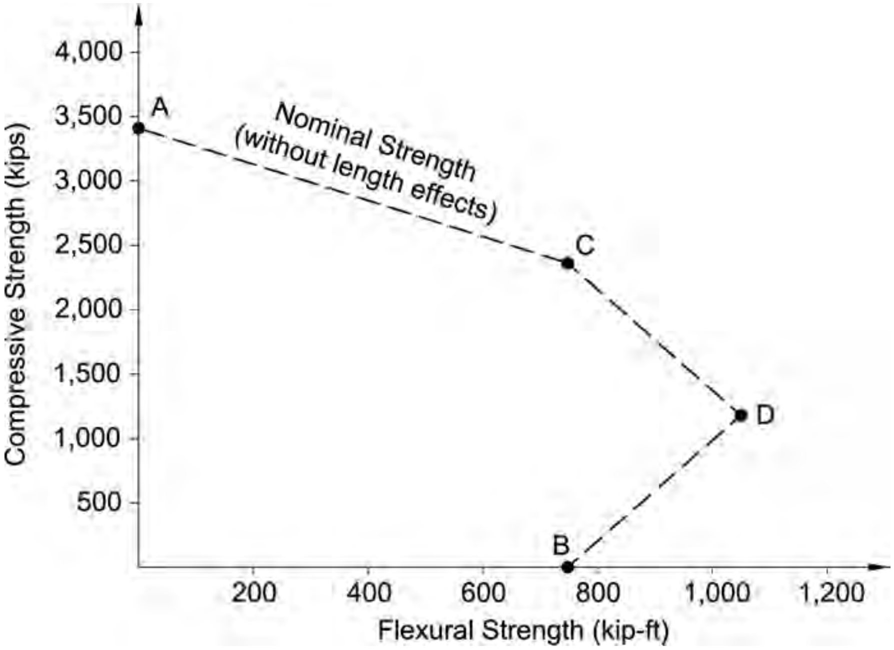


Fig. I.11-2. Nominal strength interaction surface without length effects.

$$P_e = \pi^2\left(EI_{eff}\right)/\left(L_c\right)^2 \text{, where } L_c = KL \text{ and } K = 1.0 \qquad (Spec.\ \text{Eq. I2-5})$$

in accordance with the direct analysis method

$$= \frac{\pi^2\left(51{,}300{,}000 \text{ kip-in.}^2\right)}{\left[(1.0)(14 \text{ ft})(12 \text{ in./ft})\right]^2}$$

$$= 17{,}900 \text{ kips}$$

$$\frac{P_{no}}{P_e} = \frac{3{,}410 \text{ kips}}{17{,}900 \text{ kips}}$$

$$= 0.191 < 2.25$$

Therefore, use AISC *Specification* Equation I2-2.

$$P_n = P_{no}\left(0.658^{\frac{P_{no}}{P_e}}\right) \qquad (Spec.\ \text{Eq. I2-2})$$

$$= (3{,}410 \text{ kips})(0.658)^{0.191}$$

$$= 3{,}150 \text{ kips}$$

$$\lambda = \frac{P_n}{P_{no}}$$

$$= \frac{3{,}150 \text{ kips}}{3{,}410 \text{ kips}}$$

$$= 0.924$$

In accordance with AISC *Specification* Commentary Section I5, the same slenderness reduction is applied to each of the remaining points on the interaction surface as follows:

$$P_{A'} = \lambda P_A$$

$$= 0.924(3{,}410 \text{ kips})$$

$$= 3{,}150 \text{ kips}$$

$$P_{B'} = \lambda P_B$$

$$= 0.924(0 \text{ kip})$$

$$= 0 \text{ kip}$$

$$P_{C'} = \lambda P_C$$

$$= 0.924(2{,}360 \text{ kips})$$

$$= 2{,}180 \text{ kips}$$

$$P_{D'} = \lambda P_D$$

$$= 0.924(1{,}180 \text{ kips})$$

$$= 1{,}090 \text{ kips}$$

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The modified axial strength values are plotted with the flexural strength values previously calculated to construct the nominal strength interaction surface including length effects. These values are superimposed on the nominal strength surface not including length effects for comparison purposes in Figure I.11-3.

The consideration of length effects results in a vertical reduction of the nominal strength curve as illustrated by Figure I.11-3. This vertical movement creates an unsafe zone within the shaded area of the figure where flexural capacities of the nominal strength (with length effects) curve exceed the section capacity. Application of resistance or safety factors reduces this unsafe zone as illustrated in the following step; however, designers should be cognizant of the potential for unsafe designs with loads approaching the predicted flexural capacity of the section. Alternately, the use of Method 2—Simplified eliminates this possibility altogether.

Step 3: Construct design interaction surface A′′, B′′, C′′, D′′ and verify member adequacy

The final step in the Method 2 procedure is to reduce the interaction surface for design using the appropriate resistance or safety factors.

The available compressive and flexural strengths are determined as follows:

LRFD	ASD
$\phi_c = 0.75$	$\Omega_c = 2.00$
$P_{X''} = \phi_c P_{X'}$, where X = A, B, C or D	$P_{X''} = \dfrac{P_{X'}}{\Omega_c}$, where X = A, B, C or D
$P_{A''} = 0.75(3{,}150\text{ kips})$ $= 2{,}360\text{ kips}$	$P_{A''} = 3{,}150\text{ kips} / 2.00$ $= 1{,}580\text{ kips}$
$P_{B''} = 0.75(0\text{ kip})$ $= 0\text{ kip}$	$P_{B''} = 0\text{ kip} / 2.00$ $= 0\text{ kip}$
$P_{C''} = 0.75(2{,}180\text{ kips})$ $= 1{,}640\text{ kips}$	$P_{C''} = 2{,}180\text{ kips} / 2.00$ $= 1{,}090\text{ kips}$
$P_{D''} = 0.75(1{,}090\text{ kips})$ $= 818\text{ kips}$	$P_{D''} = 1{,}090\text{ kips} / 2.00$ $= 545\text{ kips}$

LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
$M_{X''} = \phi_b M_X$, where X = A, B, C or D	$M_{X''} = \dfrac{M_X}{\Omega_b}$, where X = A, B, C or D
$M_{A''} = 0.90(0 \text{ kip-ft})$ $= 0 \text{ kip-ft}$	$M_{A''} = 0 \text{ kip-ft} / 1.67$ $= 0 \text{ kip-ft}$
$M_{B''} = 0.90(748 \text{ kip-ft})$ $= 673 \text{ kip-ft}$	$M_{B''} = 748 \text{ kip-ft} / 1.67$ $= 448 \text{ kip-ft}$
$M_{C''} = 0.90(748 \text{ kip-ft})$ $= 673 \text{ kip-ft}$	$M_{C''} = 748 \text{ kip-ft} / 1.67$ $= 448 \text{ kip-ft}$
$M_{D''} = 0.90(1{,}050 \text{ kip-ft})$ $= 945 \text{ kip-ft}$	$M_{D''} = 1{,}050 \text{ kip-ft} / 1.67$ $= 629 \text{ kip-ft}$

The available strength values for each design method can now be plotted. These values are superimposed on the nominal strength surfaces (with and without length effects) previously calculated for comparison purposes in Figure I.11-4.

By plotting the required axial and flexural strength values on the available strength surfaces indicated in Figure I.11-4, it can be seen that both ASD (M_a,P_a) and LRFD (M_u,P_u) points lie within their respective design surfaces. The member in question is therefore adequate for the applied loads.

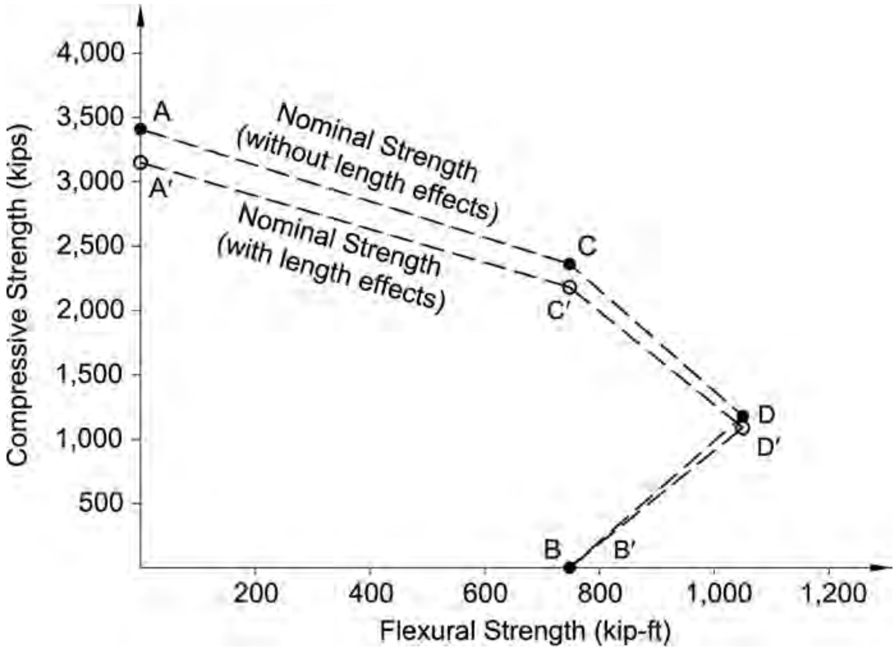

Fig. I.11-3. Nominal strength interaction surfaces (with and without length effects).

As discussed previously in Step 2 as well as in AISC *Specification* Commentary Section I5, when reducing the flexural strength of Point D for length effects and resistance or safety factors, an unsafe situation could result whereby additional flexural strength is permitted at a lower axial compressive strength than predicted by the cross section strength of the member. This effect is highlighted by the magnified portion of Figure I.11-4, where LRFD design point D′′ closely approaches the nominal strength curve. Designs falling outside the nominal strength curve are unsafe and not permitted.

Method 2—Simplified

The unsafe zone discussed in the previous section for Method 2 is avoided in the Method 2—Simplified procedure by the removal of Point D′′ from the Method 2 interaction surface leaving only points A′′, B′′ and C′′ as illustrated in Figure I.11-5. Reducing the number of interaction points also allows for a bilinear interaction check defined by AISC *Specification* Commentary Equations C-I5-1a and C-I5-1b to be performed.

Using the available strength values previously calculated in conjunction with the Commentary equations, interaction ratios are determined as follows:

LRFD	ASD
$P_r = P_u$ $= 1{,}170 \text{ kips} < P_{C''} = 1{,}640 \text{ kips}$ Therefore, use AISC *Specification* Commentary Equation C-I5-1a. $\dfrac{M_r}{M_C} = \dfrac{M_u}{M_{C''}} \le 1.0$ (from *Spec*. Comm. Eq. C-I5-1a) $\dfrac{670 \text{ kip-ft}}{673 \text{ kip-ft}} \le 1.0$ $1.0 = 1.0$ **o.k.**	$P_r = P_a$ $= 879 \text{ kips} < P_{C''} = 1{,}090 \text{ kips}$ Therefore, use AISC *Specification* Commentary Equation C-I5-1a. $\dfrac{M_r}{M_C} = \dfrac{M_a}{M_{C''}} \le 1.0$ (from *Spec*. Comm. Eq. C-I5-1a) $\dfrac{302 \text{ kip-ft}}{448 \text{ kip-ft}} \le 1.0$ $0.67 < 1.0$ **o.k.**

Thus, the member is adequate for the applied loads.

Comparison of Methods

The composite member was found to be inadequate using Method 1—Chapter H interaction equations, but was found to be adequate using both Method 2 and Method 2—Simplified procedures. A comparison between the methods is most easily made by overlaying the design curves from each method as illustrated in Figure I.11-6 for LRFD design.

From Figure I.11-6, the conservative nature of the Chapter H interaction equations can be seen. Method 2 provides the highest available strength; however, the Method 2—Simplified procedure also provides a good representation of the design curve. The procedure in Figure I-1 for calculating the flexural strength of Point C′′ first requires the calculation of the flexural strength for Point D′′. The design effort required for the Method 2—Simplified procedure, which utilizes Point C′′, is therefore not greatly reduced from Method 2.

Available Shear Strength

According to AISC *Specification* Section I4.1, there are three acceptable options for determining the available shear strength of an encased composite member:

(1) Option 1—Available shear strength of the steel section alone in accordance with AISC *Specification* Chapter G.

(2) Option 2—Available shear strength of the reinforced concrete portion alone per ACI 318.

(3) Option 3—Available shear strength of the steel section, in addition to the reinforcing steel ignoring the contribution of the concrete.

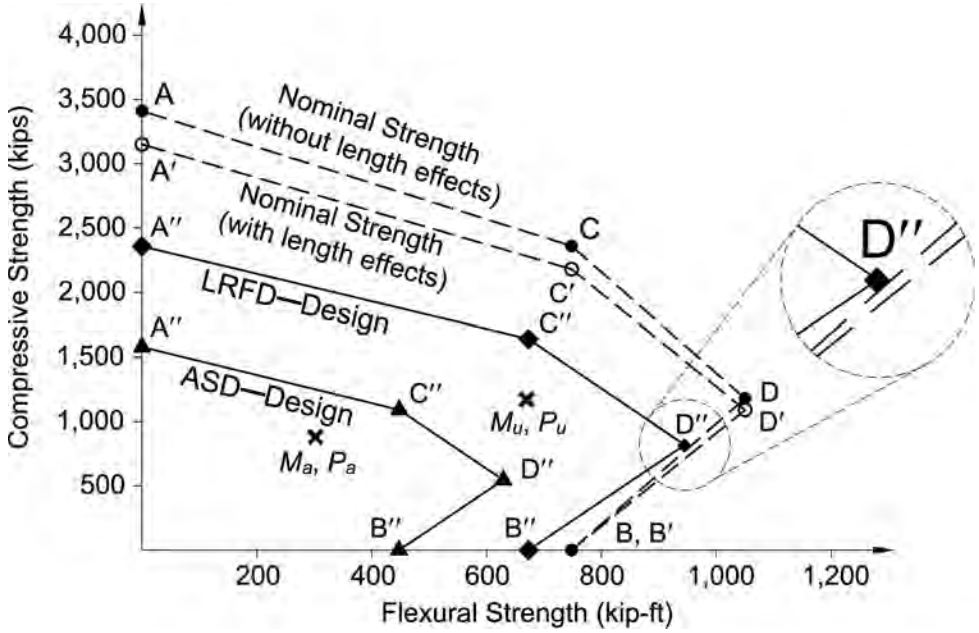

Fig. I.11-4. Available and nominal interaction surfaces.

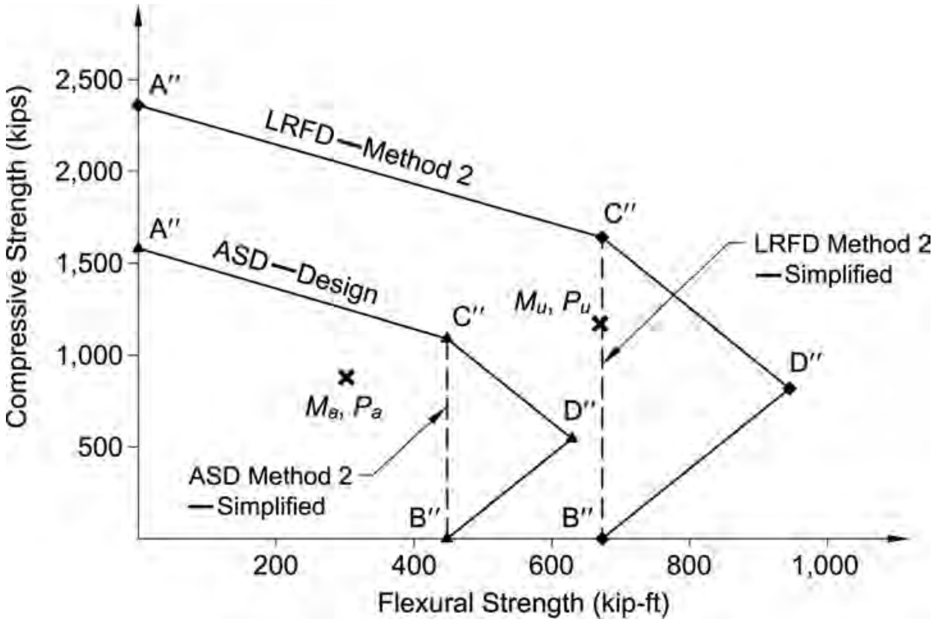

Fig. I.11-5. Comparison of Method 2 and Method 2 —Simplified.

Option 1—Available Shear Strength of Steel Section

A W10×45 member meets the criteria of AISC *Specification* Section G2.1(a) according to the User Note at the end of the section. As demonstrated in Design Example I.9, No. 3 ties at 12 in. on center as illustrated in Figure I.11-1 satisfy the minimum detailing requirements of the *Specification*. The nominal shear strength may therefore be determined as:

$$C_{v1} = 1.0$$ (*Spec*. Eq. G2-2)

$$\begin{aligned} A_w &= dt_w \\ &= (10.1\text{ in.})(0.350\text{ in.}) \\ &= 3.54\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} V_n &= 0.6F_y A_w C_{v1} \\ &= 0.6(50\text{ ksi})(3.54\text{ in.}^2)(1.0) \\ &= 106\text{ kips} \end{aligned}$$ (*Spec*. Eq. G2-1)

The available shear strength of the steel section is:

LRFD	ASD
$\phi_v = 1.00$ $\phi_v V_n = 1.00(106\text{ kips})$ $= 106\text{ kips} > 95.7\text{ kips}$ **o.k.**	$\Omega_v = 1.50$ $\dfrac{V_n}{\Omega_v} = \dfrac{106\text{ kips}}{1.50}$ $= 70.7\text{ kips} > 57.4\text{ kips}$ **o.k.**

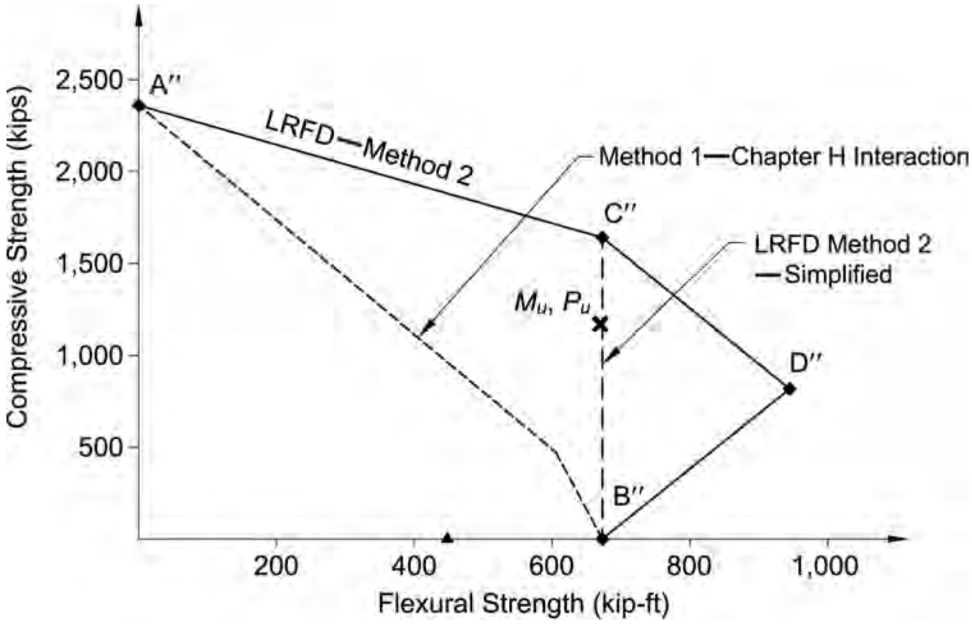

Fig. I.11-6. Comparison of interaction methods (LRFD).

Option 2—Available Shear Strength of the Reinforced Concrete (Concrete and Transverse Steel Reinforcement)

The available shear strength of the steel section alone has been shown to be sufficient; however, the amount of transverse reinforcement required for shear resistance in accordance with AISC *Specification* Section I4.1(b) will be determined for demonstration purposes.

Tie Requirements for Shear Resistance

The nominal concrete shear strength is:

$$V_c = 2\lambda\sqrt{f_c'}\,b_w d \qquad \text{(ACI 318, Eq. 22.5.5.1)}$$

where

$\lambda = 1.0$ for normal weight concrete from ACI 318, Table 19.2.4.2

$b_w = h_1$

$d =$ distance from extreme compression fiber to centroid of longitudinal tension reinforcement

$= 24\text{ in.} - 2\tfrac{1}{2}\text{ in.}$

$= 21.5\text{ in.}$

$$V_c = 2(1.0)\sqrt{5{,}000\text{ psi}}\,(24\text{ in.})(21.5\text{ in.})\left(\frac{1\text{ kip}}{1{,}000\text{ lb}}\right)$$
$$= 73.0\text{ kips}$$

The tie requirements for shear resistance are determined from ACI 318 Chapter 22 and AISC *Specification* Section I4.1(b), as follows:

LRFD	ASD
$\phi_v = 0.75$	$\Omega_v = 2.00$
$\dfrac{A_v}{s} = \dfrac{V_u - \phi_v V_c}{\phi_v f_{yr} d}$ (from ACI 318, Eq. R22.5.10.5) $= \dfrac{95.7\text{ kips} - 0.75(73.0\text{ kips})}{0.75(60\text{ ksi})(21.5\text{ in.})}$ $= 0.0423\text{ in.}$	$\dfrac{A_v}{s} = \dfrac{V_a - (V_c/\Omega_v)}{f_{yr} d/\Omega_v}$ (from ACI 318, Eq. R22.5.10.5) $= \dfrac{57.4\text{ kips} - \left(\dfrac{73.0\text{ kips}}{2.00}\right)}{\dfrac{(60\text{ ksi})(21.5\text{ in.})}{2.00}}$ $= 0.0324\text{ in.}$
Using two legs of No. 3 ties with $A_v = 0.11\text{ in.}^2$ from ACI 318, Appendix A:	Using two legs of No. 3 ties with $A_v = 0.11\text{ in.}^2$ from ACI 318, Appendix A:
$\dfrac{2\left(0.11\text{ in.}^2\right)}{s} = 0.0423\text{ in.}$ $s = 5.20\text{ in.}$	$\dfrac{2\left(0.11\text{ in.}^2\right)}{s} = 0.0324\text{ in.}$ $s = 6.79\text{ in.}$
Using two legs of the No. 4 ties with $A_v = 0.20\text{ in.}^2$:	Using two legs of the No. 4 ties with $A_v = 0.20\text{ in.}^2$:
$\dfrac{2\left(0.20\text{ in.}^2\right)}{s} = 0.0423\text{ in.}$ $s = 9.46\text{ in.}$	$\dfrac{2\left(0.20\text{ in.}^2\right)}{s} = 0.0324\text{ in.}$ $s = 12.3\text{ in.}$

LRFD	ASD
From ACI 318, Section 9.7.6.2.2, the maximum spacing is: $s_{max} = \frac{d}{2}$ $= \frac{21.5\text{ in.}}{2}$ $= 10.8\text{ in.}$ Use No. 3 ties at 5 in. o.c. or No. 4 ties at 9 in. o.c.	From ACI 318, Section 9.7.6.2.2, the maximum spacing is: $s_{max} = \frac{d}{2}$ $= \frac{21.5\text{ in.}}{2}$ $= 10.8\text{ in.}$ Use No. 3 ties at 6 in. o.c. or No. 4 ties at 10 in. o.c.

Minimum Reinforcing Limits

Check that the minimum shear reinforcement is provided as required by ACI 318, Section 9.6.3.3.

$$\frac{A_{v,min}}{s} = 0.75\sqrt{f_c'}\left(\frac{b_w}{f_{yr}}\right) \geq \frac{50b_w}{f_{yr}} \qquad \text{(ACI 318, Table 9.6.3.3)}$$

$$= \frac{0.75\sqrt{5{,}000\text{ psi}}\,(24\text{ in.})}{60{,}000\text{ psi}} \geq \frac{50(24\text{ in.})}{60{,}000\text{ psi}}$$

$$= 0.0212\text{ in.} > 0.0200\text{ in.}$$

LRFD	ASD
$\frac{A_v}{s} = 0.0423\text{ in.} > 0.0212\text{ in.}$ **o.k.**	$\frac{A_v}{s} = 0.0324\text{ in.} > 0.0212\text{ in.}$ **o.k.**

Maximum Reinforcing Limits

From ACI 318, Section 9.7.6.2.2, maximum stirrup spacing is reduced to *d*/4 if $V_s \geq 4\sqrt{f_c'}b_w d$. If No. 4 ties at 9 in. on center are selected:

$$V_s = \frac{A_v f_{yr} d}{s} \qquad \text{(ACI 318, Eq. 22.5.10.5.3)}$$

$$= \frac{2\left(0.20\text{ in.}^2\right)(60\text{ ksi})(21.5\text{ in.})}{9\text{ in.}}$$

$$= 57.3\text{ kips}$$

$$V_{s,max} = 4\sqrt{f_c'}b_w d$$

$$= 4\sqrt{5{,}000\text{ psi}}\,(24\text{ in.})(21.5\text{ in.})\left(\frac{1\text{ kip}}{1{,}000\text{ lb}}\right)$$

$$= 146\text{ kips} > 57.3\text{ kips}$$

Therefore, the stirrup spacing is acceptable.

Option 3—Determine Available Shear Strength of the Steel Section plus Reinforcing Steel

The third procedure combines the shear strength of the reinforcing steel with that of the encased steel section, ignoring the contribution of the concrete. AISC *Specification* Section I4.1(c) provides a combined resistance and safety factor for this procedure. Note that the combined resistance and safety factor takes precedence over the factors in Chapter G used for the encased steel section alone in Option 1. The amount of transverse reinforcement required for shear resistance is determined as follows:

Tie Requirements for Shear Resistance

The nominal shear strength of the encased steel section was previously determined to be:

$$V_{n,steel} = 106\text{ kips}$$

The tie requirements for shear resistance are determined from ACI 318, Chapter 22, and AISC *Specification* Section I4.1(c), as follows:

LRFD	ASD
$\phi_v = 0.75$ $\dfrac{A_v}{s} = \dfrac{V_u - \phi_v V_{n,steel}}{\phi_v f_{yr} d}$ $= \dfrac{95.7\text{ kips} - 0.75(106\text{ kips})}{0.75(60\text{ ksi})(21.5\text{ in.})}$ $= 0.0167\text{ in.}$	$\Omega_v = 2.00$ $\dfrac{A_v}{s} = \dfrac{V_a - (V_{n,steel}/\Omega_v)}{f_{yr} d/\Omega_v}$ $= \dfrac{57.4\text{ kips} - (106\text{ kips}/2.00)}{\left[\dfrac{(60\text{ ksi})(21.5\text{ in.})}{2.00}\right]}$ $= 0.00682\text{ in.}$

As determined in Option 2, the minimum value of $A_v/s = 0.0212$, and the maximum tie spacing for shear resistance is 10.8 in. Using two legs of No. 3 ties for A_v:

$$\frac{2(0.11\text{ in.}^2)}{s} = 0.0212\text{ in.}$$

$$s = 10.4\text{ in.} < s_{max} = 10.8\text{ in.}$$

Use No. 3 ties at 10 in. o.c.

Summary and Comparison of Available Shear Strength Calculations

The use of the steel section alone is the most expedient method for calculating available shear strength and allows the use of a tie spacing which may be greater than that required for shear resistance by ACI 318. Where the strength of the steel section alone is not adequate, Option 3 will generally result in reduced tie reinforcement requirements as compared to Option 2.

Force Allocation and Load Transfer

Load transfer calculations should be performed in accordance with AISC *Specification* Section I6. The specific application of the load transfer provisions is dependent upon the configuration and detailing of the connecting elements. Expanded treatment of the application of load transfer provisions for encased composite members is provided in Design Example I.8 and AISC Design Guide 6.

EXAMPLE I.12 STEEL ANCHORS IN COMPOSITE COMPONENTS

Given:

Select an appropriate ¾-in.-diameter, Type B steel headed stud anchor to resist the dead and live loads indicated in Figure I.12-1. The anchor is part of a composite system that may be designed using the steel anchor in composite components provisions of AISC *Specification* Section I8.3.

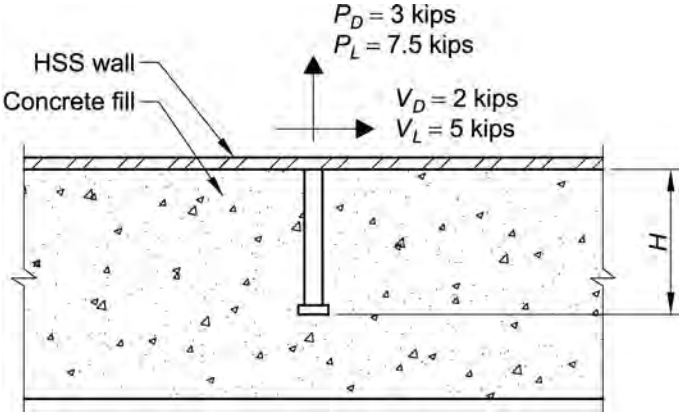

Fig. I.12-1. Steel headed stud anchor and applied loading.

The steel headed stud anchor is encased by normal weight (145 lb/ft^3) reinforced concrete having a specified concrete compressive strength, $f_c' = 5$ ksi. In accordance with AISC *Manual* Part 2, headed stud anchors shall be in accordance with AWS D1.1 with a specified minimum tensile stress, F_u, of 65 ksi.

The anchor is located away from edges such that concrete breakout in shear is not a viable limit state, and the nearest anchor is located 24 in. away. The concrete is considered to be uncracked.

Solution:

Minimum Anchor Length

AISC *Specification* Section I8.3 provides minimum length to shank diameter ratios for anchors subjected to shear, tension, and interaction of shear and tension in both normal weight and lightweight concrete. These ratios are also summarized in the User Note provided within Section I8.3. For normal weight concrete subject to shear and tension, $h / d_{sa} \geq 8$, thus:

$$\begin{aligned} h &\geq 8d_{sa} \\ &\geq 8(\text{¾ in.}) \\ &\geq 6.00 \text{ in.} \end{aligned}$$

This length is measured from the base of the steel headed stud anchor to the top of the head after installation. From anchor manufacturer's data, a standard stock length of 6³⁄₁₆ in. is selected. Using a ³⁄₁₆-in. length reduction to account for burn off during installation yields a final installed length of 6.00 in.

$$6.00 \text{ in.} = 6.00 \text{ in.} \quad \textbf{o.k.}$$

Select a ¾-in.-diameter × 6³⁄₁₆-in.-long headed stud anchor.

Required Shear and Tensile Strength

From ASCE/SEI 7, Chapter 2, the required shear and tensile strengths are:

LRFD	ASD
Governing load combination for interaction = 1.2D + 1.6L	Governing load combination for interaction = D + L
$Q_{uv} = 1.2(2\text{ kips}) + 1.6(5\text{ kips})$ $= 10.4\text{ kips (shear)}$	$Q_{av} = 2\text{ kips} + 5\text{ kips}$ $= 7.00\text{ kips (shear)}$
$Q_{ut} = 1.2(3\text{ kips}) + 1.6(7.5\text{ kips})$ $= 15.6\text{ kips (tension)}$	$Q_{at} = 3\text{ kips} + 7.5\text{ kips}$ $= 10.5\text{ kips (tension)}$

Available Shear Strength

Per the problem statement, concrete breakout is not considered to be an applicable limit state. AISC Equation I8-3 may therefore be used to determine the available shear strength of the steel headed stud anchor as follows:

$$Q_{nv} = F_u A_{sa} \qquad \text{(}\textit{Spec}\text{. Eq. I8-3)}$$

where

A_{sa} = cross-sectional area of steel headed stud anchor

$$= \frac{\pi(\text{¾ in.})^2}{4}$$

$$= 0.442\text{ in.}^2$$

$$Q_{nv} = (65\text{ ksi})(0.442\text{ in.}^2)$$
$$= 28.7\text{ kips}$$

LRFD	ASD
$\phi_v = 0.65$	$\Omega_v = 2.31$
$\phi_v Q_{nv} = 0.65(28.7\text{ kips})$ $= 18.7\text{ kips}$	$\dfrac{Q_{nv}}{\Omega_v} = \dfrac{28.7\text{ kips}}{2.31}$ $= 12.4\text{ kips}$

Alternately, available shear strengths can be selected directly from Table I.12-1 located at the end of this example.

Available Tensile Strength

The nominal tensile strength of a steel headed stud anchor is determined using AISC *Specification* Equation I8-4 provided the edge and spacing limitations of AISC *Specification* Section I8.3b are met as follows:

(1) Minimum distance from centerline of anchor to free edge: $1.5h = 1.5(6.00\text{ in.}) = 9.00\text{ in.}$

There are no free edges, therefore this limitation does not apply.

(2) Minimum distance between centerlines of adjacent anchors: $3h = 3(6.00\text{ in.}) = 18.0\text{ in.}$

$18.0\text{ in.} < 24\text{ in.}$ **o.k.**

Equation I8-4 may therefore be used as follows:

$$\begin{aligned} Q_{nt} &= F_u A_{sa} \\ &= (65\text{ ksi})(0.442\text{ in.}^2) \\ &= 28.7\text{ kips} \end{aligned} \qquad \text{(\textit{Spec}. Eq. I8-4)}$$

LRFD	ASD
$\phi_t = 0.75$ $\phi_t Q_{nt} = 0.75(28.7\text{ kips})$ $= 21.5\text{ kips}$	$\Omega_t = 2.00$ $\dfrac{Q_{nt}}{\Omega_t} = \dfrac{28.7\text{ kips}}{2.00}$ $= 14.4\text{ kips}$

Alternately, available tensile strengths can be selected directly from Table I.12-1 located at the end of this example.

Interaction of Shear and Tension

The detailing limits on edge distances and spacing imposed by AISC *Specification* Section I8.3c for shear and tension interaction are the same as those previously reviewed separately for tension and shear alone. Tension and shear interaction is checked using *Specification* Equation I8-5 which can be written in terms of LRFD and ASD design as follows:

LRFD	ASD
$\left(\dfrac{Q_{ut}}{\phi_t Q_{nt}}\right)^{5/3} + \left(\dfrac{Q_{uv}}{\phi_v Q_{nv}}\right)^{5/3} \le 1.0$ (from *Spec.* Eq. I8-5) $\left(\dfrac{15.6\text{ kips}}{21.5\text{ kips}}\right)^{5/3} + \left(\dfrac{10.4\text{ kips}}{18.7\text{ kips}}\right)^{5/3} = 0.96$ $0.96 < 1.0$ **o.k.**	$\left(\dfrac{Q_{at}}{Q_{nt}/\Omega_t}\right)^{5/3} + \left(\dfrac{Q_{av}}{Q_{nv}/\Omega_v}\right)^{5/3} \le 1.0$ (from *Spec.* Eq. I8-5) $\left(\dfrac{10.5\text{ kips}}{14.4\text{ kips}}\right)^{5/3} + \left(\dfrac{7.00\text{ kips}}{12.4\text{ kips}}\right)^{5/3} = 0.98$ $0.98 < 1.0$ **o.k.**

Thus, a ¾-in.-diameter × 6³⁄₁₆-in.-long headed stud anchor is adequate for the applied loads.

Limits of Application

The application of the steel anchors in composite component provisions have strict limitations as summarized in the User Note provided at the beginning of AISC *Specification* Section I8.3. These provisions do not apply to typical composite beam designs nor do they apply to hybrid construction where the steel and concrete do not resist loads together via composite action such as in embed plates. This design example is intended solely to illustrate the calculations associated with an isolated anchor that is part of an applicable composite system.

Available Strength Table

Table I.12-1 provides available shear and tension strengths for standard Type B steel headed stud anchors conforming to the requirements of AWS D1.1 for use in composite components.

Table I.12-1
Steel Headed Stud Anchor Available Strengths

Anchor Shank Diameter	A_{sa}	Q_{nv}/Ω_v	$\phi_v Q_{nv}$	Q_{nv}/Ω_v	$\phi_v Q_{nv}$
		kips	kips	kips	kips
in.	in.^{2}	ASD	LRFD	ASD	LRFD
½	0.196	5.52	8.30	6.38	9.57
⅝	0.307	8.63	13.0	9.97	15.0
¾	0.442	12.4	18.7	14.4	21.5
⅞	0.601	16.9	25.4	N/A[a]	N/A[a]
1	0.785	22.1	33.2	25.5	38.3

ASD	LRFD
$\Omega_v = 2.31$	$\phi_v = 0.65$
$\Omega_t = 2.00$	$\phi_t = 0.75$

[a] ⅞-in.-diameter anchors conforming to AWS D1.1, Figure 7.1, do not meet the minimum head-to-shank diameter ratio of 1.6 as required for tensile resistance per AISC *Specification* Section I8.3.

EXAMPLE I.13 COMPOSITE COLLECTOR BEAM DESIGN

Given:

Determine if the composite beam designed in Example I.1 is adequate to serve as a collector beam for the transfer of wind-induced compression forces in combination with gravity loading as indicated in Figure I.13. Applied forces were generated from an elastic analysis and stability shall be accounted for using the effective length method of design.

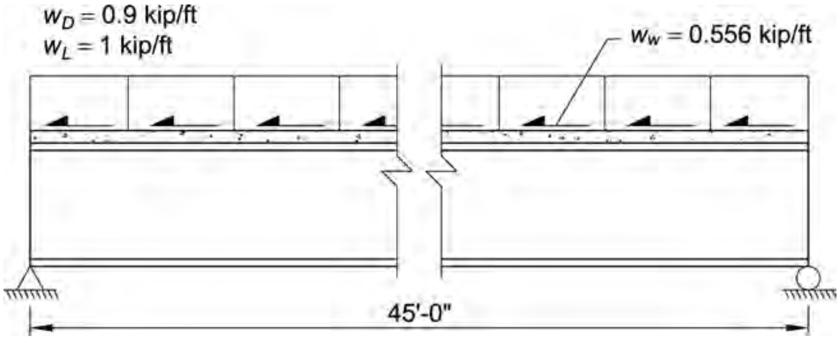

Fig. I.13. Composite collector beam and applied loading elevation.

Solution:

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W21×50

A = 14.7 in.^{2}	I_x = 984 in.^{4}	I_y = 24.9 in.^{4}	J = 1.14 in.^{4}
b_f = 6.53 in.	d = 20.8 in.	r_x = 8.18 in.	r_y = 1.30 in.
t_w = 0.380 in.	$b_f/2t_f$ = 6.10	h/t_w = 49.4	h_o = 20.3 in.

Refer to Example I.1 for additional information regarding strength and serviceability requirements associated with pre-composite and composite gravity load conditions.

Required Compressive Strength

From ASCE/SEI 7, Chapter 2, the required axial strength for the governing load combination, including wind, is:

LRFD	ASD
$P_u = 1.2D + 1.0W + L$ $= 1.2(0\text{ kips}) + 1.0(0.556\text{ kip/ft})(45\text{ ft}) + 0\text{ kips}$ $= 25.0\text{ kips}$	$P_a = D + 0.75L + 0.75(0.6W)$ $= 0\text{ kips} + 0.75(0\text{ kips})$ $+ 0.75(0.6)(0.556\text{ kip/ft})(45\text{ ft})$ $= 11.3\text{ kips}$

Available Compressive Strength (General)

The collector element is conservatively treated as a bare steel member for the determination of available compressive strength as discussed in AISC *Specification* Commentary Section I7. The effective length factor, *K*, for a pin-ended member is taken as 1.0 in accordance with Table C-A-7.1. Potential limit states are flexural buckling about both the minor and major axes, and torsional buckling.

Lateral movement is assumed to be braced by the composite slab, thus weak-axis flexural buckling will not govern by inspection as $L_{cy} = (KL)_y = 0$.

The member is slender for compression as indicated in AISC *Manual* Table 1-1, thus strong-axis flexural buckling strength is determined in accordance with AISC *Specification* Section E7 for members with slender elements for $L_{cx} = (KL)_x = 45.0$ ft.

The composite slab will prevent the member from twisting about its shear center, thus torsional buckling is not a valid limit state; however, constrained-axis torsional buckling may occur as discussed in AISC *Specification* Commentary Section E4 with $L_{cz} = (KL)_z = 1.0(45\text{ ft}) = 45.0$ ft.

Compute the available compressive strengths for the limit states of strong-axis flexural buckling and constrained-axis torsional buckling to determine the controlling strength.

Strong-Axis Flexural Buckling

Calculate the critical stress about the strong axis, F_{crx}, in accordance with AISC *Specification* Section E3 as directed by *Specification* Section E7 for members with slender elements.

$$\frac{L_{cx}}{r_x} = \frac{(45.0\text{ ft})(12\text{ in./ft})}{8.18\text{ in.}}$$
$$= 66.0$$

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}$$
$$= 113 > 66.0;\ \text{therefore, use AISC } \textit{Specification} \text{ Equation E3-2}$$

$$F_{ex} = \frac{\pi^2 E}{\left(\dfrac{L_{cx}}{r_x}\right)^2} \qquad (\textit{Spec.}\text{ Eq. E3-4})$$
$$= \frac{\pi^2(29{,}000\text{ ksi})}{(66.0)^2}$$
$$= 65.7\text{ ksi}$$

$$F_{crx} = \left(0.658^{\frac{F_y}{F_{ex}}}\right)F_y \qquad (\textit{Spec.}\text{ Eq. E3-2})$$
$$= \left(0.658^{\frac{50\text{ ksi}}{65.7\text{ ksi}}}\right)(50\text{ ksi})$$
$$= 36.4\text{ ksi}$$

Classify each component of the wide-flange member for local buckling.

Flange local buckling classification as determined from AISC *Specification* Table B4.1a, Case 1:

$$\begin{aligned}\lambda_r &= 0.56\sqrt{\frac{E}{F_y}}\\ &= 0.56\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}\\ &= 13.5\end{aligned}$$

$$\begin{aligned}\lambda &= \frac{b_f}{2t_f}\\ &= 6.10 < 13.5;\ \text{therefore, the flanges are nonslender}\end{aligned}$$

Therefore, the flanges are fully effective.

Web local buckling classification as determined from AISC *Specification* Table B4.1a, Case 5:

$$\begin{aligned}\lambda_r &= 1.49\sqrt{\frac{E}{F_y}}\\ &= 1.49\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}\\ &= 35.9\end{aligned}$$

$$\begin{aligned}\lambda &= \frac{h}{t_w}\\ &= 49.4 > 35.9;\ \text{therefore, the web is slender}\end{aligned}$$

To evaluate the impact of web slenderness on strong-axis flexural buckling, determine if a reduced effective web width, h_e, is required in accordance with AISC *Specification* Section E7.1 as follows:

$$\begin{aligned}\lambda_r\sqrt{\frac{F_y}{F_{crx}}} &= 35.9\sqrt{\frac{50\text{ ksi}}{36.4\text{ ksi}}}\\ &= 42.1 < \lambda = 49.4;\ \text{therefore, use AISC } \textit{Specification} \text{ Equation E7-3 to determine } h_e\end{aligned}$$

The effective width imperfection adjustment factors, c_1 and c_2, are selected from AISC *Specification* Table E7.1, Case (a):

$$\begin{aligned}c_1 &= 0.18\\ c_2 &= 1.31\end{aligned}$$

$$\begin{aligned}F_{el} &= \left(c_2\frac{\lambda_r}{\lambda}\right)^2 F_y\\ &= \left[1.31\left(\frac{35.9}{49.4}\right)\right]^2(50\text{ ksi})\\ &= 45.3\text{ ksi}\end{aligned}$$ (*Spec*. Eq. E7-5)

$$
\begin{aligned}
h &= \left(\frac{h}{t_w}\right)t_w \\
&= (49.4)(0.380\text{ in.}) \\
&= 18.8\text{ in.}
\end{aligned}
$$

$$
\begin{aligned}
h_e &= h\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} \qquad \text{(from } \textit{Spec.} \text{ Eq. E7-3)} \\
&= (18.8\text{ in.})\left(1 - 0.18\sqrt{\frac{45.3\text{ ksi}}{36.4\text{ ksi}}}\right)\sqrt{\frac{45.3\text{ ksi}}{36.4\text{ ksi}}} \\
&= 16.8\text{ in.}
\end{aligned}
$$

Calculate the effective area of the section:

$$
\begin{aligned}
A_e &= A - (h - h_e)t_w \\
&= 14.7\text{ in.}^2 - (18.8\text{ in.} - 16.8\text{ in.})(0.380\text{ in.}) \\
&= 13.9\text{ in.}^2
\end{aligned}
$$

Calculate the nominal compressive strength:

$$
\begin{aligned}
P_{nx} &= F_{crx}A_e \qquad (\textit{Spec.} \text{ Eq. E7-1}) \\
&= (36.4\text{ ksi})(13.9\text{ in.}^2) \\
&= 506\text{ kips}
\end{aligned}
$$

Calculate the available compressive strength:

LRFD	ASD
$\phi_c = 0.90$	$\Omega_c = 1.67$
$\phi_c P_n = 0.90(506\text{ kips})$ $= 455\text{ kips}$	$\dfrac{P_n}{\Omega_c} = \dfrac{506\text{ kips}}{1.67}$ $= 303\text{ kips}$

Constrained-Axis Torsional Buckling

Assuming the composite slab provides a lateral bracing point at the top flange of the beam, the constrained-axis buckling stress, F_{ez}, can be determined using AISC *Specification* Commentary Equaation C-E4-1 as follows:

The distance to bracing point from shear center along weak axis:

$$
\begin{aligned}
a &= \frac{d}{2} \\
&= \frac{20.8\text{ in.}}{2} \\
&= 10.4\text{ in.}
\end{aligned}
$$

The distance to bracing point from shear center along strong axis is:

$$b = 0$$

$$r_o^2 = r_x^2 + r_y^2 + a^2 + b^2 \qquad \text{(}\textit{Spec}\text{. Eq. C-E4-3)}$$
$$= (8.18\text{ in.})^2 + (1.30\text{ in.})^2 + (10.4\text{ in.})^2 + (0\text{ in.})^2$$
$$= 177\text{ in.}^2$$

From AISC *Specification* Commentary Section E4, the finite brace stiffness factor is:

$$\omega = 0.9$$

$$F_{ez} = \omega\left[\frac{\pi^2 EI_y}{(L_{cz})^2}\left(\frac{h_o{}^2}{4} + a^2\right) + GJ\right]\frac{1}{Ar_o^2} \qquad \text{(}\textit{Spec}\text{. Eq. C-E4-1)}$$
$$= 0.9\left\{\frac{\pi^2(29{,}000\text{ ksi})(24.9\text{ in.}^4)}{\left[(45.0\text{ ft})(12\text{ in./ft})\right]^2}\left[\frac{(20.3\text{ in.})^2}{4} + (10.4\text{ in.})^2\right] + (11{,}200\text{ ksi})(1.14\text{ in.}^4)\right\}$$
$$\times\left[\frac{1}{(14.7\text{ in.}^2)(177\text{ in.}^2)}\right]$$
$$= 6.20\text{ ksi}$$

To evaluate the impact of web slenderness on constrained-axis torsional buckling, determine if a reduced effective web width, h_e, is required in accordance with AISC *Specification* Section E7.1 as follows:

$$\lambda_r\sqrt{\frac{F_y}{F_{cr}}} = 35.9\sqrt{\frac{50\text{ ksi}}{6.20\text{ ksi}}}$$
$$= 102 > \lambda = 46.4;\ \text{therefore use AISC } \textit{Specification} \text{ Equation E7-2}$$

$$h_e = h \qquad \text{(from }\textit{Spec}\text{. Eq. E7-2)}$$

Thus the full steel area may be used without reduction and the available compressive strength for constrained axis buckling strength is calculated as follows:

$$L_{cz} = (KL)_z$$
$$= (45.0\text{ ft})(12\text{ in./ft})$$
$$= 540\text{ in.}$$

$$\frac{F_y}{F_{ez}} = \frac{50\text{ ksi}}{6.20\text{ ksi}}$$
$$= 8.06 > 2.25,\ \text{therefore, use AISC } \textit{Specification} \text{ Equation E3-3}$$

$$F_{crz} = 0.877F_{ez} \qquad \text{(}\textit{Spec}\text{. Eq. E3-3)}$$
$$= 0.877(6.20\text{ ksi})$$
$$= 5.44\text{ ksi}$$

The nominal compressive strength is calculated with no reduction for slenderness, $A_e = A$, as follows:

$$\begin{aligned} P_{nz} &= F_{crz}A_e \\ &= (5.44 \text{ ksi})(14.7 \text{ in.}^2) \\ &= 80.0 \text{ kips} \end{aligned}$$ (*Spec*. Eq. E7-1)

The available compressive strength is determined as follows:

LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_{nz} = 0.90(80.0 \text{ kips})$ $= 72.0 \text{ kips}$	$\Omega_c = 1.67$ $\dfrac{P_{nz}}{\Omega_c} = \dfrac{80.0 \text{ kips}}{1.67}$ $= 47.9 \text{ kips}$

Note that it may be possible to utilize the flexural stiffness and strength of the slab as a continuous torsional restraint, resulting in increased constrained-axis torsional buckling capacity; however, that exercise is beyond the scope of this design example.

A summary of the available compressive strength for each of the viable limit states is as follows:

LRFD	ASD
Strong-axis flexural buckling: $\phi_c P_{nx} = 455 \text{ kips}$	Strong-axis flexural buckling: $\dfrac{P_{nx}}{\Omega_c} = 303 \text{ kips}$
Constrained-axis torsional buckling: $\phi_c P_{nz} = 72.0 \text{ kips}$ **controls**	Constrained-axis torsional buckling: $\dfrac{P_{nz}}{\Omega_c} = 47.9 \text{ kips}$ **controls**

Required First-Order Flexural Strength

From ASCE/SEI 7, Chapter 2, the required first-order flexural strength for the governing load combination including wind is:

LRFD	ASD
$w_u = 1.2D + 1.0W + L$ $= 1.2(0.9 \text{ kip/ft}) + 1.0(0 \text{ kip/ft}) + 1 \text{ kip/ft}$ $= 2.08 \text{ kip/ft}$ $M_u = \dfrac{w_u L^2}{8}$ $= \dfrac{(2.08 \text{ kip/ft})(45 \text{ ft})^2}{8}$ $= 527 \text{ kip-ft}$	$w_a = D + 0.75L + 0.75(0.6W)$ $= 0.9 \text{ kip/ft} + 0.75(1 \text{ kip/ft}) + 0.75(0.6)(0 \text{ kip/ft})$ $= 1.65 \text{ kip/ft}$ $M_a = \dfrac{w_a L^2}{8}$ $= \dfrac{(1.65 \text{ kip/ft})(45 \text{ ft})^2}{8}$ $= 418 \text{ kip-ft}$

Required Second-Order Flexural Strength

The effective length method is utilized to consider stability for this element as permitted by AISC *Specification* Section C1.2 and Appendix 7.2. The addition of axial load will magnify the required first-order flexural strength due to member slenderness (P-δ) effects. This magnification (second-order analysis) can be approximated utilizing the procedure provided in AISC *Specification* Appendix 8 as permitted by Section C2.1b.

Calculate the elastic critical buckling strength of the member in the plane of bending (in this case about the strong-axis of the beam) from AISC *Specification* Appendix 8, Section 8.2.1. For the effective length method, EI^* is taken as EI in accordance with Appendix 8.2.1, and the effective length, L_{cx} is taken as $(KL)_x$ in accordance with Appendix 7.2.3. As illustrated previously, K, is taken as 1.0 for a pin-ended member. Conservatively using the bare steel beam moment of inertia, the buckling strength is calculated as follows:

$$P_{e1} = \frac{\pi^2 EI^*}{\left(L_{c1}\right)^2} \qquad \text{(}\textit{Spec}\text{. Eq. A-8-5)}$$

$$= \frac{\pi^2 EI}{\left(KL\right)_x^2} \qquad \text{(for the effective length method)}$$

$$= \frac{\pi^2 \left(29{,}000 \text{ ksi}\right)\left(984 \text{ in.}^4\right)}{\left[\left(45.0 \text{ ft}\right)\left(12 \text{ in./ft}\right)\right]^2}$$

$$= 966 \text{ kips}$$

For beam-columns subject to transverse loading between supports, the value of C_m is taken as 1.0 as permitted by AISC *Specification* Appendix 8, Section 8.2.1(b), and B_1 is calculated from *Specification* Equation A-8-3 as follows:

LRFD	ASD
$B_1 = \dfrac{C_m}{1 - \alpha P_u / P_{e1}} \geq 1$ $= \dfrac{1.0}{1 - 1.0\left(\dfrac{25.0 \text{ kips}}{966 \text{ kips}}\right)} \geq 1$ $= 1.03$	$B_1 = \dfrac{C_m}{1 - \alpha P_a / P_{e1}} \geq 1$ $= \dfrac{1.0}{1 - 1.6\left(\dfrac{11.3 \text{ kips}}{966 \text{ kips}}\right)} \geq 1$ $= 1.02$

Noting that the first-order moment is induced by vertical dead and live loading, it is classified as a non-translational moment, M_{nt}, in accordance with AISC *Specification* Section 8.2. The required second-order flexural strength is therefore calculated using AISC *Specification* Equation A-8-1 as:

LRFD	ASD
$M_u = B_1 M_{nt} + B_2 M_{lt}$ $= 1.03\left(527 \text{ kip-ft}\right) + 0$ $= 543 \text{ kip-ft}$	$M_a = B_1 M_{nt} + B_2 M_{lt}$ $= 1.02\left(418 \text{ kip-ft}\right) + 0$ $= 426 \text{ kip-ft}$

Available Flexural Strength

The available flexural strength of the composite beam is calculated in Example I.1 as:

LRFD	ASD
$\phi_b M_{nx} = 769 \text{ kip-ft}$	$\dfrac{M_{nx}}{\Omega_b} = 512 \text{ kip-ft}$

Interaction of Axial Force and Flexure

Interaction between axial forces and flexure in composite collector beams is addressed in AISC *Specification* Commentary Section I7, which states that the non-composite axial strength and the composite flexural strength may be used with the interaction equations provided in Chapter H as a reasonable simplification for design purposes. This procedure is illustrated as follows:

LRFD	ASD
$\phi_c P_n = 72.0\text{ kips}$	$\dfrac{P_n}{\Omega_c} = 47.9\text{ kips}$
$\phi_b M_{nx} = 769\text{ kip-ft}$	$\dfrac{M_{nx}}{\Omega_c} = 512\text{ kip-ft}$
$\dfrac{P_r}{P_c} = \dfrac{P_u}{\phi_c P_n}$ $= \dfrac{25.0\text{ kips}}{72.0\text{ kips}}$ $= 0.347 > 0.2$	$\dfrac{P_r}{P_c} = \dfrac{P_a}{P_n / \Omega_c}$ $= \dfrac{11.3\text{ kips}}{47.9\text{ kips}}$ $= 0.236 > 0.2$
Therefore, use AISC *Specification* Equation H1-1a.	Therefore, use AISC *Specification* Equation H1-1a.
$\dfrac{P_u}{\phi_c P_n} + \dfrac{8}{9}\left(\dfrac{M_u}{\phi_b M_{nx}}\right) \le 1.0$ $0.347 + \dfrac{8}{9}\left(\dfrac{543\text{ kip-ft}}{769\text{ kip-ft}}\right) \le 1.0$ $0.975 < 1.0$ **o.k.**	$\dfrac{P_a}{P_n / \Omega_c} + \dfrac{8}{9}\left(\dfrac{M_a}{M_{nx} / \Omega_b}\right) \le 1.0$ $0.236 + \dfrac{8}{9}\left(\dfrac{426\text{ kip-ft}}{512\text{ kip-ft}}\right) \le 1.0$ $0.976 < 1.0$ **o.k.**

The collector element is adequate to resist the imposed loads.

Load Introduction Effects

AISC *Specification* Commentary Section I7 indicates that the effect of the vertical offset between the plane of the diaphragm and the collector element should be investigated. It has been shown that the resulting eccentricity between the plane of axial load introduction in the slab and the centroid of the beam connections does not result in any additional flexural demand assuming the axial load is introduced uniformly along the length of the beam; however, this eccentricity will result in additional shear reactions (Burmeister and Jacobs, 2008). The additional shear reaction assuming an eccentricity of *d*/2 is calculated as follows:

LRFD	ASD
$V_{u\text{-}add} = \dfrac{P_u d}{2L}$ $= \dfrac{(25.0\text{ kips})(20.8\text{ in.})}{2(45\text{ ft})(12\text{ in./ft})}$ $= 0.481\text{ kips}$	$V_{a\text{-}add} = \dfrac{P_a d}{2L}$ $= \dfrac{(11.3\text{ kips})(20.8\text{ in.})}{2(45\text{ ft})(12\text{ in./ft})}$ $= 0.218\text{ kips}$

As can be seen from these results, the additional vertical shear due to the axial collector force is quite small and in most instances will be negligible versus the governing shear resulting from gravity-only load combinations.

Shear Connection

AISC *Specification* Commentary Section I7 notes that it is not required to superimpose the horizontal shear due to lateral forces with the horizontal shear due to flexure for the determination of steel anchor requirements, thus the summation of nominal strengths for all steel anchors along the beam length may be used for axial force transfer. Specific resistance and safety factors for this condition are not provided in Section I8.2 as they are implicitly accounted for within the system resistance and safety factors used for the determination of the available flexural strength of the beam. Until additional research becomes available, a conservative approach is to apply the composite component factors from *Specification* Section I8.3 to the nominal steel anchor strengths determined from *Specification* Section I8.2.

From Example I.1, the strength for ¾-in.-diameter anchors in normal weight concrete with $f_c' = 4$ ksi and deck oriented perpendicular to the beam is:

1 anchor per rib: $Q_n = 17.2 \text{ kips/anchor}$

2 anchors per rib: $Q_n = 14.6 \text{ kips/anchor}$

Over the entire beam length, there are 42 anchors in positions with one anchor per rib and four anchors in positions with two anchors per rib, thus the total available strength for diaphragm shear transfer is:

LRFD	ASD
$\phi_v = 0.65$ $\phi_c P_n = 0.65\left[42\left(17.2 \text{ kips/anchor}\right) + 4(14.6 \text{ kips/anchor})\right]$ $= 508 \text{ kips} > 25.0 \text{ kips}$ **o.k.**	$\Omega_v = 2.31$ $\dfrac{P_n}{\Omega_c} = \dfrac{42\left(17.2 \text{ kips/anchor}\right) + 4\left(14.6 \text{ kips/anchor}\right)}{2.31}$ $= 338 \text{ kips} > 11.3 \text{ kips}$ **o.k.**

Note that the longitudinal available shear strength of the diaphragm itself (consisting of the composite deck and concrete fill) will often limit the amount of force that can be introduced into the collector beam and should also be evaluated as part of the overall design.

Summary

A W21×50 collector with 46, ¾-in.-diameter by 4⅞-in.-long, steel headed stud anchors is adequate to resist the imposed loads.

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CHAPTER I DESIGN EXAMPLE REFERENCES

ACI 318 (2014), *Building Code Requirements for Structural Concrete*, ACI 318-14; *and Commentary*, ACI 318R-14, American Concrete Institute, Farmington Hills, MI.

ASCE (2014), *Design Loads on Structures During Construction*, ASCE/SEI 37-14, American Society of Civil Engineers, Reston, VA.

AWS (2015), *Structural Welding Code—Steel*, AWS D1.1/D1.1M:2015, American Welding Society, Miami, FL.

Burmeister, S. and Jacobs, W.P. (2008), "Under Foot: Horizontal Floor Diaphragm Load Effects on Composite Beam Design," *Modern Steel Construction*, AISC, December.

Griffis, L.G. (1992), *Load and Resistance Factor Design of W-Shapes Encased in Concrete*, Design Guide 6, AISC, Chicago, IL.

ICC (2015), *International Building Code*, International Code Council, Falls Church, VA.

Leon, R.T. and Hajjar, J.F. (2008), "Limit State Response of Composite Columns and Beam-Columns Part 2: Application of Design Provisions for the 2005 AISC Specification," Engineering Journal, AISC, Vol. 45, No. 1, pp. 21–46.

Murray, T.M., Allen, D.E., Ungar, E.E. and Davis, D.B. (2016), *Floor Vibrations Due to Human Activity*, Design Guide 11, 2nd Ed., AISC, Chicago, IL.

Park, R. and Gamble, W.L. (2000), *Reinforced Concrete Slabs*, 2nd Ed., John Wiley & Sons, New York, NY.

SDI (2011), *Standard for Composite Steel Floor Deck-Slabs*, ANSI/SDI C1.0-2011, Glenshaw, PA.

West, M.A. and Fisher, J.M. (2003), *Serviceability Design Consideration for Steel Buildings*, Design Guide 3, 2nd Ed., AISC, Chicago, IL.

Young, W.C. and Budynas, R.C. (2002), *Roark's Formulas for Stress and Strain*, 7th Ed., McGraw-Hill, New York, NY.

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Chapter J
Design of Connections

AISC *Specification* Chapter J addresses the design of connections. The chapter's primary focus is the design of welded and bolted connections. Design requirements for fillers, splices, column bases, concentrated forces, anchors rods and other threaded parts are also covered. See AISC *Specification* Appendix 3 for special requirements for connections subject to fatigue.

EXAMPLE J.1 FILLET WELD IN LONGITUDINAL SHEAR

Given:

As shown in Figure J.1-1, a ¼-in.-thick × 18-in. wide plate is fillet welded to a ⅜-in.-thick plate. The plates are ASTM A572 Grade 50 and have been properly sized. Use 70-ksi electrodes. Note that the plates could be specified as ASTM A36, but F_y = 50 ksi plate has been used here to demonstrate the requirements for long welds.

Confirm that the size and length of the welds shown are adequate to resist the applied loading.

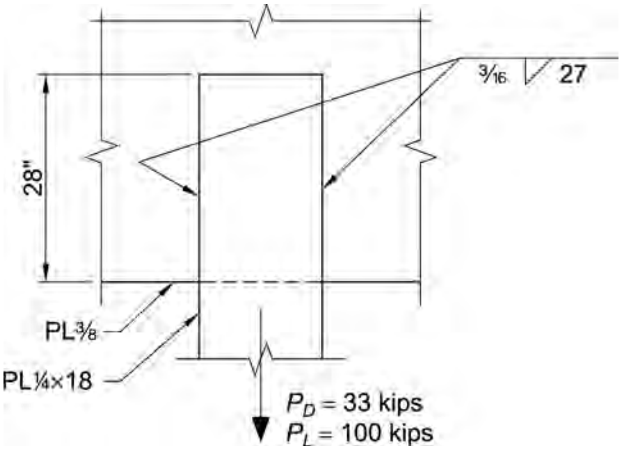

Fig. J.1-1. Geometry and loading for Example J.1.

Solution:

From AISC *Manual* Table 2-5, the material properties are as follows:

ASTM A572 Grade 50
F_y = 50 ksi
F_u = 65 ksi

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$P_u = 1.2(33\text{ kips}) + 1.6(100\text{ kips})$ $= 200\text{ kips}$	$P_a = 33\text{ kips} + 100\text{ kips}$ $= 133\text{ kips}$

Maximum and Minimum Weld Size

Because the thickness of the overlapping plate is ¼ in., the maximum fillet weld size that can be used without special notation per AISC *Specification* Section J2.2b, is a 3⁄16-in. fillet weld. A 3⁄16-in. fillet weld can be deposited in the flat or horizontal position in a single pass (true up to 5⁄16-in.).

From AISC *Specification* Table J2.4, the minimum size of the fillet weld, based on a material thickness of ¼ in. is ⅛ in.

Weld Strength

The nominal weld strength per inch of 3⁄16-in. weld, determined from AISC *Specification* Section J2.4(b) is:

$$\begin{aligned} R_n &= F_{nw}A_{we} \qquad \text{(\textit{Spec}. Eq. J2-4)} \\ &= \left(0.60F_{EXX}\right)A_{we} \\ &= 0.60\left(70\text{ ksi}\right)\left(\frac{3/16\text{ in.}}{\sqrt{2}}\right) \\ &= 5.57\text{ kip/in.} \end{aligned}$$

From AISC *Specification* Section J2.2b, check the weld length to weld size ratio, because this is an end-loaded fillet weld.

$$\begin{aligned} \frac{l}{w} &= \frac{27.0\text{ in.}}{3/16\text{ in.}} \\ &= 144 > 100;\ \text{therefore, AISC \textit{Specification} Equation J2-1 must be applied} \end{aligned}$$

$$\begin{aligned} \beta &= 1.2 - 0.002\left(l/w\right) \le 1.0 \qquad \text{(\textit{Spec}. Eq. J2-1)} \\ &= 1.2 - 0.002\left(144\right) \le 1.0 \\ &= 0.912 \end{aligned}$$

The nominal weld shear rupture strength is:

$$\begin{aligned} R_n &= 0.912\left(5.57\text{ kip/in.}\right)\left(2\text{ welds}\right)\left(27\text{ in.}\right) \\ &= 274\text{ kips} \end{aligned}$$

From AISC *Specification* Section J2.4, the available shear rupture strength is:

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75\left(274\text{ kips}\right)$ $= 206\text{ kips} > 200\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{274\text{ kips}}{2.00}$ $= 137\text{ kips} > 133\text{ kips}$ **o.k.**

The base metal strength is determined from AISC *Specification* Section J2.4(a). The ¼-in.-thick plate controls:

$$\begin{aligned} R_n &= F_{nBM}A_{BM} \qquad \text{(\textit{Spec}. Eq. J2-2)} \\ &= 0.60F_u t_p l_{weld} \\ &= 0.60\left(65\text{ ksi}\right)\left(1/4\text{ in.}\right)\left(2\text{ welds}\right)\left(27\text{ in.}\right) \\ &= 527\text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75\left(527\text{ kips}\right)$ $= 395\text{ kips} > 200\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{527\text{ kips}}{2.00}$ $= 264\text{ kips} > 133\text{ kips}$ **o.k.**

EXAMPLE J.2 FILLET WELD LOADED AT AN ANGLE

Given:

Verify a fillet weld at the edge of a gusset plate is adequate to resist a force of 50 kips due to dead load and 150 kips due to live load, at an angle of 60° relative to the weld, as shown in Figure J.2-1. Assume the beam and the gusset plate thickness and length have been properly sized. Use a 70-ksi electrode.

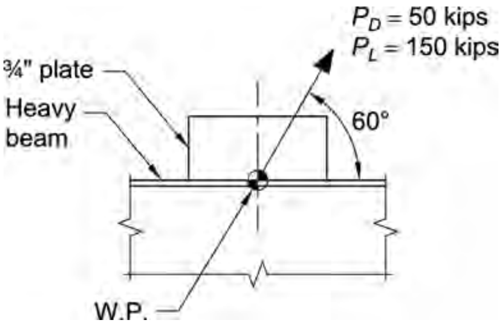

Fig. J.2-1. Geometry and loading for Example J.2.

Solution:

From ASCE/SEI 7, Chapter 2, the required tensile strength is:

LRFD	ASD
$P_u = 1.2(50\text{ kips}) + 1.6(150\text{ kips})$ $= 300\text{ kips}$	$P_a = 50\text{ kips} + 150\text{ kips}$ $= 200\text{ kips}$

Assume a ⁵⁄₁₆-in. fillet weld is used on each side of the plate.

Note that from AISC *Specification* Table J2.4, the minimum size of fillet weld, based on a material thickness of ¾ in. is ¼ in. (assuming the beam flange thickness exceeds ¾ in.).

Available Shear Strength of the Fillet Weld Per Inch of Length

From AISC *Specification* Section J2.4(b), the nominal strength of the fillet weld is determined as follows:

$$R_n = F_{nw}A_{we} \qquad \text{(\textit{Spec}. Eq. J2-4)}$$

$$= 0.60F_{EXX}\left(1.0 + 0.50\sin^{1.5}60°\right)A_{we}$$

$$= 0.60(70\text{ ksi})\left(1.0 + 0.50\sin^{1.5}60°\right)\left(\frac{5/16\text{ in.}}{\sqrt{2}}\right)$$

$$= 13.0\text{ kip/in.}$$

From AISC *Specification* Section J2.4(b), the available shear strength per inch of weld for fillet welds on two sides is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(13.0\text{ kip/in.})(2\text{ sides})$ $= 19.5\text{ kip/in.}$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{13.0\text{ kip/in.}}{2.00}(2\text{ sides})$ $= 13.0\text{ kip/in.}$

Required Length of Weld

LRFD	ASD
$l = \dfrac{300\text{ kips}}{19.5\text{ kip/in.}}$ $= 15.4\text{ in.}$ Use 16 in. on each side of the plate.	$l = \dfrac{200\text{ kips}}{13.0\text{ kip/in.}}$ $= 15.4\text{ in.}$ Use 16 in. on each side of the plate.

EXAMPLE J.3 COMBINED TENSION AND SHEAR IN BEARING-TYPE CONNECTIONS

Given:

A ¾-in.-diameter, Group A bolt with threads not excluded from the shear plane (thread condition N) is subjected to a tension force of 3.5 kips due to dead load and 12 kips due to live load, and a shear force of 1.33 kips due to dead load and 4 kips due to live load. Check the combined stresses according to AISC *Specification* Equations J3-3a and J3-3b.

Solution:

From ASCE/SEI 7, Chapter 2, the required tensile and shear strengths are:

LRFD	ASD
Tension: $T_u = 1.2(3.5\text{ kips}) + 1.6(12\text{ kips})$ $= 23.4\text{ kips}$ Shear: $V_u = 1.2(1.33\text{ kips}) + 1.6(4\text{ kips})$ $= 8.00\text{ kips}$	Tension: $T_a = 3.5\text{ kips} + 12\text{ kips}$ $= 15.5\text{ kips}$ Shear: $V_a = 1.33\text{ kips} + 4\text{ kips}$ $= 5.33\text{ kips}$

Available Tensile Strength

When a bolt is subject to combined tension and shear, the available tensile strength is determined according to the limit states of tension and shear rupture, from AISC *Specification* Section J3.7 as follows.

From AISC *Specification* Table J3.2, Group A bolts:

$F_{nt} = 90\text{ ksi}$
$F_{nv} = 54\text{ ksi}$

From AISC *Manual* Table 7-2, for a ¾-in.-diameter bolt:

$A_b = 0.442\text{ in.}^2$

The available shear stress is determined as follows and must equal or exceed the required shear stress.

LRFD	ASD
$\phi = 0.75$ $\phi F_{nv} = 0.75(54\text{ ksi})$ $= 40.5\text{ ksi}$ $f_{rv} = \dfrac{V_u}{A_b}$ $= \dfrac{8.00\text{ kips}}{0.442\text{ in.}^2}$ $= 18.1\text{ ksi} < 40.5\text{ ksi}$ **o.k.**	$\Omega = 2.00$ $\dfrac{F_{nv}}{\Omega} = \dfrac{54\text{ ksi}}{2.00}$ $= 27.0\text{ ksi}$ $f_{rv} = \dfrac{V_a}{A_b}$ $= \dfrac{5.33\text{ kips}}{0.442\text{ in.}^2}$ $= 12.1\text{ ksi} < 27.0\text{ ksi}$ **o.k.**

The available tensile strength of a bolt subject to combined tension and shear is as follows:

LRFD	ASD
$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt}$ (*Spec*. Eq. J3-3a) $= 1.3(90\text{ ksi}) - \frac{90\text{ ksi}}{40.5\text{ ksi}}(18.1\text{ ksi}) \le 90\text{ ksi}$ $= 76.8\text{ ksi}$ For combined tension and shear, ϕ = 0.75, from AISC *Specification* Section J3.7. $\phi R_n = \phi F'_{nt} A_b$ (*Spec*. Eq. J3-2) $= 0.75(76.8\text{ ksi})(0.442\text{ in.}^2)$ $= 25.5\text{ kips} > 23.4\text{ kips}$ **o.k.**	$F'_{nt} = 1.3F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_{rv} \le F_{nt}$ (*Spec*. Eq. J3-3b) $= 1.3(90\text{ ksi}) - \frac{90\text{ ksi}}{27.0\text{ ksi}}(12.1\text{ ksi}) \le 90\text{ ksi}$ $= 76.7\text{ ksi}$ For combined tension and shear, Ω = 2.00, from AISC *Specification* Section J3.7. $\frac{R_n}{\Omega} = \frac{F'_{nt} A_b}{\Omega}$ (*Spec*. Eq. J3-2) $= \frac{(76.7\text{ ksi})(0.442\text{ in.}^2)}{2.00}$ $= 17.0\text{ kips} > 15.5\text{ kips}$ **o.k.**

The effects of combined shear and tensile stresses need not be investigated if either the required shear or tensile stress is less than or equal to 30% of the corresponding available stress per the User Note at the end of AISC *Specification* Section J3.7. In the example herein, both the required shear and tensile stresses exceeded the 30% threshold and evaluation of combined stresses was necessary.

AISC *Specification* Equations J3-3a and J3-3b may be rewritten so as to find a nominal shear stress, F'_{nv}, as a function of the required tensile stress as is shown in AISC *Specification* Commentary Equations C-J3-7a and C-J3-7b.

EXAMPLE J.4A SLIP-CRITICAL CONNECTION WITH SHORT-SLOTTED HOLES

Slip-critical connections shall be designed to prevent slip and for the limit states of bearing-type connections.

Given:

Refer to Figure J.4A-1 and select the number of bolts that are required to support the loads shown when the connection plates have short slots transverse to the load and no fillers are provided. Select the number of bolts required for slip resistance only.

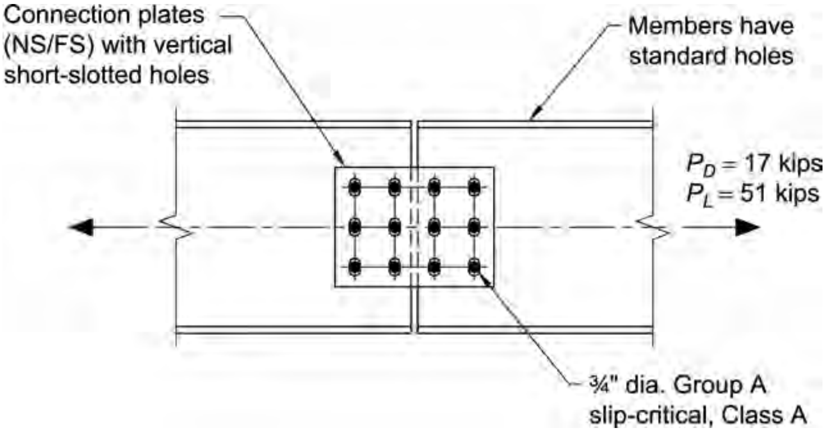

Fig. J.4A-1. Geometry and loading for Example J.4A.

Solution:

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$P_u = 1.2(17\text{ kips}) + 1.6(51\text{ kips})$ $= 102\text{ kips}$	$P_a = 17\text{ kips} + 51\text{ kips}$ $= 68.0\text{ kips}$

From AISC *Specification* Section J3.8(a), the available slip resistance for the limit state of slip for standard size and short-slotted holes perpendicular to the direction of the load is determined as follows:

ϕ = 1.00
Ω = 1.50
μ = 0.30 for Class A surface
D_u = 1.13
h_f = 1.0, no filler is provided
T_b = 28 kips, from AISC *Specification* Table J3.1, Group A
n_s = 2, number of slip planes

$$R_n = \mu D_u h_f T_b n_s \qquad \text{(\textit{Spec}. Eq. J3-4)}$$
$$= 0.30(1.13)(1.0)(28\text{ kips})(2)$$
$$= 19.0\text{ kips/bolt}$$

The available slip resistance is:

LRFD	ASD
$\phi R_n = 1.00(19.0 \text{ kips/bolt})$ $= 19.0 \text{ kips/bolt}$	$\dfrac{R_n}{\Omega} = \dfrac{19.0 \text{ kips/bolt}}{1.50}$ $= 12.7 \text{ kips/bolt}$

Required Number of Bolts

LRFD	ASD
$n_b = \dfrac{P_u}{\phi R_n}$ $= \dfrac{102 \text{ kips}}{19.0 \text{ kips/bolt}}$ $= 5.37 \text{ bolts}$ Use 6 bolts	$n_b = \dfrac{P_a}{\left(\dfrac{R_n}{\Omega}\right)}$ $= \dfrac{68.0 \text{ kips}}{12.7 \text{ kips/bolt}}$ $= 5.35 \text{ bolts}$ Use 6 bolts

Note: To complete the verification of this connection, the limit states of bolt shear, bearing, tearout, tensile yielding, tensile rupture, and block shear rupture must also be checked.

EXAMPLE J.4B SLIP-CRITICAL CONNECTION WITH LONG-SLOTTED HOLES

Given:

Repeat Example J.4A with the same loads, but assuming that the connection plates have long-slotted holes in the direction of the load, as shown in Figure J.4B-1.

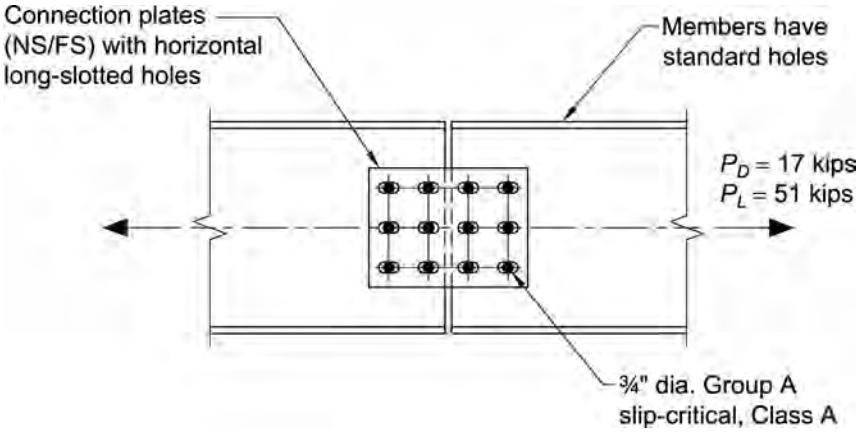

Fig. J.4B-1. Geometry and loading for Example J.4B.

Solution:

The required strength from Example J.4A is:

LRFD	ASD
$P_u = 102$ kips	$P_a = 68.0$ kips

From AISC *Specification* Section J3.8(c), the available slip resistance for the limit state of slip for long-slotted holes is determined as follows:

ϕ = 0.70
Ω = 2.14
μ = 0.30 for Class A surface
D_u = 1.13
h_f = 1.0, no filler is provided
T_b = 28 kips, from AISC *Specification* Table J3.1, Group A
n_s = 2, number of slip planes

$$\begin{aligned} R_n &= \mu D_u h_f T_b n_s \\ &= 0.30(1.13)(1.0)(28 \text{ kips})(2) \\ &= 19.0 \text{ kips/bolt} \end{aligned}$$ (*Spec*. Eq. J3-4)

The available slip resistance is:

LRFD	ASD
$\phi R_n = 0.70(19.0 \text{ kips/bolt})$ $= 13.3 \text{ kips/bolt}$	$\dfrac{R_n}{\Omega} = \dfrac{19.0 \text{ kips/bolt}}{2.14}$ $= 8.88 \text{ kips/bolt}$

Required Number of Bolts

LRFD	ASD
$n_b = \dfrac{P_u}{\phi R_n}$ $= \dfrac{102 \text{ kips}}{13.3 \text{ kips/bolt}}$ $= 7.67 \text{ bolts}$ Use 8 bolts	$n_b = \dfrac{P_a}{\left(\dfrac{R_n}{\Omega}\right)}$ $= \dfrac{68.0 \text{ kips}}{8.88 \text{ kips/bolt}}$ $= 7.66 \text{ bolts}$ Use 8 bolts

Note: To complete the verification of this connection, the limit states of bolt shear, bearing, tearout, tensile yielding, tensile rupture, and block shear rupture must be determined.

EXAMPLE J.5 COMBINED TENSION AND SHEAR IN A SLIP-CRITICAL CONNECTION

Because the pretension of a bolt in a slip-critical connection is used to create the clamping force that produces the shear strength of the connection, the available shear strength must be reduced for any load that produces tension in the connection.

Given:

The slip-critical bolt group shown in Figure J.5-1 is subjected to tension and shear. This example shows the design for bolt slip resistance only, and assumes that the beams and plates are adequate to transmit the loads. Determine if the bolts are adequate.

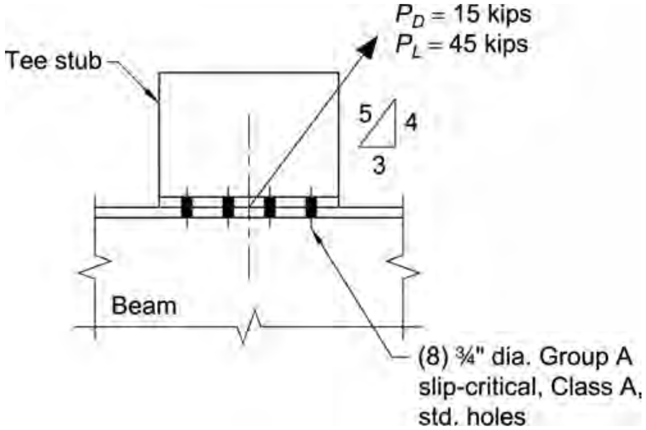

Fig. J.5-1. Geometry and loading for Example J.5.

Solution:

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$P_u = 1.2(15\text{ kips}) + 1.6(45\text{ kips})$ $= 90.0\text{ kips}$ By geometry: $T_u = \frac{4}{5}(90.0\text{ kips})$ $= 72.0\text{ kips}$ $V_u = \frac{3}{5}(90.0\text{ kips})$ $= 54.0\text{ kips}$	$P_a = 15\text{ kips} + 45\text{ kips}$ $= 60.0\text{ kips}$ By geometry: $T_a = \frac{4}{5}(60.0\text{ kips})$ $= 48.0\text{ kips}$ $V_a = \frac{3}{5}(60.0\text{ kips})$ $= 36.0\text{ kips}$

Available Bolt Tensile Strength

The available tensile strength is determined from AISC *Specification* Section J3.6.

From AISC *Specification* Table J3.2 for Group A bolts, the nominal tensile strength in ksi is, F_{nt} = 90 ksi. From AISC *Manual* Table 7-1, for a ¾-in.-diameter bolt:

$A_b = 0.442\text{ in.}^2$

The nominal tensile strength is:

$$\begin{aligned} R_n &= F_{nt} A_b \qquad \text{(from } \textit{Spec}\text{. Eq. J3-1)} \\ &= (90\text{ ksi})(0.442\text{ in.}^2) \\ &= 39.8\text{ kips} \end{aligned}$$

The available tensile strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(39.8\text{ kips/bolt}) > \dfrac{72.0\text{ kips}}{8\text{ bolts}}$ $= 29.9\text{ kips/bolt} > 9.00\text{ kips/bolt}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{39.8\text{ kips/bolt}}{2.00} > \dfrac{48.0\text{ kips}}{8\text{ bolts}}$ $= 19.9\text{ kips/bolt} > 6.00\text{ kips/bolt}$ **o.k.**

Note that the available tensile strength per bolt can also be taken from AISC *Manual* Table 7-2.

Available Slip Resistance per Bolt

The available slip resistance for one bolt in standard size holes is determined using AISC *Specification* Section J3.8(a):

ϕ = 1.00
Ω = 1.50
μ = 0.30 for Class A surface
D_u = 1.13
h_f = 1.0, factor for fillers, assuming no more than one filler
T_b = 28 kips, from AISC *Specification* Table J3.1, Group A
n_s = 1, number of slip planes

LRFD	ASD
Determine the available slip resistance (T_u = 0) of a bolt: $\phi R_n = \phi \mu D_u h_f T_b n_s$ (from *Spec*. Eq. J3-4) $= 1.00(0.30)(1.13)(1.0)(28\text{ kips})(1)$ $= 9.49\text{ kips/bolt}$	Determine the available slip resistance (T_a = 0) of a bolt: $\dfrac{R_n}{\Omega} = \dfrac{\mu D_u h_f T_b n_s}{\Omega}$ (from *Spec*. Eq. J3-4) $= \dfrac{0.30(1.13)(1.0)(28\text{ kips})(1)}{1.50}$ $= 6.33\text{ kips/bolt}$

Note that the available slip resistance for one bolt with a Class A faying surface can also be taken from AISC *Manual* Table 7-3.

Available Slip Resistance of the Connection

Because the slip-critical connection is subject to combined tension and shear, the available slip resistance is multiplied by a reduction factor provided in AISC *Specification* Section J3.9.

LRFD	ASD
Slip-critical combined tension and shear factor:	Slip-critical combined tension and shear factor:
$k_{sc} = 1 - \dfrac{T_u}{D_u T_b n_b} \geq 0$ (*Spec*. Eq. J3-5a) $= 1 - \dfrac{72.0\text{ kips}}{1.13(28\text{ kips})(8)} > 0$ $= 0.716$	$k_{sc} = 1 - \dfrac{1.5T_a}{D_u T_b n_b} \geq 0$ (*Spec*. Eq. J3-5b) $= 1 - \dfrac{1.5(48.0\text{ kips})}{1.13(28\text{ kips})(8)} > 0$ $= 0.716$
$\phi R_n = \phi R_n k_{sc} n_b$ $= (9.49\text{ kips/bolt})(0.716)(8\text{ bolts})$ $= 54.4\text{ kips} > 54.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{R_n}{\Omega} k_{sc} n_b$ $= (6.33\text{ kips/bolt})(0.716)(8\text{ bolts})$ $= 36.3\text{ kips} > 36.0\text{ kips}$ **o.k.**

Note: The bolt group must still be checked for all applicable strength limit states for a bearing-type connection.

EXAMPLE J.6 BASE PLATE BEARING ON CONCRETE

Given:

As shown in Figure J.6-1, an ASTM A992 column bears on a concrete pedestal with f'_c = 3 ksi. The space between the base plate and the concrete pedestal has grout with f'_c = 4 ksi. Verify the ASTM A36 base plate will support the following loads in axial compression:

P_D = 115 kips
P_L = 345 kips

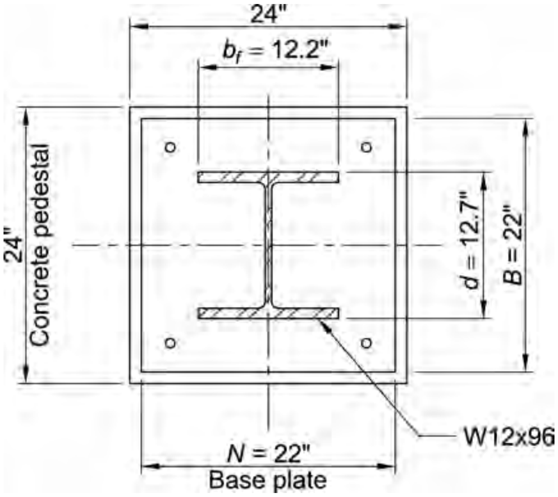

Fig. J.6-1. Geometry for Example J.6.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Column
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Base Plate
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Column
W12×96
d = 12.7 in.
b_f = 12.2 in.
t_f = 0.900 in.
t_w = 0.550 in.

From ASCE/SEI 7, Chapter 2, the required compressive strength is:

LRFD	ASD
$P_u = 1.2(115\text{ kips}) + 1.6(345\text{ kips})$ $= 690\text{ kips}$	$P_a = 115\text{ kips} + 345\text{ kips}$ $= 460\text{ kips}$

Base Plate Dimensions

Determine the required base plate area from AISC *Specification* Section J8 conservatively assuming bearing on the full area of the concrete support.

LRFD	ASD
$\phi_c = 0.65$ $A_{1(req)} = \dfrac{P_u}{\phi_c 0.85 f_c'}$ (from *Spec*. Eq. J8-1) $= \dfrac{690\text{ kips}}{0.65(0.85)(3\text{ ksi})}$ $= 416\text{ in.}^2$	$\Omega_c = 2.31$ $A_{1(req)} = \dfrac{\Omega_c P_a}{0.85 f_c'}$ (from *Spec*. Eq. J8-1) $= \dfrac{2.31(460\text{ kips})}{0.85(3\text{ ksi})}$ $= 417\text{ in.}^2$

Note: The strength of the grout has conservatively been neglected, as its strength is greater than that of the concrete pedestal.

Try a 22-in. × 22-in. base plate.

Verify $N \geq d + 2(3\text{ in.})$ and $B \geq b_f + 2(3\text{ in.})$ for anchor rod pattern shown in diagram:

$$\begin{aligned} d + 2(3\text{ in.}) &= 12.7\text{ in.} + 2(3\text{ in.}) \\ &= 18.7\text{ in.} < 22\text{ in.} \quad \textbf{o.k.} \end{aligned}$$

$$\begin{aligned} b_f + 2(3\text{ in.}) &= 12.2\text{ in.} + 2(3\text{ in.}) \\ &= 18.2\text{ in.} < 22\text{ in.} \quad \textbf{o.k.} \end{aligned}$$

Base plate area:

$$\begin{aligned} A_1 &= NB \\ &= (22\text{ in.})(22\text{ in.}) \\ &= 484\text{ in.}^2 > 417\text{ in.}^2 \quad \textbf{o.k.} \text{ (conservatively compared to ASD value for } A_{1(req)}) \end{aligned}$$

Note: A square base plate with a square anchor rod pattern will be used to minimize the chance for field and shop problems.

Concrete Bearing Strength

Use AISC *Specification* Equation J8-2 because the base plate covers less than the full area of the concrete support.

Because the pedestal is square and the base plate is a concentrically located square, the full pedestal area is also the geometrically similar area. Therefore:

$$A_2 = (24\text{ in.})(24\text{ in.})$$
$$= 576\text{ in.}^2$$

The available bearing strength is:

LRFD	ASD
$\phi_c = 0.65$ $\phi_c P_p = \phi_c 0.85 f'_c A_1 \sqrt{\dfrac{A_2}{A_1}} \le \phi_c 1.7 f'_c A_1$ (from *Spec.* Eq. J8-2) $= 0.65(0.85)(3\text{ ksi})(484\text{ in.}^2)\sqrt{\dfrac{576\text{ in.}^2}{484\text{ in.}^2}}$ $\le 0.65(1.7)(3\text{ ksi})(484\text{ in.}^2)$ $= 875\text{ kips} < 1{,}600\text{ kips, use } 875\text{ kips}$ 875 kips > 690 kips **o.k.**	$\Omega_c = 2.31$ $\dfrac{P_p}{\Omega_c} = \dfrac{0.85 f'_c A_1}{\Omega_c}\sqrt{\dfrac{A_2}{A_1}} \le \dfrac{1.7 f'_c A_1}{\Omega_c}$ (from *Spec.* Eq. J8-2) $= \dfrac{0.85(3\text{ ksi})(484\text{ in.}^2)}{2.31}\sqrt{\dfrac{576\text{ in.}^2}{484\text{ in.}^2}}$ $\le \dfrac{1.7(3\text{ ksi})(484\text{ in.}^2)}{2.31}$ $= 583\text{ kips} < 1{,}070\text{ kips, use } 583\text{ kips}$ 583 kips > 460 kips **o.k.**

Notes:

1. $A_2/A_1 \le 4$; therefore, the upper limit in AISC *Specification* Equation J8-2 does not control.
2. As the area of the base plate approaches the area of concrete, the modifying ratio, $\sqrt{A_2/A_1}$, approaches unity and AISC *Specification* Equation J8-2 converges to AISC *Specification* Equation J8-1.

Required Base Plate Thickness

The base plate thickness is determined in accordance with AISC *Manual* Part 14.

$$m = \frac{N - 0.95d}{2} \qquad \text{(}\textit{Manual}\text{ Eq. 14-2)}$$
$$= \frac{22\text{ in.} - 0.95(12.7\text{ in.})}{2}$$
$$= 4.97\text{ in.}$$

$$n = \frac{B - 0.8b_f}{2} \qquad \text{(}\textit{Manual}\text{ Eq. 14-3)}$$
$$= \frac{22\text{ in.} - 0.8(12.2\text{ in.})}{2}$$
$$= 6.12\text{ in.}$$

$$n' = \frac{\sqrt{db_f}}{4} \qquad \text{(}\textit{Manual}\text{ Eq. 14-4)}$$
$$= \frac{\sqrt{(12.7\text{ in.})(12.2\text{ in.})}}{4}$$
$$= 3.11\text{ in.}$$

LRFD	ASD
$X = \left[\dfrac{4db_f}{\left(d+b_f\right)^2}\right]\dfrac{P_u}{\phi_c P_p}$ (*Manual* Eq. 14-6a) $= \left[\dfrac{4(12.7\text{ in.})(12.2\text{ in.})}{(12.7\text{ in.}+12.2\text{ in.})^2}\right]\left(\dfrac{690\text{ kips}}{875\text{ kips}}\right)$ $= 0.788$	$X = \left[\dfrac{4db_f}{\left(d+b_f\right)^2}\right]\dfrac{\Omega_c P_a}{P_p}$ (*Manual* Eq. 14-6b) $= \left[\dfrac{4(12.7\text{ in.})(12.2\text{ in.})}{(12.7\text{ in.}+12.2\text{ in.})^2}\right]\left(\dfrac{460\text{ kips}}{583\text{ kips}}\right)$ $= 0.789$

Conservatively, use the LRFD value for *X*.

$$\lambda = \frac{2\sqrt{X}}{1+\sqrt{1-X}} \le 1 \qquad \text{(*Manual* Eq. 14-5)}$$

$$= \frac{2\sqrt{0.788}}{1+\sqrt{1-0.788}} \le 1$$

$$= 1.22 > 1,\ \text{use } \lambda = 1$$

Note: λ can always be conservatively taken equal to 1.

$$\lambda n' = 1(3.11\text{ in.})$$

$$= 3.11\text{ in.}$$

$$l = \max\{m,\ n,\ \lambda n'\}$$

$$= \max\{4.97\text{ in., } 6.12\text{ in., } 3.11\text{ in.}\}$$

$$= 6.12\text{ in.}$$

LRFD	ASD
$f_{pu} = \dfrac{P_u}{BN}$ $= \dfrac{690\text{ kips}}{(22\text{ in.})(22\text{ in.})}$ $= 1.43\text{ ksi}$ From AISC *Manual* Equation 14-7a: $t_{min} = l\sqrt{\dfrac{2f_{pu}}{0.90F_y}}$ $= (6.12\text{ in.})\sqrt{\dfrac{2(1.43\text{ ksi})}{0.90(36\text{ ksi})}}$ $= 1.82\text{ in.}$	$f_{pa} = \dfrac{P_a}{BN}$ $= \dfrac{460\text{ kips}}{(22\text{ in.})(22\text{ in.})}$ $= 0.950\text{ ksi}$ From AISC *Manual* Equation 14-7b: $t_{min} = l\sqrt{\dfrac{1.67(2f_{pa})}{F_y}}$ $= (6.12\text{ in.})\sqrt{\dfrac{1.67(2)(0.950\text{ ksi})}{36\text{ ksi}}}$ $= 1.82\text{ in.}$

Use PL2 in. × 22 in. × 1 ft 10 in., ASTM A36.

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Chapter K
Additional Requirements for HSS and Box Section Connections

Examples K.1 through K.6 illustrate common beam-to-column shear connections that have been adapted for use with HSS columns. Example K.7 illustrates a through-plate shear connection, which is unique to HSS columns. Calculations for transverse and longitudinal forces applied to HSS are illustrated in Example K.8. Examples of HSS base plate and end plate connections are given in Examples K.9 and K.10.

EXAMPLE K.1 WELDED/BOLTED WIDE TEE CONNECTION TO AN HSS COLUMN

Given:

Verify a connection between an ASTM A992 W16×50 beam and an ASTM A500, Grade C, HSS8×8×¼ column using an ASTM A992 WT-shape, as shown in Figure K.1-1. Design, assuming a flexible support condition, for the following vertical shear loads:

P_D = 6.2 kips
P_L = 18.5 kips

Note: A tee with a flange width wider than 8 in. was selected to provide sufficient surface for flare bevel groove welds on both sides of the column, because the tee will be slightly offset from the column centerline.

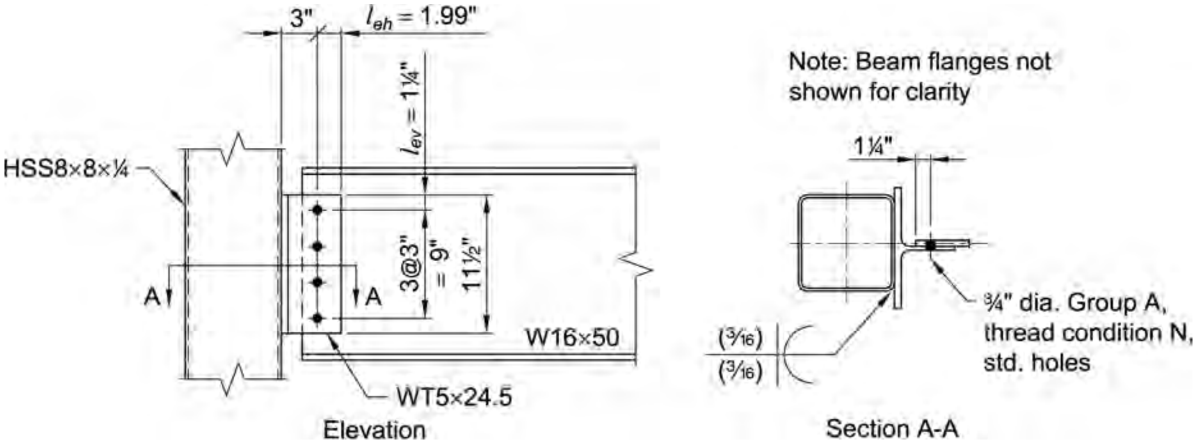

Fig K.1-1. Connection geometry for Example K.1.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Tee
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Column
ASTM A500 Grade C
F_y = 50 ksi
F_u = 62 ksi

From AISC *Manual* Tables 1-1, 1-8 and 1-12, the geometric properties are as follows:

W16×50
t_w = 0.380 in.
d = 16.3 in.
t_f = 0.630 in.
T = 13⅝ in.

WT5×24.5
$t_{sw} = t_w$ = 0.340 in.
d = 4.99 in.
t_f = 0.560 in.
b_f = 10.0 in.
k_1 = 13⁄16 in. (see W10×49)

HSS8×8×¼
t = 0.233 in.
B = 8.00 in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$P_u = 1.2(6.2\text{ kips}) + 1.6(18.5\text{ kips})$ $= 37.0\text{ kips}$	$P_a = 6.2\text{ kips} + 18.5\text{ kips}$ $= 24.7\text{ kips}$

Calculate the available strength assuming a flexible support condition.

Required Number of Bolts

The required number of bolts will ultimately be determined using the coefficient, C, from AISC *Manual* Table 7-6. First, the available strength per bolt must be determined.

Determine the available shear strength of a single bolt. From AISC *Manual* Table 7-1, for ¾-in.-diameter Group A bolts:

LRFD	ASD
$\phi r_n = 17.9\text{ kips}$	$\dfrac{r_n}{\Omega} = 11.9\text{ kips}$

The edge distance is checked against the minimum edge distance requirement provided in AISC *Specification* Table J3.4.

$l_{ev} = 1\tfrac{1}{4}\text{ in.} > 1\text{ in.}$ **o.k.**

The available bearing and tearout strength per bolt on the tee stem based on edge distance is determined from AISC *Manual* Table 7-5, for l_{ev} = 1¼ in., as follows:

LRFD	ASD
$\phi r_n = (49.4\text{ kip/in.})(0.340\text{ in.})$ $= 16.8\text{ kips}$	$\dfrac{r_n}{\Omega} = (32.9\text{ kip/in.})(0.340\text{ in.})$ $= 11.2\text{ kips}$

The bolt spacing is checked against the minimum spacing requirement between centers of standard holes provided in AISC *Specification* Section J3.3.

$$\begin{aligned} 2\tfrac{2}{3}d &= 2\tfrac{2}{3}\left(\tfrac{3}{4}\text{ in.}\right) \\ &= 2.00\text{ in.} > s = 3\text{ in.} \quad \textbf{o.k.} \end{aligned}$$

The available bearing and tearout strength per bolt on the tee stem based on spacing is determined from AISC *Manual* Table 7-4, for s = 3 in., as follows:

LRFD	ASD
$\phi r_n = (87.8\text{ kip/in.})(0.340\text{ in.})$ $= 29.9\text{ kips}$	$\dfrac{r_n}{\Omega} = (58.5\text{ kip/in.})(0.340\text{ in.})$ $= 19.9\text{ kips}$

Bolt bearing and tearout strength based on edge distance controls over the available shear strength of the bolt.

Determine the coefficient for the eccentrically loaded bolt group.

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$ $= \dfrac{37.0\text{ kips}}{16.8\text{ kips}}$ $= 2.20$ Using e = 3 in. and s = 3 in., determine C from AISC *Manual* Table 7-6, Angle = 0°. Try four rows of bolts: $C = 2.81 > 2.20$ **o.k.**	$C_{min} = \dfrac{P_a}{r_n / \Omega}$ $= \dfrac{24.7\text{ kips}}{11.2\text{ kips}}$ $= 2.21$ Using e = 3 in. and s = 3 in., determine C from AISC *Manual* Table 7-6, Angle = 0°. Try four rows of bolts: $C = 2.81 > 2.21$ **o.k.**

Tee Stem Thickness and Length

AISC *Manual* Part 9 stipulates a maximum tee stem thickness that should be provided for rotational ductility as follows:

$$\begin{aligned} t_{sw\,max} &= \frac{d}{2} + \tfrac{1}{16}\text{ in.} \\ &= \frac{\tfrac{3}{4}\text{ in.}}{2} + \tfrac{1}{16}\text{ in.} \\ &= 0.438\text{ in.} > 0.340\text{ in.} \quad \textbf{o.k.} \end{aligned}$$

(from *Manual* Eq. 9-39)

Note: The beam web thickness is greater than the tee stem thickness. If the beam web were thinner than the tee stem, this check could be satisfied by checking the thickness of the beam web.

As discussed in AISC *Manual* Part 10, it is recommended that the minimum length of a simple shear connection is one-half the *T*-dimension of the beam to be supported. The minimum length of the tee is determined as follow:

$$l_{min} = \frac{T}{2}$$
$$= \frac{13\tfrac{5}{8}\text{ in.}}{2}$$
$$= 6.81\text{ in.}$$

As discussed in AISC *Manual* Part 10, the detailed length of connection elements must be compatible with the *T*-dimension of the beam. The tee length is checked using the number of bolts, bolt spacing, and edge distances determined previously.

$$l = 3(3\text{ in.}) + 2(1\tfrac{1}{4}\text{ in.})$$
$$= 11.5\text{ in.} < T = 13\tfrac{5}{8}\text{ in.} \quad \textbf{o.k.}$$

Try l = 11.5 in.

Tee Stem Shear Yielding Strength

Determine the available shear strength of the tee stem based on the limit state of shear yielding from AISC *Specification* Section J4.2(a).

$$A_{gv} = lt_s$$
$$= (11.5\text{ in.})(0.340\text{ in.})$$
$$= 3.91\text{ in.}^2$$

$$R_n = 0.60F_yA_{gv} \qquad (\textit{Spec}.\text{ Eq. J4-3})$$
$$= 0.60(50\text{ ksi})(3.91\text{ in.}^2)$$
$$= 117\text{ kips}$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(117\text{ kips})$ $= 117\text{ kips} > 37.0\text{ kips} \quad \textbf{o.k.}$	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{117\text{ kips}}{1.50}$ $= 78.0\text{ kips} > 24.7\text{ kips} \quad \textbf{o.k.}$

Because of the geometry of the tee and because the tee flange is thicker than the stem and carries only half of the beam reaction, flexural yielding and shear yielding of the flange are not controlling limit states.

Tee Stem Shear Rupture Strength

Determine the available shear strength of the tee stem based on the limit state of shear rupture from AISC *Specification* Section J4.2(b).

$$A_{nv} = \left[l - n\left(d_n + \tfrac{1}{16}\text{ in.}\right)\right]t_s$$
$$= \left[11.5\text{ in.} - (4)\left(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right](0.340\text{ in.})$$
$$= 2.72\text{ in.}^2$$

$$\begin{aligned} R_n &= 0.60F_uA_{nv} \\ &= 0.60(65\text{ ksi})(2.72\text{ in.}^2) \\ &= 106\text{ kips} \end{aligned}$$ (*Spec*. Eq. J4-4)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(106\text{ kips})$ $= 79.5\text{ kips} > 37.0\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{106\text{ kips}}{2.00}$ $= 53.0\text{ kips} > 24.7\text{ kips}$ **o.k.**

Tee Stem Block Shear Rupture Strength

The nominal strength for the limit state of block shear rupture is given by AISC *Specification* Section J4.3.

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt}$$ (*Spec*. Eq. J4-5)

The available block shear rupture strength of the tee stem is determined as follows, using AISC *Manual* Tables 9-3a, 9-3b and 9-3c and AISC *Specification* Equation J4-5, with n = 4, l_{eh} = 1.99 in. (assume l_{eh} = 2.00 in. to use Table 9-3a), l_{ev} = 1¼ in. and U_{bs} = 1.0.

LRFD	ASD
Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{\phi F_uA_{nt}}{t} = 76.2\text{ kip/in.}$	Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{F_uA_{nt}}{\Omega t} = 50.8\text{ kip/in.}$
Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{\phi 0.60F_yA_{gv}}{t} = 231\text{ kip/in.}$	Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{0.60F_yA_{gv}}{\Omega t} = 154\text{ kip/in.}$
Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{\phi 0.60F_uA_{nv}}{t} = 210\text{ kip/in.}$	Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{0.60F_uA_{nv}}{\Omega t} = 140\text{ kip/in.}$

LRFD	ASD
The design block shear rupture strength is: $\phi R_n = \phi 0.60 F_u A_{vn} + \phi U_{bs} F_u A_{nt}$ $\leq \phi 0.60 F_y A_{gv} + \phi U_{bs} F_u A_{nt}$ $= (210 \text{ kip/in.} + 76.2 \text{ kip/in.})(0.340 \text{ in.})$ $\leq (231 \text{ kip/in.} + 76.2 \text{ kip/in.})(0.340 \text{ in.})$ $= 97.3 \text{ kips} < 104 \text{ kips}$ $= 97.3 \text{ kips} > 37.0 \text{ kips}$ **o.k.**	The allowable block shear rupture strength is: $\dfrac{R_n}{\Omega} = \dfrac{0.60 F_u A_{nv}}{\Omega} + \dfrac{U_{bs} F_u A_{nt}}{\Omega}$ $\leq \dfrac{0.60 F_y A_{gv}}{\Omega} + \dfrac{U_{bs} F_u A_{nt}}{\Omega}$ $= (140 \text{ kip/in.} + 50.8 \text{ kip/in.})(0.340 \text{ in.})$ $\leq (154 \text{ kip/in.} + 50.8 \text{ kip/in.})(0.340 \text{ in.})$ $= 64.9 \text{ kips} < 69.6 \text{ kips}$ $= 64.9 \text{ kips} > 24.7 \text{ kips}$ **o.k.**

Tee Stem Flexural Strength

The required flexural strength for the tee stem is:

LRFD	ASD
$M_u = P_u e$ $= (37.0 \text{ kips})(3 \text{ in.})$ $= 111 \text{ kip-in.}$	$M_a = P_a e$ $= (24.7 \text{ kips})(3 \text{ in.})$ $= 74.1 \text{ kip-in.}$

The tee stem available flexural strength due to yielding is determined as follows, from AISC *Specification* Section F11.1. The stem, in this case, is treated as a rectangular bar.

$$
\begin{aligned}
Z &= \frac{t_s d^2}{4} \\
&= \frac{(0.340 \text{ in.})(11.5 \text{ in.})^2}{4} \\
&= 11.2 \text{ in.}^3
\end{aligned}
$$

$$
\begin{aligned}
S_x &= \frac{t_s d^2}{6} \\
&= \frac{(0.340 \text{ in.})(11.5 \text{ in.})^2}{6} \\
&= 7.49 \text{ in.}^3
\end{aligned}
$$

$$
\begin{aligned}
M_n &= M_p = F_y Z \leq 1.6 F_y S_x \\
&= (50 \text{ ksi})(11.2 \text{ in.}^3) \leq 1.6(50 \text{ ksi})(7.49 \text{ in.}^3) \\
&= 560 \text{ kip-in.} < 599 \text{ kip-in.} \\
&= 560 \text{ kips-in.}
\end{aligned}
$$

(*Spec*. Eq. F11-1)

Note: The 1.6 limit will never control for a plate because the shape factor (Z/S) for a plate is 1.5.

The tee stem available flexural yielding strength is:

LRFD	ASD
$\phi = 0.90$ $\phi M_n = 0.90(560 \text{ kip-in.})$ $= 504 \text{ kip-in.} > 111 \text{ kip-in.}$ **o.k.**	$\Omega = 1.67$ $\dfrac{M_n}{\Omega} = \dfrac{560 \text{ kip-in.}}{1.67}$ $= 335 \text{ kip-in.} > 74.1 \text{ kip-in.}$ **o.k.**

The tee stem available flexural strength due to lateral-torsional buckling is determined from Section F11.2.

$$\begin{aligned}\frac{L_b d}{t_s^2} &= \frac{(3 \text{ in.})(11.5 \text{ in.})}{(0.340 \text{ in.})^2}\\ &= 298\end{aligned}$$

$$\begin{aligned}\frac{0.08E}{F_y} &= \frac{0.08(29{,}000 \text{ ksi})}{50 \text{ ksi}}\\ &= 46.4\end{aligned}$$

$$\begin{aligned}\frac{1.9E}{F_y} &= \frac{1.9(29{,}000 \text{ ksi})}{50 \text{ ksi}}\\ &= 1{,}102\end{aligned}$$

Because 46.4 < 298 < 1,102, Equation F11-2 is applicable with C_b = 1.00.

$$M_n = C_b\left[1.52 - 0.274\left(\frac{L_b d}{t^2}\right)\frac{F_y}{E}\right]M_y \le M_p \qquad \text{(\textit{Spec.} Eq. F11-2)}$$

$$\begin{aligned}&= 1.00\left[1.52 - 0.274(298)\left(\frac{50 \text{ ksi}}{29{,}000 \text{ ksi}}\right)\right](50 \text{ ksi})(7.49 \text{ in.}^2) \le (50 \text{ ksi})(11.2 \text{ in.}^3)\\ &= 517 \text{ kip-in.} < 560 \text{ kip-in.}\\ &= 517 \text{ kip-in.}\end{aligned}$$

LRFD	ASD
$\phi = 0.90$ $\phi M_n = 0.90(517 \text{ kip-in.})$ $= 465 \text{ kip-in.} > 111 \text{ kip-in.}$ **o.k.**	$\Omega = 1.67$ $\dfrac{M_n}{\Omega} = \dfrac{517 \text{ kip-in.}}{1.67}$ $= 310 \text{ kip-in.} > 74.1 \text{ kip-in.}$ **o.k.**

The tee stem available flexural strength is determined from AISC *Manual* Part 9 as follows:

$$\begin{aligned}Z_{net} &= \frac{td^2}{4} - 2t_{sw}\left(d_h + \tfrac{1}{16} \text{ in.}\right)(1.5 \text{ in.} + 4.5 \text{ in.})\\ &= \frac{(0.340 \text{ in.})(11.5 \text{ in.})^2}{4} - 2(0.340 \text{ in.})\left(\tfrac{13}{16} \text{ in.} + \tfrac{1}{16} \text{ in.}\right)(1.5 \text{ in.} + 4.5 \text{ in.})\\ &= 7.67 \text{ in.}^3\end{aligned}$$

$$M_n = F_u Z_{net}$$ (*Manual* Eq. 9-4)

$$= (65 \text{ ksi})(7.67 \text{ in.}^3)$$

$$= 499 \text{ kip-in.}$$

LRFD	ASD
$\phi_b = 0.75$ $\phi M_n = 0.75(499 \text{ kip-in.})$ $= 374 \text{ kip-in.} > 111 \text{ kip-in.}$ **o.k.**	$\Omega_b = 2.00$ $\dfrac{M_n}{\Omega} = \dfrac{499 \text{ kip-in.}}{2.00}$ $= 250 \text{ kip-in.} > 74.1 \text{ kip-in.}$ **o.k.**

Beam Web Bearing

Because t_w = 0.380 in. > t_{sw} = 0.340 in., bolt bearing does not control the strength of the beam web.

Weld Size

Because the flange width of the tee is larger than the width of the HSS, a flare bevel groove weld is required. Taking the outside radius as $R = 2t$ = 2(0.233 in.) = 0.466 in. and using AISC *Specification* Table J2.2, the effective throat thickness of the flare bevel groove weld is E = 5⁄16R = 5⁄16(0.466 in.) = 0.146 in. This effective throat thickness will be used for subsequent calculations; however, for the detail drawing, a 3⁄16-in. weld is specified.

Using AISC *Specification* Table J2.3, the minimum effective throat thickness of the flare bevel groove weld, based on the 0.233 in. thickness of the HSS column, is 1⁄8 in.

$$E = 0.146 \text{ in.} > \tfrac{1}{8} \text{ in.}$$

The equivalent fillet weld that provides the same throat dimension is:

$$\left(\frac{D}{16}\right)\left(\frac{1}{\sqrt{2}}\right) = 0.146$$

$$D = 16\sqrt{2}(0.146)$$

$$= 3.30 \text{ sixteenths of an inch}$$

The equivalent fillet weld size is used in the following calculations.

Weld Ductility

Check weld ductility using AISC *Manual* Part 9.

Let $b_f = B$ = 8.00 in.

$$b = \frac{b_f - 2k_1}{2}$$

$$= \frac{8.00 \text{ in.} - 2(\tfrac{13}{16} \text{ in.})}{2}$$

$$= 3.19 \text{ in}$$

$$w_{min} = 0.0155\frac{F_y t_f^{\,2}}{b}\left(\frac{b^2}{l^2}+2\right) \le \left(5/8\right)t_{sw} \qquad \text{(}\textit{Manual}\text{ Eq. 9-37)}$$

$$= 0.0155\frac{\left(50\text{ ksi}\right)\left(0.560\text{ in.}\right)^2}{3.19\text{ in.}}\left[\frac{\left(3.19\text{ in.}\right)^2}{\left(11.5\text{ in.}\right)^2}+2\right] \le \left(5/8\right)\left(0.340\text{ in.}\right)$$

$$= 0.158\text{ in.} < 0.213\text{in.}$$

0.158 in. = 2.53 sixteenths of an inch

$D_{min} = 2.53 < 3.30$ sixteenths of an inch **o.k.**

Nominal Weld Shear Strength

The load is assumed to act concentrically with the weld group (i.e., a flexible support condition).

a = 0 and *k* = 0; therefore, *C* = 3.71 from AISC *Manual* Table 8-4, Angle = 0°.

$$\begin{aligned} R_n &= CC_1Dl \\ &= 3.71\left(1.00\right)\left(3.30\text{ sixteenths of an inch}\right)\left(11.5\text{ in.}\right) \\ &= 141\text{ kips} \end{aligned}$$

Shear Rupture of the HSS at the Weld

$$t_{min} = \frac{3.09D}{F_u} \qquad \text{(}\textit{Manual}\text{ Eq. 9-2)}$$

$$= \frac{3.09\left(3.30\text{ sixteenths}\right)}{62\text{ ksi}}$$

$$= 0.164\text{ in.} < 0.233\text{ in.}$$

By inspection, shear rupture of the tee flange at the welds will not control.

Therefore, the weld controls.

Available Weld Shear Strength

From AISC *Specification* Section J2.4, the available weld strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(141\text{ kips}\right)$ $= 106\text{ kips} > 37.0\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{141\text{ kips}}{2.00}$ $= 70.5\text{ kips} > 24.7\text{ kips}$ **o.k.**

EXAMPLE K.2 WELDED/BOLTED NARROW TEE CONNECTION TO AN HSS COLUMN

Given:

Verify a connection for an ASTM A992 W16×50 beam to an ASTM A500 Grade C HSS8×8×¼ column using an ASTM A992 WT5×24.5 with fillet welds against the flat width of the HSS, as shown in Figure K.2-1. Use 70-ksi weld electrodes. Assume that, for architectural purposes, the flanges of the WT from the previous example have been stripped down to a width of 5 in. Design assuming a flexible support condition for the following vertical shear loads:

P_D = 6.2 kips
P_L = 18.5 kips

Note: This is the same problem as Example K.1 with the exception that a narrow tee will be selected which will permit fillet welds on the flat of the column. The beam will still be centered on the column centerline; therefore, the tee will be slightly offset.

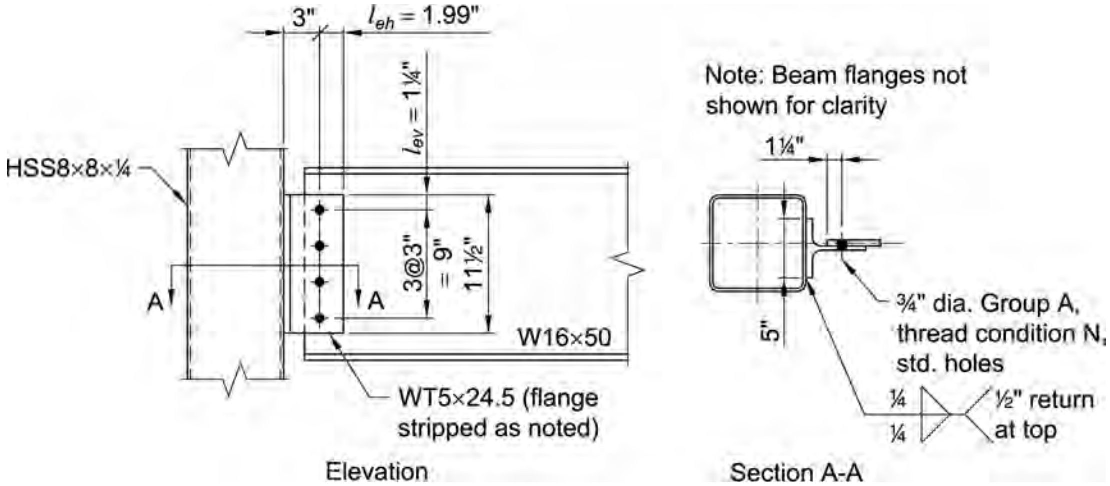

Fig K.2-1. Connection geometry for Example K.2.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Tee
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Column
ASTM A500 Grade C
F_y = 50 ksi
F_u = 62 ksi

From AISC *Manual* Tables 1-1, 1-8 and 1-12, the geometric properties are as follows:

W16×50
$t_w = 0.380$ in.
$d = 16.3$ in.
$t_f = 0.630$ in.

HSS8×8×¼
$t = 0.233$ in.
$B = 8.00$ in.

WT5×24.5
$t_{sw} = t_w = 0.340$ in.
$d = 4.99$ in.
$t_f = 0.560$ in.
$k_1 = {}^{13}\!/_{16}$ in. (see W10×49)

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$P_u = 1.2(6.2\text{ kips}) + 1.6(18.5\text{ kips})$ $= 37.0\text{ kips}$	$P_a = 6.2\text{ kips} + 18.5\text{ kips}$ $= 24.7\text{ kips}$

The tee stem thickness, tee length, tee stem strength, and beam web bearing strength are verified in Example K.1. The required number of bolts is also determined in Example K.1.

Maximum Tee Flange Width

Assume ¼-in. welds and HSS corner radius equal to 2.25 times the nominal thickness 2.25(¼ in.) = 9⁄16 in. (refer to AISC *Manual* Part 1 discussion).

The recommended minimum shelf dimension for ¼-in. fillet welds from AISC *Manual* Figure 8-13 is ½ in.

Connection offset (centerline of the column to the centerline of the tee stem):

$$\frac{0.380\text{ in.}}{2} + \frac{0.340\text{ in.}}{2} = 0.360\text{ in.}$$

The stripped flange must not exceed the flat face of the tube minus the shelf dimension on each side:

$$b_f \le 8.00\text{ in.} - 2\left({}^{9}\!/_{16}\text{ in.}\right) - 2\left({}^{1}\!/_{2}\text{ in.}\right) - 2\left(0.360\text{ in.}\right)$$

$$5.00\text{ in.} < 5.16\text{ in.} \quad \textbf{o.k.}$$

Minimum Fillet Weld Size

From AISC *Specification* Table J2.4, the minimum fillet weld size = ⅛ in. ($D = 2$) for welding to 0.233-in.-thick material.

Weld Ductility

The flexible width of the connecting element, b, is defined in Figure 9-6 of AISC *Manual* Part 9:

$$\begin{aligned} b &= \frac{b_f - 2k_1}{2} \\ &= \frac{5.00\text{ in.} - 2\left(^{13}\!/_{16}\text{ in.}\right)}{2} \\ &= 1.69\text{ in.} \end{aligned}$$

$$w_{min} = 0.0155\frac{F_y t_f^{\,2}}{b}\left(\frac{b^2}{l^2}+2\right) \le \left(^5\!/_8\right)t_{sw} \qquad (\textit{Manual}\text{ Eq. 9-37})$$

$$\begin{aligned} &= 0.0155\frac{(50\text{ ksi})(0.560\text{ in.})^2}{1.69\text{ in.}}\left[\frac{(1.69\text{ in.})^2}{(11.5\text{ in.})^2}+2\right] \le \left(^5\!/_8\right)(0.340\text{ in.}) \\ &= 0.291\text{ in.} > 0.213\text{ in.; therefore, use } w_{min} = 0.213\text{ in.} \end{aligned}$$

$$\begin{aligned} D_{min} &= (0.213\text{ in.})(16) \\ &= 3.41\text{ sixteenths of an inch} \end{aligned}$$

Try a ¼-in. fillet weld as a practical minimum, which is less than the maximum permitted weld size of t_f – 1⁄16 in. = 0.560 in. – 1⁄16 in. = 0.498 in., in accordance with AISC *Specification* Section J2.2b. Provide ½-in. return welds at the top of the tee to meet the criteria listed in AISC *Specification* Section J2.2b.

Minimum HSS Wall Thickness to Match Weld Strength

$$t_{min} = \frac{3.09D}{F_u} \qquad (\textit{Manual}\text{ Eq. 9-2})$$

$$\begin{aligned} &= \frac{3.09(4)}{62\text{ ksi}} \\ &= 0.199\text{ in.} < 0.233\text{ in.} \end{aligned}$$

By inspection, shear rupture of the flange of the tee at the welds will not control.

Therefore, the weld controls.

Available Weld Shear Strength

The load is assumed to act concentrically with the weld group (i.e., a flexible support condition).

a = 0 and k = 0, therefore, C = 3.71 from AISC *Manual* Table 8-4, Angle = 0°.

$$\begin{aligned} R_n &= CC_1Dl \\ &= 3.71(1.00)(4\text{ sixteenths of an inch})(11.5\text{ in.}) \\ &= 171\text{ kips} \end{aligned}$$

From AISC *Specification* Section J2.4, the available fillet weld shear strength is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(171\text{ kips})$ $= 128\text{ kips} > 37.0\text{ kips}$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{171\text{ kips}}{2.00}$ $= 85.5\text{ kips} > 24.7\text{ kips}$

EXAMPLE K.3 DOUBLE-ANGLE CONNECTION TO AN HSS COLUMN

Given:

Use AISC *Manual* Tables 10-1 and 10-2 to design a double-angle connection for an ASTM A992 W36×231 beam to an ASTM A500 Grade C HSS14×14×½ column, as shown in Figure K.3-1. The angles are ASTM A36 material. Use 70-ksi weld electrodes. The bottom flange cope is required for erection. Use the following vertical shear loads:

P_D = 37.5 kips
P_L = 113 kips

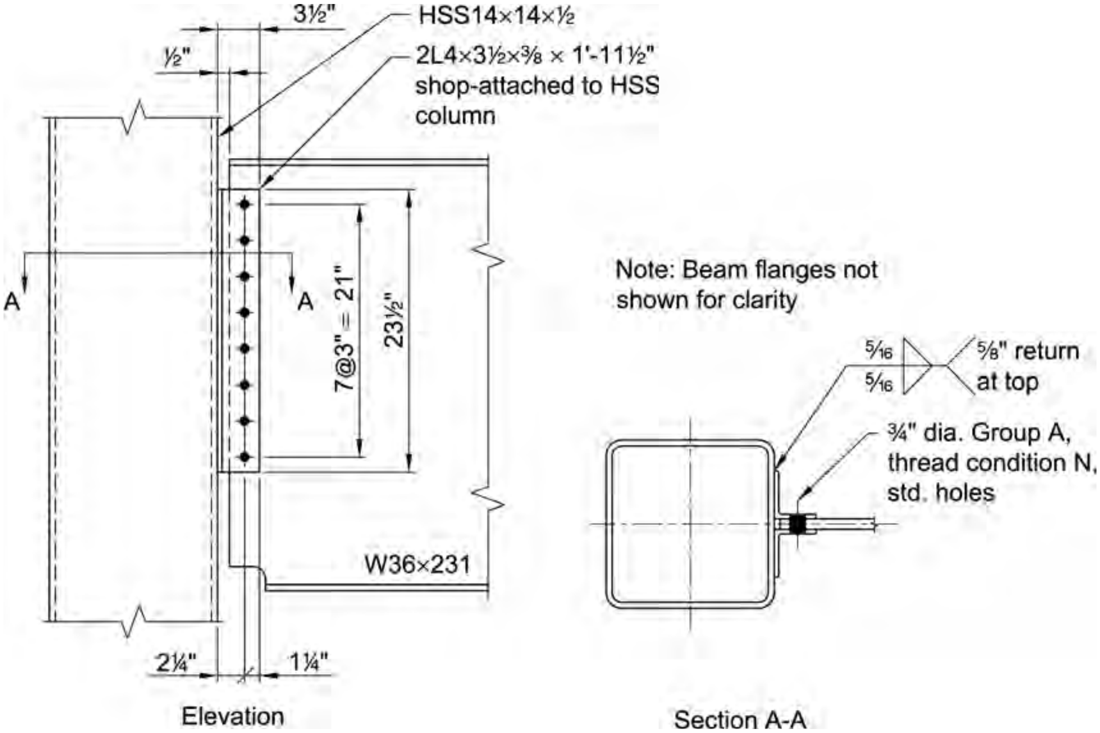

Fig K.3-1. Connection geometry for Example K.3.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Column
ASTM A500 Grade C
F_y = 50 ksi
F_u = 62 ksi

Angles
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Tables 1-1 and 1-12, the geometric properties are as follows:

W36×231
t_w = 0.760 in.
T = 31⅜ in.

HSS14×14×½
t = 0.465 in.
B = 14.0 in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(37.5 \text{ kips}) + 1.6(113 \text{ kips})$ $= 226 \text{ kips}$	$R_a = 37.5 \text{ kips} + 113 \text{ kips}$ $= 151 \text{ kips}$

Bolt and Weld Design

Try eight rows of bolts and 5⁄16-in. welds.

Obtain the bolt group and angle available strength from AISC *Manual* Table 10-1, Group A.

LRFD	ASD
$\phi R_n = 284 \text{ kips} > 226 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 189 \text{ kips} > 151 \text{ kips}$ **o.k.**

Obtain the available weld strength from AISC *Manual* Table 10-2 (welds B).

LRFD	ASD
$\phi R_n = 279 \text{ kips} > 226 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 186 \text{ kips} > 151 \text{ kips}$ **o.k.**

Minimum Support Thickness

The minimum required support thickness using AISC *Manual* Table 10-2 is determined as follows for F_u = 62 ksi material.

$$0.238 \text{ in.}\left(\frac{65 \text{ ksi}}{62 \text{ ksi}}\right) = 0.250 \text{ in.} < 0.465 \text{ in.} \quad \textbf{o.k.}$$

Minimum Angle Thickness

$$\begin{aligned} t_{min} &= w + \tfrac{1}{16} \text{ in., from AISC \textit{Specification} Section J2.2b} \\ &= \tfrac{5}{16} \text{ in.} + \tfrac{1}{16} \text{ in.} \\ &= \tfrac{3}{8} \text{ in.} \end{aligned}$$

Use ⅜-in. angle thickness to accommodate the welded legs of the double-angle connection.

Use 2L4×3½×⅜×1′-11½″.

Minimum Angle Length

As discussed in AISC *Manual* Part 10, it is recommended that the minimum length of a simple shear connection is one-half the *T*-dimension of the beam to be supported. The minimum length of the connection is determined as follow:

$$\begin{aligned} l_{min} &= \frac{T}{2} \\ &= \frac{31\tfrac{3}{8}\text{ in.}}{2} \\ &= 15.7\text{ in.} < 23.5\text{ in.} \quad \textbf{o.k.} \end{aligned}$$

Minimum Column Width

The workable flat for the HSS column is 11¾ in. from AISC *Manual* Table 1-12.

The recommended minimum shelf dimension for ⁵⁄₁₆-in. fillet welds from AISC *Manual* Figure 8-13 is ⁹⁄₁₆ in.

The minimum acceptable width to accommodate the connection is:

$$2(4.00\text{ in.}) + 0.760\text{ in.} + 2(\tfrac{9}{16}\text{ in.}) = 9.89\text{ in.} < 11\tfrac{3}{4}\text{ in.} \quad \textbf{o.k.}$$

Available Beam Web Strength

The available beam web strength, from AISC *Manual* design table discussion for Table 10-1, is the lesser of the limit states of block shear rupture, shear yielding, shear rupture, and the sum of the effective strengths of the individual fasteners. The beam is not coped, so the only applicable limit state is the effective strength of the individual fasteners. The effective strength of an individual fastener is the lesser of the fastener shear strength, bearing strength at the bolt hole, and the tearout strength at the bolt hole.

For the limit state of fastener shear strength, with A_b = 0.442 in.^{2} from AISC *Manual* Table 7-1 for a ¾-in. bolt:

$$\begin{aligned} r_n &= F_{nv}A_b \\ &= (54\text{ ksi})(0.442\text{ in.}^2)(2\text{ shear planes}) \\ &= 47.7\text{ kips/bolt} \end{aligned} \qquad \text{(from } \textit{Spec.}\text{ Eq. J3-1)}$$

where F_{nv} is the nominal shear strength from AISC *Specification* Table J3.2 of a Group A bolt in a bearing-type connection when threads are not excluded from the shear planes.

Assume that deformation at the bolt hole at service load is a design consideration.

For the limit state of bearing:

$$\begin{aligned} r_n &= 2.4dtF_u \\ &= 2.4(\tfrac{3}{4}\text{ in.})(0.760\text{ in.})(65\text{ ksi}) \\ &= 88.9\text{ kips/bolt} \end{aligned} \qquad \text{(from } \textit{Spec.}\text{ Eq. J3-6a)}$$

For the limit state of tearout:

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$$
\begin{aligned}
r_n &= 1.2 l_c t F_u \qquad \text{(from } \textit{Spec}\text{. Eq. J3-6c)}\\
&= 1.2\left(3\text{ in.} - \tfrac{13}{16}\text{ in.}\right)\left(0.760\text{ in.}\right)\left(65\text{ ksi}\right)\\
&= 130\text{ kips/bolt}
\end{aligned}
$$

where l_c is the clear distance, in the direction of the force, between the edges of the bolt holes.

Fastener shear strength is the governing limit state for all bolts at the beam web. Fastener shear strength is one of the limit states included in the available strength given in Table 10-1 and was previously shown to be adequate.

EXAMPLE K.4 UNSTIFFENED SEATED CONNECTION TO AN HSS COLUMN

Given:

Use AISC *Manual* Table 10-6 to verify an unstiffened seated connection for an ASTM A992 W21×62 beam to an ASTM A500 Grade C HSS12×12×½ column, as shown in Figure K.4-1. The angles are ASTM A36 material. Use 70-ksi weld electrodes. Use the following vertical shear loads:

P_D = 9 kips
P_L = 27 kips

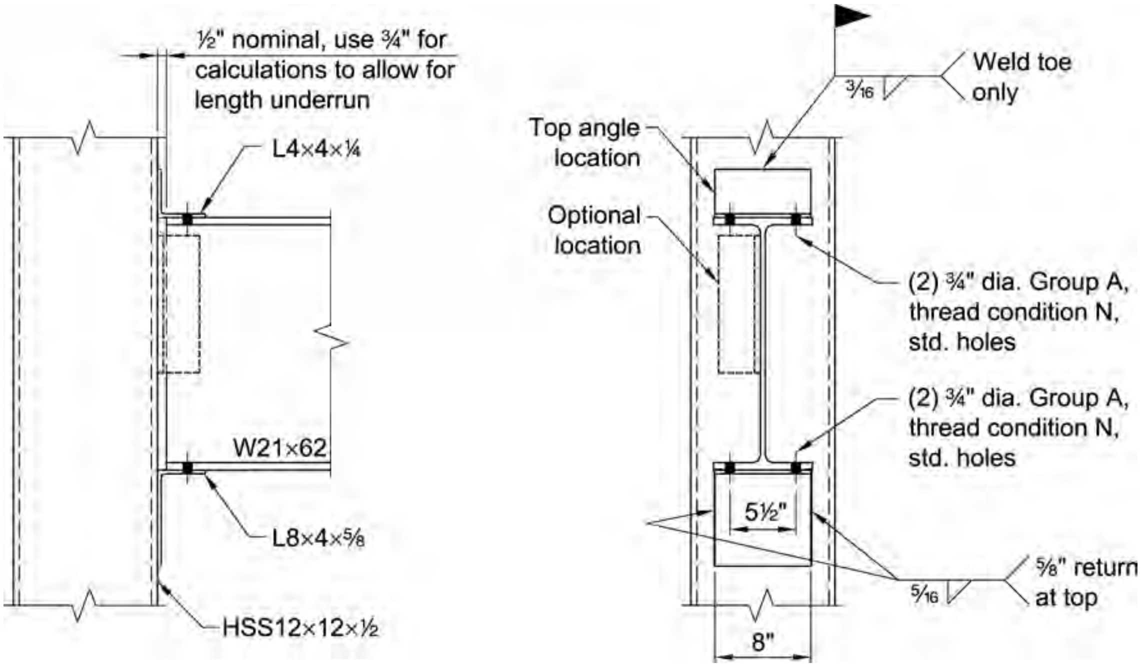

Fig K.4-1. Connection geometry for Example K.4.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Column
ASTM A500 Grade C
F_y = 50 ksi
F_u = 62 ksi

Angles
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Tables 1-1 and 1-12, the geometric properties are as follows:

W21×62
t_w = 0.400 in.
d = 21.0 in.
k_{des} = 1.12 in.

HSS12×12×½
t = 0.465 in.
B = 12.0 in.

From of ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(9\text{ kips}) + 1.6(27\text{ kips})$ $= 54.0\text{ kips}$	$R_a = 9\text{ kips} + 27\text{ kips}$ $= 36.0\text{ kips}$

Seat Angle and Weld Design

Check web local yielding of the W21×62 using AISC *Manual* Part 9.

LRFD	ASD
From AISC *Manual* Equation 9-46a and Table 9-4: $l_{b\ min} = \dfrac{R_u - \phi R_1}{\phi R_2} \geq k_{des}$ $= \dfrac{54.0\text{ kips} - 56.0\text{ kips}}{20.0\text{ kip/in.}}$ which results in a negative quantity. Use $l_{b\ min} = k_{des} = 1.12$ in. Check web local crippling when $l_b/d \leq 0.2$. From AISC *Manual* Equation 9-48a: $l_{b\ min} = \dfrac{R_u - \phi R_3}{\phi R_4}$ $= \dfrac{54.0\text{ kips} - 71.7\text{ kips}}{5.37\text{ kip/in.}}$ which results in a negative quantity. Check web local crippling when $l_b/d > 0.2$.	From AISC *Manual* Equation 9-46b and Table 9-4: $l_{b\ min} = \dfrac{R_a - R_1/\Omega}{R_2/\Omega} \geq k_{des}$ $= \dfrac{36.0\text{ kips} - 37.3\text{ kips}}{13.3\text{ kip/in.}}$ which results in a negative quantity. Use $l_{b\ min} = k_{des} = 1.12$ in. Check web local crippling when $l_b/d \leq 0.2$. From AISC *Manual* Equation 9-48b: $l_{b\ min} = \dfrac{R_a - R_3/\Omega}{R_4/\Omega}$ $= \dfrac{36.0\text{ kips} - 47.8\text{ kips}}{3.58\text{ kip/in.}}$ which results in a negative quantity. Check web local crippling when $l_b/d > 0.2$.

LRFD	ASD
From AISC *Manual* Equation 9-49a: $l_{b\ min} = \dfrac{R_u - \phi R_5}{\phi R_6}$ $= \dfrac{54.0\text{ kips} - 64.2\text{ kips}}{7.16\text{ kip/in.}}$ which results in a negative quantity.	From AISC *Manual* Equation 9-49b: $l_{b\ min} = \dfrac{R_a - R_5/\Omega}{R_6/\Omega}$ $= \dfrac{36.0\text{ kips} - 42.8\text{ kips}}{4.77\text{ kip/in.}}$ which results in a negative quantity.

Note: Generally, the value of l_b/d is not initially known and the larger value determined from the web local crippling equations in the preceding text can be used conservatively to determine the bearing length required for web local crippling.

For this beam and end reaction, the beam web available strength exceeds the required strength (hence the negative bearing lengths) and the lower-bound bearing length controls ($l_{b\ req} = k_{des} = 1.12$ in.). Thus, $l_{b\ min} = 1.12$ in.

Try an L8×4×⅝ seat with 5⁄16-in. fillet welds.

Outstanding Angle Leg Available Strength

From AISC *Manual* Table 10-6 for an 8-in. angle length and $l_{b\ req} = 1.12$ in. ≈ 1⅛ in., the outstanding angle leg available strength is:

LRFD	ASD
$\phi R_n = 81.0\text{ kips} > 54.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 53.9\text{ kips} > 36.0\text{ kips}$ **o.k.**

Available Weld Strength

From AISC *Manual* Table 10-6, for an 8 in. x 4 in. angle and 5⁄16-in. weld size, the available weld strength is:

LRFD	ASD
$\phi R_n = 66.7\text{ kips} > 54.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 44.5\text{ kips} > 36.0\text{ kips}$ **o.k.**

Minimum HSS Wall Thickness to Match Weld Strength

$$t_{min} = \frac{3.09D}{F_u} \qquad \text{(\textit{Manual} Eq. 9-2)}$$
$$= \frac{3.09(5)}{62\text{ ksi}}$$
$$= 0.249\text{ in.} < 0.465\text{ in.}$$

Because t of the HSS is greater than t_{min} for the 5⁄16-in. weld, no reduction in the weld strength is required to account for the shear in the HSS.

Connection to Beam and Top Angle (AISC Manual *Part 10)*

Use a L4×4×¼ top angle for stability. Use a 3⁄16-in. fillet weld across the toe of the angle for attachment to the HSS. Attach both the seat and top angles to the beam flanges with two ¾-in.-diameter Group A bolts.

EXAMPLE K.5 STIFFENED SEATED CONNECTION TO AN HSS COLUMN

Given:

Use AISC *Manual* Tables 10-8 and 10-15 to verify a stiffened seated connection for an ASTM A992 W21×68 beam to an ASTM A500 Grade C HSS14×14×½ column, as shown in Figure K.5-1. Use 70-ksi electrode welds to connect the stiffener, seat plate and top angle to the HSS. The angle and plate material are ASTM A36.
Use the following vertical shear loads:

P_D = 20 kips
P_L = 60 kips

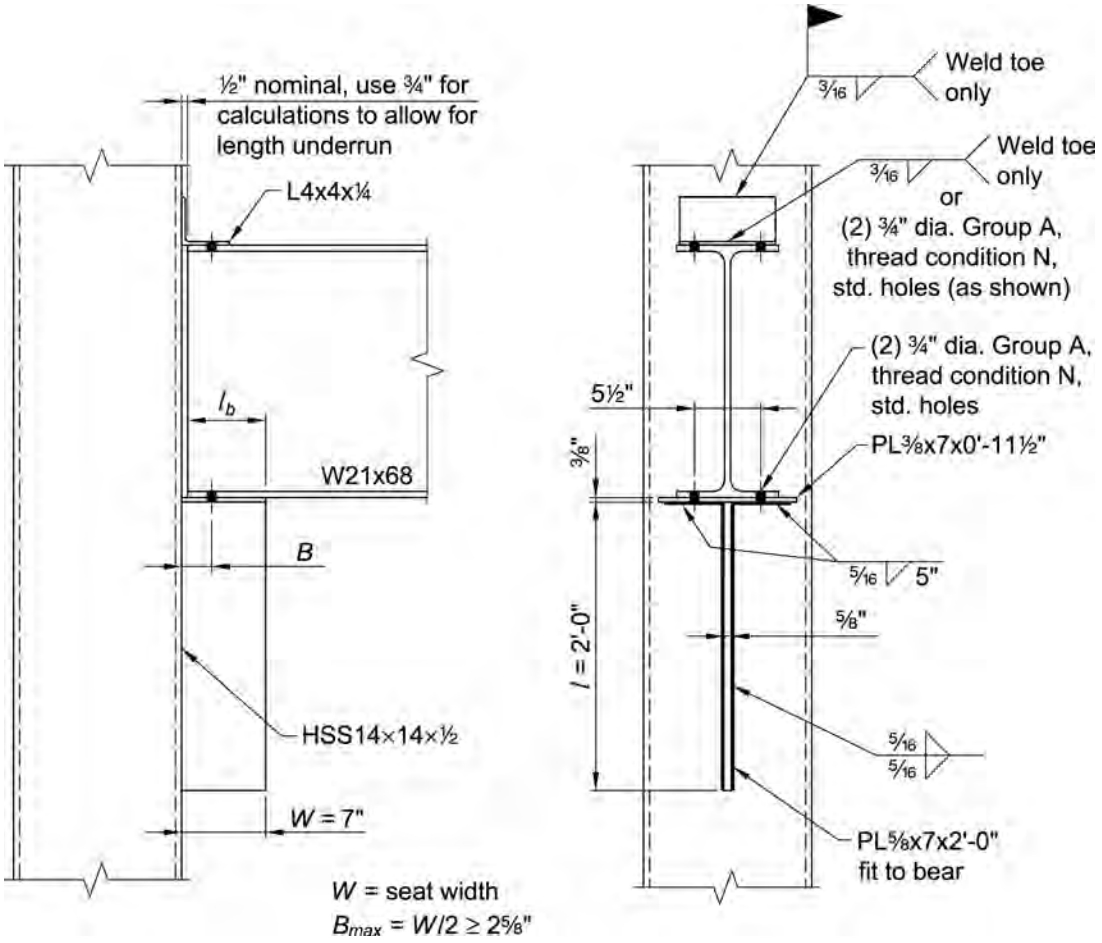

Fig K.5-1. Connection geometry for Example K.5.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Beam
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Column
ASTM A500 Grade C
F_y = 50 ksi
F_u = 62 ksi

Angles and Plates
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Tables 1-1 and 1-12, the geometric properties are as follows:

W21×68
t_w = 0.430 in.
d = 21.1 in.
k_{des} = 1.19 in.

HSS14×14×½
t = 0.465 in.
B = 14.0 in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$P_u = 1.2(20\text{ kips}) + 1.6(60\text{ kips})$ $= 120\text{ kips}$	$P_a = 20\text{ kips} + 60\text{ kips}$ $= 80.0\text{ kips}$

The available strength of connections to rectangular HSS with concentrated loads are determined based on the applicable limit states from Chapter J.

Stiffener Width, W, Required for Web Local Crippling and Web Local Yielding

The stiffener width is determined based on web local crippling and web local yielding of the beam, assuming a ¾-in. beam end setback in the calculations. Note that according to AISC *Specification* Section J10, the length of bearing, l_b, cannot be less than the beam k_{des}.

For web local crippling, assume $l_b/d > 0.2$ and use constants R_5 and R_6 from AISC *Manual* Table 9-4.

LRFD	ASD
From AISC *Manual* Equation 9-49a and Table 9-4: $W_{min} = \dfrac{R_u - \phi R_5}{\phi R_6} + \text{setback} \geq k_{des} + \text{setback}$ $= \dfrac{120\text{ kips} - 75.9\text{ kips}}{7.95\text{ kip/in.}} + \tfrac{3}{4}\text{ in.} \geq 1.19\text{ in.} + \tfrac{3}{4}\text{ in.}$ $= 6.30\text{ in.} > 1.94\text{ in.}$	From AISC *Manual* Equation 9-49b and Table 9-4: $W_{min} = \dfrac{R_a - R_5/\Omega}{R_6/\Omega} + \text{setback} \geq k_{des} + \text{setback}$ $= \dfrac{80.0\text{ kips} - 50.6\text{ kips}}{5.30\text{ kip/in.}} + \tfrac{3}{4}\text{ in.} \geq 1.19\text{ in.} + \tfrac{3}{4}\text{ in.}$ $= 6.30\text{ in.} > 1.94\text{ in.}$

For web local yielding, use constants R_1 and R_2 from AISC *Manual* Table 9-4.

LRFD	ASD
From AISC *Manual* Equation 9-46a and Table 9-4: $W_{min} = \dfrac{R_u - \phi R_1}{\phi R_2} + \text{setback} \geq k_{des} + \text{setback}$ $= \dfrac{120 \text{ kips} - 64.0 \text{ kips}}{21.5 \text{ kip/in.}} + \tfrac{3}{4} \text{ in.} \geq 1.19 \text{ in.} + \tfrac{3}{4} \text{ in.}$ $= 3.35 \text{ in.} > 1.94 \text{ in.}$	From AISC *Manual* Equation 9-46a and Table 9-4: $W_{min} = \dfrac{R_a - R_1/\Omega}{R_2/\Omega} + \text{setback} \geq k_{des} + \text{setback}$ $= \dfrac{80.0 \text{ kips} - 42.6 \text{ kips}}{14.3 \text{ kip/in.}} + \tfrac{3}{4} \text{ in.} \geq 1.19 \text{ in.} + \tfrac{3}{4} \text{ in.}$ $= 3.37 \text{ in.} > 1.94 \text{ in.}$

The minimum stiffener width, W_{min}, for web local crippling controls. The stiffener width of 7 in. is adequate.

Check the assumption that $l_b/d > 0.2$.

$$
\begin{aligned}
l_b &= 7 \text{ in.} - \tfrac{3}{4} \text{ in.} \\
&= 6.25 \text{ in.}
\end{aligned}
$$

$$
\begin{aligned}
\frac{l_b}{d} &= \frac{6.25 \text{ in.}}{21.1 \text{ in.}} \\
&= 0.296 > 0.2, \text{ as assumed}
\end{aligned}
$$

Weld Strength Requirements for the Seat Plate

Check the stiffener length, l = 24 in., with $\tfrac{5}{16}$-in. fillet welds. Enter AISC *Manual* Table 10-8, using W = 7 in. as verified in the preceding text.

LRFD	ASD
$\phi R_n = 293 \text{ kips} > 120 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 195 \text{ kips} > 80.0 \text{ kips}$ **o.k.**

From AISC *Manual* Part 10, Figure 10-10(b), the minimum length of the seat-plate-to-HSS weld on each side of the stiffener is 0.2l = 4.80 in. This establishes the minimum weld between the seat plate and stiffener. A 5-in.-long $\tfrac{5}{16}$-in. weld on each side of the stiffener is adequate.

Minimum HSS Wall Thickness to Match Weld Strength

The minimum HSS wall thickness required to match the shear rupture strength of the base metal to that of the weld is:

$$
\begin{aligned}
t_{min} &= \frac{3.09D}{F_u} \qquad \text{(\textit{Manual} Eq. 9-2)} \\
&= \frac{3.09(5)}{62 \text{ ksi}} \\
&= 0.249 \text{ in.} < 0.465 \text{ in.}
\end{aligned}
$$

Because t of the HSS is greater than t_{min} for the $\tfrac{5}{16}$-in. fillet weld, no reduction in the weld strength to account for shear in the HSS is required.

Stiffener Plate Thickness

From AISC *Manual* Part 10, Table 10-8 discussion, to develop the stiffener-to-seat-plate welds, the minimum stiffener thickness is:

$$\begin{aligned} t_{p\,min} &= 2w \\ &= 2(\tfrac{5}{16}\text{ in.}) \\ &= \tfrac{5}{8}\text{ in.} \end{aligned}$$

Also, from AISC *Manual* Part 10, Table 10-8 discussion, for a stiffener with F_y = 36 ksi and a beam with F_y = 50 ksi, the minimum stiffener thickness is:

$$\begin{aligned} t_{p\,min} &= \left(\frac{F_{y\,beam}}{F_{y\,stiffener}}\right)t_w \\ &= \left(\frac{50\text{ ksi}}{36\text{ ksi}}\right)(0.430\text{ in.}) \\ &= 0.597\text{ in.} \end{aligned}$$

The stiffener thickness of ⅝ in. is adequate.

Determine the stiffener length using AISC *Manual* Table 10-15.

The required HSS wall strength factor is:

LRFD	ASD
$\left(\frac{R_uW}{t^2}\right)_{req} = \frac{(120\text{ kips})(7\text{ in.})}{(0.465\text{ in.})^2}$ $= 3{,}880\text{ kip/in.}$	$\left(\frac{R_aW}{t^2}\right)_{req} = \frac{(80.0\text{ kips})(7\text{ in.})}{(0.465\text{ in.})^2}$ $= 2{,}590\text{ kip/in.}$

To satisfy the minimum, select a stiffener with l = 24 in. from AISC *Manual* Table 10-15. The HSS wall strength factor is:

LRFD	ASD
$\frac{R_uW}{t^2} = 3{,}910\text{ kip/in.} > 3{,}880\text{ kip/in.}$ **o.k.**	$\frac{R_aW}{t^2} = 2{,}600\text{ kip/in.} > 2{,}590\text{ kip/in.}$ **o.k.**

Use PL⅝ in.×7 in. × 2 ft 0 in. for the stiffener.

HSS Width Check

The minimum width is $0.4l + t_p + 2(2.25t)$; however, because the specified weld length of 5 in. on each side of the stiffener is greater than $0.4l$, the weld length will be used. The nominal wall thickness, t_{nom}, is used, as would be used to calculate a workable flat dimension.

$$\begin{aligned} B &= 14.0\text{ in.} > (2\text{ welds})(5.00\text{ in.}) + \tfrac{5}{8}\text{ in.} + 2(2.25)(\tfrac{1}{2}\text{ in.}) \\ &= 14.0\text{ in.} > 12.9\text{ in.} \quad \textbf{o.k.} \end{aligned}$$

Seat Plate Dimensions

To accommodate two ¾-in.-diameter Group A bolts on a 5½-in. gage connecting the beam flange to the seat plate, a minimum width of 8 in. is required. To accommodate the seat-plate-to-HSS weld, the required width is:

$$2(5.00\text{ in.}) + \tfrac{5}{8}\text{ in.} = 10.6\text{ in.}$$

Note: To allow room to start and stop welds, an 11.5 in. width is used.

Use PL⅜ in.×7 in.× 0 ft-11½ in. for the seat plate.

Top Angle, Bolts and Welds (AISC Manual *Part 10)*

The minimum weld size for the HSS thickness according to AISC *Specification* Table J2.4 is 3⁄16 in. The angle thickness should be 1⁄16 in. larger.

Use L4×4×¼ with 3⁄16-in. fillet welds along the toes of the angle to the beam flange and HSS for stability. Alternatively, two ¾-in.-diameter Group A bolts may be used to connect the leg of the angle to the beam flange.

EXAMPLE K.6 SINGLE-PLATE CONNECTION TO A RECTANGULAR HSS COLUMN

Given:

Use AISC *Manual* Table 10-10a to verify the design of a single-plate connection for an ASTM A992 W18×35 beam framing into an ASTM A500 Grade C HSS6×6×⅜ column, as shown in Figure K.6-1. Use 70-ksi weld electrodes. The plate material is ASTM A36. Use the following vertical shear loads:

P_D = 6.5 kips
P_L = 19.5 kips

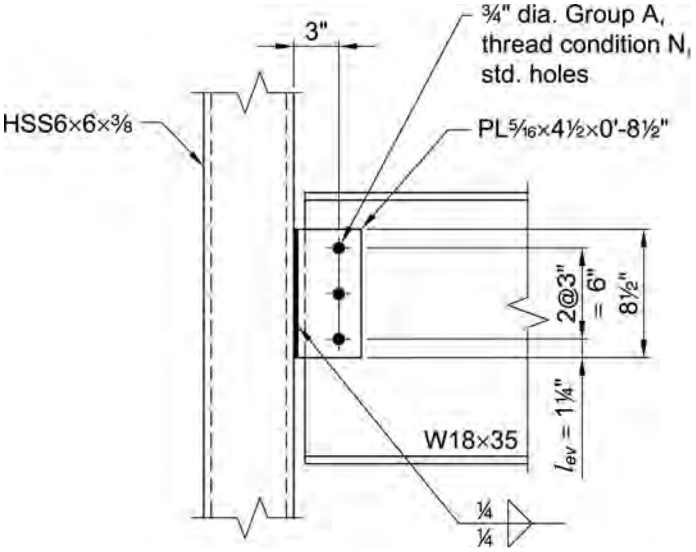

Fig K.6-1. Connection geometry for Example K.6.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Beam
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Column
ASTM A500 Grade C
F_y = 50 ksi
F_u = 62 ksi

Plate
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Tables 1-1 and 1-12, the geometric properties are as follows:

W18×35
d = 17.7 in.
t_w = 0.300 in.
T = 15½ in.

HSS6×6×⅜
$B = H = 6.00$ in.
t = 0.349 in.
b/t = 14.2

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(6.5\text{ kips}) + 1.6(19.5\text{ kips})$ $= 39.0\text{ kips}$	$R_a = 6.5\text{ kips} + 19.5\text{ kips}$ $= 26.0\text{ kips}$

Single-Plate Connection

As discussed in AISC *Manual* Part 10, a single-plate connection may be used as long as the HSS wall is not classified as a slender element.

$$\frac{b}{t} \le 1.40\sqrt{\frac{E}{F_y}}$$

$$14.2 \le 1.40\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}$$

$$14.2 < 33.7$$

Therefore, the HSS wall is not slender.

The available strength of the face of the HSS for the limit state of punching shear is determined from AISC *Manual* Part 10 as follows:

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$R_u e \le \dfrac{\phi F_u t l_p^{\,2}}{5}$ (*Manual* Eq. 10-7a)	$R_a e \le \dfrac{F_u t l_p^{\,2}}{5\Omega}$ (*Manual* Eq. 10-7b)
$(39.0\text{ kips})(3\text{ in.}) \le \dfrac{0.75(62\text{ ksi})(0.349\text{ in.})(8.50\text{ in.})^2}{5}$	$(26.0\text{ kips})(3\text{ in.}) \le \dfrac{(62\text{ ksi})(0.349\text{ in.})(8.50\text{ in.})^2}{5(2.00)}$
$117\text{ kip-in.} < 235\text{ kip-in.}$ **o.k.**	$78.0\text{ kip-in.} < 156\text{ kip-in.}$ **o.k.**

Try three rows of bolts and a 5⁄16-in. plate thickness with ¼-in. fillet welds. From AISC *Manual* Table 10-9, either the plate or the beam web must satisfy:

$$t \le \frac{d}{2} + \tfrac{1}{16}\text{ in.}$$

$$\tfrac{5}{16}\text{ in.} \le \frac{\tfrac{3}{4}\text{ in.}}{2} + \tfrac{1}{16}\text{ in.}$$

$$\tfrac{5}{16}\text{ in.} < 0.438\text{ in.} \quad \textbf{o.k.}$$

Obtain the available single-plate connection strength from AISC *Manual* Table 10-10a:

LRFD	ASD
$\phi R_n = 44.2 \text{ kips} > 39.0 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 29.4 \text{ kips} > 26.0 \text{ kips}$ **o.k.**

Use a PL5⁄16 in.×4½ in. × 0 ft 8½ in.

HSS Shear Rupture at Welds

The minimum HSS wall thickness required to match the shear rupture strength of the HSS wall to that of the weld is:

$$\begin{aligned} t_{min} &= \frac{3.09D}{F_u} \qquad \text{(\textit{Manual} Eq. 9-2)} \\ &= \frac{3.09(4)}{62 \text{ ksi}} \\ &= 0.199 \text{ in.} < t = 0.349 \text{ in.} \quad \textbf{o.k.} \end{aligned}$$

Available Beam Web Strength

The available beam web strength is the lesser of the limit states of block shear rupture, shear yielding, shear rupture, and the sum of the effective strengths of the individual fasteners. The beam is not coped, so the only applicable limit state is the effective strength of the individual fasteners. The effective strength of an individual fastener is the lesser of the fastener shear strength, the bearing strength at the bolt hole and the tearout strength at the bolt hole.

For the limit state of fastener shear strength, with A_b = 0.442 in.^{2} from AISC *Manual* Table 7-1 for a ¾-in. bolt.:

$$\begin{aligned} r_n &= F_{nv} A_b \qquad \text{(from \textit{Spec}. Eq. J3-1)} \\ &= (54 \text{ ksi})(0.442 \text{ in.}^2) \\ &= 23.9 \text{ kips/bolt} \end{aligned}$$

where F_{nv} is the nominal shear strength of a Group A bolt in a bearing-type connection when threads are not excluded from the shear plane as found in AISC *Specification* Table J3.2.

Assume that deformation at the bolt hole at service load is a design consideration.

For the limit state of bearing:

$$\begin{aligned} r_n &= 2.4dtF_u \qquad \text{(from \textit{Spec}. Eq. J3-6a)} \\ &= 2.4(\tfrac{3}{4} \text{ in.})(0.300 \text{ in.})(65 \text{ ksi}) \\ &= 35.1 \text{ kips/bolt} \end{aligned}$$

For the limit state of tearout:

$$\begin{aligned} r_n &= 1.2 l_c t F_u \qquad \text{(from \textit{Spec}. Eq. J3-6c)} \\ &= 1.2(3 \text{ in.} - \tfrac{13}{16} \text{ in.})(0.300 \text{ in.})(65 \text{ ksi}) \\ &= 51.2 \text{ kips/bolt} \end{aligned}$$

where l_c is the clear distance, in the direction of the force, between the edges of the bolt holes.

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Fastener shear strength is the governing limit state for all bolts at the beam web. Fastener shear strength is one of the limit states included in the available strengths given in Table 10-10a and used in the preceding calculations. Thus, the effective strength of the fasteners is adequate.

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EXAMPLE K.7 THROUGH-PLATE CONNECTION TO A RECTANGULAR HSS COLUMN

Given:

Use AISC *Manual* Table 10-10a to verify a through-plate connection between an ASTM A992 W18×35 beam and an ASTM A500 Grade C HSS6×4×⅛ with the connection to one of the 6 in. faces, as shown in Figure K.7-1. A thin-walled column is used to illustrate the design of a through-plate connection. Use 70-ksi weld electrodes. The plate is ASTM A36 material. Use the following vertical shear loads:

P_D = 3.3 kips
P_L = 9.9 kips

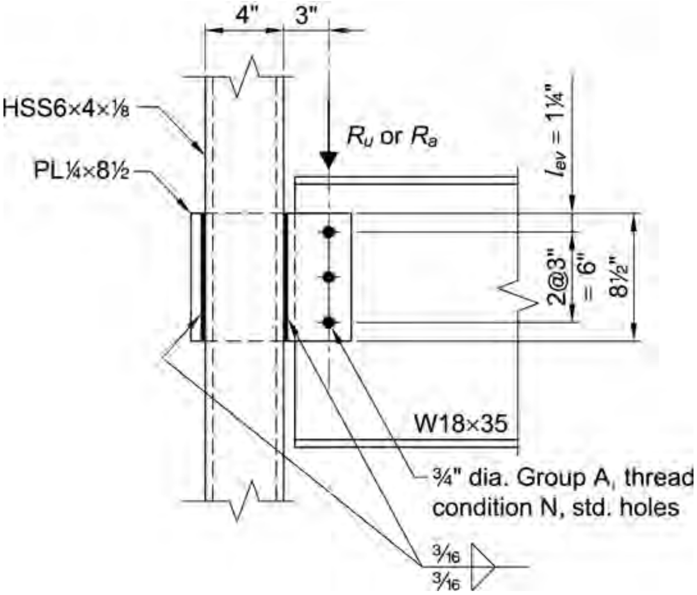

Fig K.7-1. Connection geometry for Example K.7.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Beam
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Column
ASTM A500 Grade C
F_y = 50 ksi
F_u = 62 ksi

Plate
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Tables 1-1 and 1-11, the geometric properties are as follows:

W18×35
d = 17.7 in.
t_w = 0.300 in.
T = 15½ in.

HSS6×4×⅛
B = 4.00 in.
H = 6.00 in.
t = 0.116 in.
h/t = 48.7
b/t = 31.5

HSS wall slenderness

From AISC *Manual* Part 10, the limiting width-to-thickness for a nonslender HSS wall is:

$$\begin{aligned} 1.40\sqrt{\frac{E}{F_y}} &= 1.40\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} \\ &= 33.7 \end{aligned}$$

Because $h/t = 48.7 > 33.7$, the HSS6×4×⅛ is slender and a through-plate connection should be used instead of a single-plate connection. Through-plate connections are typically very expensive. When a single-plate connection is not adequate, another type of connection, such as a double-angle connection may be preferable to a through-plate connection.

AISC *Specification* Chapter K does not contain provisions for the design of through-plate shear connections. The following procedure treats the connection of the through-plate to the beam as a single-plate connection.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$\begin{aligned} R_u &= 1.2(3.3\text{ kips}) + 1.6(9.9\text{ kips}) \\ &= 19.8\text{ kips} \end{aligned}$	$\begin{aligned} R_a &= 3.3\text{ kips} + 9.9\text{ kips} \\ &= 13.2\text{ kips} \end{aligned}$

Portion of the Through-Plate Connection that Resembles a Single-Plate

Try three rows of bolts (l = 8½ in.) and a ¼-in. plate thickness with $^3\!/_{16}$-in. fillet welds.

$$\begin{aligned} \frac{T}{2} &= \frac{15\tfrac{1}{2}\text{ in.}}{2} \\ &= 7.75\text{ in.} < l = 8\tfrac{1}{2}\text{ in.} \quad \textbf{o.k.} \end{aligned}$$

Note: From AISC *Manual* Table 10-9, the larger of the plate thickness or the beam web thickness must satisfy:

$$t \le \frac{d}{2} + \tfrac{1}{16}\text{ in.}$$

$$\tfrac{1}{4}\text{ in.} \le \frac{\tfrac{3}{4}\text{ in.}}{2} + \tfrac{1}{16}\text{ in.}$$

$$\tfrac{1}{4}\text{ in.} < 0.438\text{ in.} \quad \textbf{o.k.}$$

Obtain the available single-plate connection strength from AISC *Manual* Table 10-10a:

LRFD	ASD
$\phi R_n = 38.3 \text{ kips} > 19.8 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 25.6 \text{ kips} > 13.2 \text{ kips}$ **o.k.**

Required Weld Strength

The available strength for the welds in this connection is checked at the location of the maximum reaction, which is along the weld line closest to the bolt line. The reaction at this weld line is determined by taking a moment about the weld line farthest from the bolt line.

a = 3 in. (distance from bolt line to nearest weld line)

LRFD	ASD
$V_{fu} = \dfrac{R_u\left(B+a\right)}{B}$ $= \dfrac{\left(19.8 \text{ kips}\right)\left(4.00 \text{ in.} + 3 \text{ in.}\right)}{4.00 \text{ in.}}$ $= 34.7 \text{ kips}$	$V_{fa} = \dfrac{R_a\left(B+a\right)}{B}$ $= \dfrac{\left(13.2 \text{ kips}\right)\left(4.00 \text{ in.} + 3 \text{ in.}\right)}{4.00 \text{ in.}}$ $= 23.1 \text{ kips}$

Available Weld Strength

The minimum required weld size is determined using AISC *Manual* Part 8.

LRFD	ASD
$D_{req} = \dfrac{V_{fu}}{1.392l}$ (from *Manual* Eq. 8-2a) $= \dfrac{34.7 \text{ kips}}{\left(1.392 \text{ kip/in.}\right)\left(8.50 \text{ in.}\right)\left(2\right)}$ $= 1.47 \text{ sixteenths} < 3 \text{ sixteenths}$ **o.k.**	$D_{req} = \dfrac{V_{fa}}{0.928l}$ (from *Manual* Eq. 8-2b) $= \dfrac{23.1 \text{ kips}}{\left(0.928 \text{ kip/in.}\right)\left(8.50 \text{ in.}\right)\left(2\right)}$ $= 1.46 \text{ sixteenths} < 3 \text{ sixteenths}$ **o.k.**

HSS Shear Yielding and Rupture Strength

The available shear yielding strength of the HSS is determined from AISC *Specification* Section J4.2.

LRFD	ASD
$\phi = 1.00$ $\phi R_n = \phi 0.60 F_y A_{gv}$ (from *Spec.* Eq. J4-3) $= 1.00\left(0.60\right)\left(50 \text{ ksi}\right)\left(0.116 \text{ in.}\right)\left(8.50 \text{ in.}\right)\left(2\right)$ $= 59.2 \text{ kips} > 34.7 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{0.60 F_y A_{gv}}{\Omega}$ (from *Spec.* Eq. J4-3) $= \dfrac{\left(0.60\right)\left(50 \text{ ksi}\right)\left(0.116 \text{ in.}\right)\left(8.50 \text{ in.}\right)\left(2\right)}{1.50}$ $= 39.4 \text{ kips} > 23.1 \text{ kips}$ **o.k.**

The available shear rupture strength of the HSS is determined from AISC *Specification* Section J4.2.

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LRFD	ASD
$\phi = 0.75$ $\phi R_n = \phi 0.60 F_u A_{nv}$ (from *Spec*. Eq. J4-4) $= 0.75(0.60)(62\text{ ksi})(0.116\text{ in.})(8.50\text{ in.})(2)$ $= 55.0\text{ kips} > 34.7\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{0.60 F_u A_{nv}}{\Omega}$ (from *Spec*. Eq. J4-4) $= \dfrac{(0.60)(62\text{ ksi})(0.116\text{ in.})(8.50\text{ in.})(2)}{2.00}$ $= 36.7\text{ kips} > 23.1\text{ kips}$ **o.k.**

Available Beam Web Strength

The available beam web strength is the lesser of the limit states of block shear rupture, shear yielding, shear rupture, and the sum of the effective strengths of the individual fasteners. The beam is not coped, so the only applicable limit state is the effective strength of the individual fasteners. The effective strength of an individual fastener is the lesser of the fastener shear strength, the bearing strength at the bolt hole and the tearout strength at the bolt hole.

For the limit state of fastener shear strength, with $A_b = 0.442\text{ in.}^2$ from AISC *Manual* Table 7-1 for a ¾-in. bolt:

$$
\begin{aligned}
r_n &= F_{nv} A_b \qquad \text{(from \textit{Spec}. Eq. J3-1)}\\
&= (54\text{ ksi})(0.442\text{ in.}^2)\\
&= 23.9\text{ kips/bolt}
\end{aligned}
$$

where F_{nv} is the nominal shear strength of a Group A bolt in a bearing-type connection when threads are not excluded from the shear planes as found in AISC *Specification* Table J3.2.

Assume that deformation at the bolt hole at service load is a design consideration.

For the limit state of bearing:

$$
\begin{aligned}
r_n &= 2.4 d t F_u \qquad \text{(from \textit{Spec}. Eq. J3-6a)}\\
&= 2.4(\tfrac{3}{4}\text{ in.})(0.300\text{ in.})(65\text{ ksi})\\
&= 35.1\text{ kips/bolt}
\end{aligned}
$$

For the limit state of tearout:

$$
\begin{aligned}
r_n &= 1.2 l_c t F_u \qquad \text{(from \textit{Spec}. Eq. J3-6c)}\\
&= 1.2(3\text{ in.} - \tfrac{13}{16}\text{ in.})(0.300\text{ in.})(65\text{ ksi})\\
&= 51.2\text{ kips/bolt}
\end{aligned}
$$

where l_c is the clear distance, in the direction of the force, between the edges of the bolt holes.

Fastener shear strength is the governing limit state for all bolts at the beam web. Fastener shear strength is one of the limit states included in the available strengths shown in Table 10-10a as used in the preceding calculations. Thus, the effective strength of the fasteners is adequate.

EXAMPLE K.8 LONGITUDINAL PLATE LOADED PERPENDICULAR TO THE HSS AXIS ON A ROUND HSS

Given:

Verify the local strength of the ASTM A500 Grade C HSS6.000×0.375 tension chord subject to transverse loads, P_D = 4 kips and P_L = 12 kips, applied through an ASTM A36 plate, as shown in Figure K.8-1.

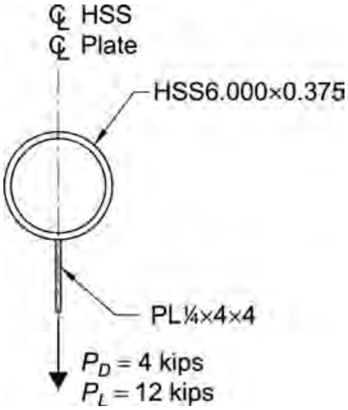

Fig K.8-1. Loading and geometry for Example K.8.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Chord
ASTM A500 Grade C
F_y = 46 ksi
F_u = 62 ksi

Plate
ASTM A36
F_{yp} = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-13, the geometric properties are as follows:

HSS6.000×0.375
D = 6.00 in.
t = 0.349 in.
D/t = 17.2

Limits of Applicability of AISC Specification *Section K2.2, Table K2.1A*

AISC *Specification* Table K2.1A provides the limits of applicability for plate-to-round connections. The applicable limits for this example are:

HSS wall slenderness:

$D/t \leq 50$ for T-connections

$17.2 < 50$ **o.k.**

Material strength:

$$F_y \le 52 \text{ ksi}$$

$$46 \text{ ksi} < 52 \text{ ksi} \quad \textbf{o.k.}$$

Ductility:

$$\frac{F_y}{F_u} \le 0.8$$

$$\frac{46 \text{ ksi}}{62 \text{ ksi}} \le 0.8$$

$$0.741 < 0.8 \quad \textbf{o.k.}$$

End distance:

$$l_{end} \ge D\left(1.25 - \frac{B_b/D}{2}\right)$$

$$= (6.00 \text{ in.})\left[1.25 - \frac{(\tfrac{1}{4} \text{ in.}/6.00 \text{ in.})}{2}\right]$$

$$= 7.38 \text{ in.}$$

Thus, the edge of the plate must be located a minimum of 7.38 in. from the end of the HSS.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$P_u = 1.2(4 \text{ kips}) + 1.6(12 \text{ kips})$ $= 24.0 \text{ kips}$	$P_a = 4 \text{ kips} + 12 \text{ kips}$ $= 16.0 \text{ kips}$

HSS Plastification Limit State

The limit state of HSS plastification applies and is determined from AISC *Specification* Table K2.1.

$$R_n \sin\theta = 5.5F_y t^2\left(1 + 0.25\frac{l_b}{D}\right)Q_f \qquad (\textit{Spec}.\text{ Eq. K2-2a})$$

From the AISC *Specification* Table K2.1 Functions listed at the bottom of the table, for an HSS connecting surface in tension, $Q_f = 1.0$.

$$R_n = \frac{5.5(46 \text{ ksi})(0.349 \text{ in.})^2\left[1 + 0.25\left(\dfrac{4 \text{ in.}}{6.00 \text{ in.}}\right)\right](1.0)}{\sin 90^\circ}$$

$$= 36.0 \text{ kips}$$

The available strength is:

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(36.0 \text{ kips})$ $= 32.4 \text{ kips} > 24.0 \text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{36.0 \text{ kips}}{1.67}$ $= 21.6 \text{ kips} > 16.0 \text{ kips}$ **o.k.**

EXAMPLE K.9 RECTANGULAR HSS COLUMN BASE PLATE

Given:

An ASTM A500 Grade C HSS6×6×½ column is supporting loads of 40 kips of dead load and 120 kips of live load. The column is supported by a 7 ft 6 in. × 7 ft 6 in. concrete spread footing with f_c' = 3,000 psi. Verify the ASTM A36 base plate size shown in Figure K.9-1 for this column.

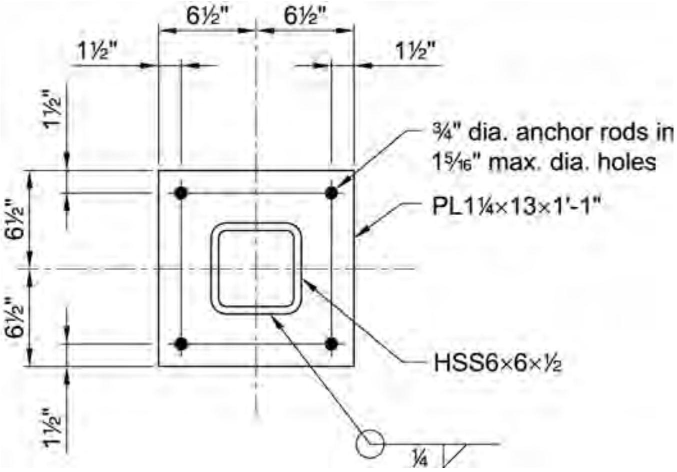

Fig K.9-1. Base plate geometry for Example K.9.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Column
ASTM A500 Grade C
F_y = 50 ksi
F_u = 62 ksi

Base Plate
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-12, the geometric properties are as follows:

HSS6×6×½
$B = H = 6.00$ in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$P_u = 1.2(40 \text{ kips}) + 1.6(120 \text{ kips})$ $= 240 \text{ kips}$	$P_a = 40 \text{ kips} + 120 \text{ kips}$ $= 160 \text{ kips}$

Note: The procedure illustrated here is similar to that presented in AISC Design Guide 1, *Base Plate and Anchor Rod Design* (Fisher and Kloiber, 2006), and AISC *Manual* Part 14.

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Try a base plate which extends 3½ in. from each face of the HSS column, or 13 in. × 13 in.

Available Strength for the Limit State of Concrete Crushing

On less than the full area of a concrete support:

$$P_p = 0.85 f_c' A_1 \sqrt{A_2/A_1} \;\le\; 1.7 f_c' A_1 \qquad \text{(}\textit{Spec}\text{. Eq. J8-2)}$$

$$\begin{aligned} A_1 &= BN \\ &= (13 \text{ in.})(13 \text{ in.}) \\ &= 169 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_2 &= \left[(7.5 \text{ ft})(12 \text{ in./ft})\right]^2 \\ &= 8{,}100 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} P_p &= 0.85(3 \text{ ksi})\left(169 \text{ in.}^2\right)\sqrt{\frac{8{,}100 \text{ in.}^2}{169 \text{ in.}^2}} \le 1.7(3 \text{ ksi})\left(169 \text{ in.}^2\right) \\ &= 2{,}980 \text{ kips} > 862 \text{ kips} \end{aligned}$$

Use P_p = 862 kips.

Note: The limit on the right side of AISC *Specification* Equation J8-2 will control when A_2/A_1 exceeds 4.0.

LRFD	ASD
From AISC *Specification* Section J8: $\phi_c = 0.65$ $\phi_c P_p = 0.65(862 \text{ kips})$ $= 560 \text{ kips} > 240 \text{ kips}$ **o.k.**	From AISC *Specification* Section J8: $\Omega_c = 2.31$ $\dfrac{P_p}{\Omega_c} = \dfrac{862 \text{ kips}}{2.31}$ $= 373 \text{ kips} > 160 \text{ kips}$ **o.k.**

Pressure under Bearing Plate and Required Thickness

For a rectangular HSS, the distance *m* or *n* is determined using 0.95 times the depth and width of the HSS.

$$\begin{aligned} m &= n \qquad \text{(from }\textit{Manual}\text{ Eq. 14-2)} \\ &= \frac{N - 0.95(B \text{ or } H)}{2} \\ &= \frac{13 \text{ in.} - 0.95(6.00 \text{ in.})}{2} \\ &= 3.65 \text{ in.} \end{aligned}$$

Note: As discussed in AISC Design Guide 1, the $\lambda n'$ cantilever distance is not used for HSS and pipe.

The critical bending moment is the cantilever moment outside the HSS perimeter. Therefore, $m = n = l$.

LRFD	ASD
$f_{pu} = \frac{P_u}{A_1}$ $= \frac{240 \text{ kips}}{169 \text{ in.}^2}$ $= 1.42 \text{ ksi}$ $M_u = \frac{f_{pu} l^2}{2}$ $Z = \frac{t_p^{\,2}}{4}$ $\phi_b = 0.90$ $M_n = M_p = F_y Z$ (from *Spec*. Eq. F11-1) Note: the upper limit of $1.6F_yS_x$ will not govern for a rectangular plate. Equating: $M_u = \phi_b M_n$ and solving for t_p gives: $t_{p(req)} = \sqrt{\frac{2 f_{pu} l^2}{\phi_b F_y}}$ $= \sqrt{\frac{2(1.42 \text{ ksi})(3.65 \text{ in.})^2}{0.90(36 \text{ ksi})}}$ $= 1.08 \text{ in.}$ Or use AISC *Manual* Equation 14-7a: $t_{min} = l\sqrt{\frac{2P_u}{0.90 F_y BN}}$ $= (3.65 \text{ in.})\sqrt{\frac{2(240 \text{ kips})}{0.90(36 \text{ ksi})(13 \text{ in.})(13 \text{ in})}}$ $= 1.08 \text{ in.}$	$f_{pa} = \frac{P_a}{A_1}$ $= \frac{160 \text{ kips}}{169 \text{ in.}^2}$ $= 0.947 \text{ ksi}$ $M_a = \frac{f_{pa} l^2}{2}$ $Z = \frac{t_p^{\,2}}{4}$ $\Omega_b = 1.67$ $M_n = M_p = F_y Z$ (from *Spec*. Eq. F11-1) Note: the upper limit of $1.6F_yS_x$ will not govern for a rectangular plate. Equating: $M_a = M_n/\Omega_b$ and solving for t_p gives: $t_{p(req)} = \sqrt{\frac{2 f_{pa} l^2}{F_y / \Omega_b}}$ $= \sqrt{\frac{2(0.947 \text{ ksi})(3.65 \text{ in.})^2}{(36 \text{ ksi})/1.67}}$ $= 1.08 \text{ in.}$ Or use AISC *Manual* Equation 14-7b: $t_{min} = l\sqrt{\frac{1.67(2P_a)}{F_y BN}}$ $= (3.65 \text{ in.})\sqrt{\frac{1.67(2)(160 \text{ kips})}{(36 \text{ ksi})(13 \text{ in.})(13 \text{ in.})}}$ $= 1.08 \text{ in.}$

Therefore, the PL1¼ in. × 13 in. × 1 ft 1 in. is adequate.

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EXAMPLE K.10 RECTANGULAR HSS STRUT END PLATE

Given:

Determine the weld leg size, end-plate thickness, and the bolt size required to resist forces of 16 kips from dead load and 50 kips from live load on an ASTM A500 Grade C section, as shown in Figure K.10-1. The end plate is ASTM A36. Use 70-ksi weld electrodes.

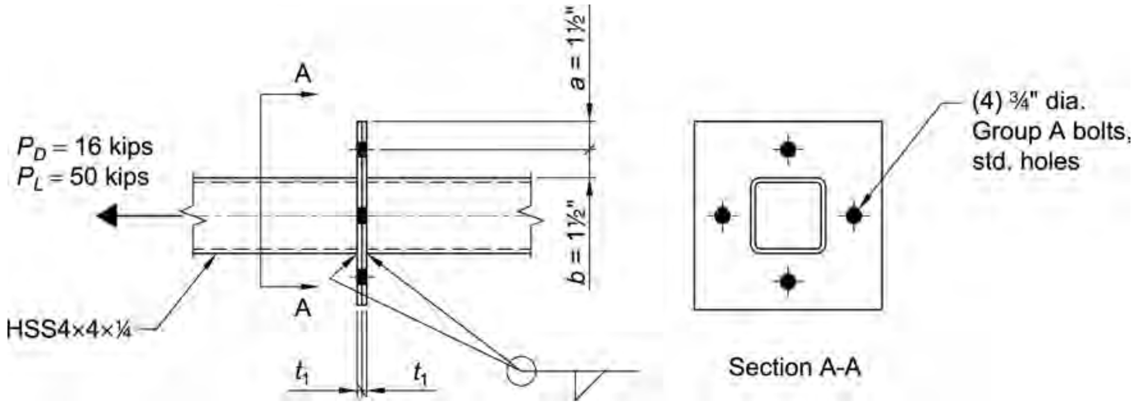

Fig K.10-1. Loading and geometry for Example K.10.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Strut
ASTM A500 Grade C
F_y = 50 ksi
F_u = 62 ksi

End Plate
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-12, the geometric properties are as follows:

HSS4×4×¼
t = 0.233 in.
A = 3.37 in.^{2}

From ASCE/SEI 7, Chapter 2, the required tensile strength is:

LRFD	ASD
$P_u = 1.2(16\text{ kips}) + 1.6(50\text{ kips})$ $= 99.2\text{ kips}$	$P_a = 16\text{ kips} + 50\text{ kips}$ $= 66.0\text{ kips}$

Preliminary Size of the (4) Group A Bolts

LRFD	ASD
$r_{ut} = \dfrac{P_u}{n}$ $= \dfrac{99.2 \text{ kips}}{4}$ $= 24.8 \text{ kips}$ Using AISC *Manual* Table 7-2, try ¾-in.-diameter Group A bolts. $\phi r_n = 29.8 \text{ kips}$	$r_{at} = \dfrac{P_a}{n}$ $= \dfrac{66.0 \text{ kips}}{4}$ $= 16.5 \text{ kips}$ Using AISC *Manual* Table 7-2, try ¾-in.-diameter Group A bolts. $\dfrac{r_n}{\Omega} = 19.9 \text{ kips}$

End-Plate Thickness with Consideration of Prying Action (AISC Manual *Part 9)*

$$a' = \left(a + \frac{d_b}{2}\right) \leq \left(1.25b + \frac{d_b}{2}\right) \qquad \text{(\textit{Manual} Eq. 9-23)}$$

$$= 1\tfrac{1}{2} \text{ in.} + \frac{\tfrac{3}{4} \text{ in.}}{2} \leq 1.25\left(1\tfrac{1}{2} \text{ in.}\right) + \frac{\tfrac{3}{4} \text{ in.}}{2}$$

$$= 1.88 \text{ in.} < 2.25 \text{ in.}$$

$$= 1.88 \text{ in.}$$

$$b' = b - \frac{d_b}{2} \qquad \text{(\textit{Manual} Eq. 9-18)}$$

$$= 1\tfrac{1}{2} \text{ in.} - \frac{\tfrac{3}{4} \text{ in.}}{2}$$

$$= 1.13 \text{ in.}$$

$$\rho = \frac{b'}{a'} \qquad \text{(\textit{Manual} Eq. 9-22)}$$

$$= \frac{1.13}{1.88}$$

$$= 0.601$$

$$d' = \tfrac{13}{16} \text{ in.}$$

The tributary length per bolt (Packer et al., 2010),

$$p = \frac{\text{full plate width}}{\text{number of bolts per side}}$$

$$= \frac{10.0 \text{ in.}}{1}$$

$$= 10.0 \text{ in.}$$

$$\delta = 1 - \frac{d'}{p} \qquad \text{(\textit{Manual} Eq. 9-20)}$$

$$= 1 - \frac{^{13}\!/_{16}\text{ in.}}{10.0\text{ in.}}$$

$$= 0.919$$

LRFD	ASD
$\beta = \dfrac{1}{\rho}\left(\dfrac{\phi r_n}{r_{ut}} - 1\right)$ (from *Manual* Eq. 9-21) $= \dfrac{1}{0.601}\left(\dfrac{29.8\text{ kips}}{24.8\text{ kips}} - 1\right)$ $= 0.335$ Because β < 1, from AISC *Manual* Part 9: $\alpha' = \dfrac{1}{\delta}\left(\dfrac{\beta}{1-\beta}\right) \le 1.0$ $= \dfrac{1}{0.919}\left(\dfrac{0.335}{1-0.335}\right) \le 1.0$ $= 0.548$	$\beta = \dfrac{1}{\rho}\left(\dfrac{r_n/\Omega}{r_{at}} - 1\right)$ (from *Manual* Eq. 9-21) $= \dfrac{1}{0.601}\left(\dfrac{19.9\text{ kips}}{16.5\text{ kips}} - 1\right)$ $= 0.343$ Because β < 1, from AISC *Manual* Part 9: $\alpha' = \dfrac{1}{\delta}\left(\dfrac{\beta}{1-\beta}\right) \le 1.0$ $= \dfrac{1}{0.919}\left(\dfrac{0.343}{1-0.343}\right) \le 1.0$ $= 0.568$

Use Equation 9-19 for t_{min} in Chapter 9 of the AISC *Manual*, except that F_u is replaced by F_y per the recommendation of Willibald, Packer and Puthli (2003) and Packer et al. (2010).

LRFD	ASD
$t_{min} = \sqrt{\dfrac{4r_{ut}b'}{\phi p F_y(1+\delta\alpha')}}$ (from *Manual* Eq. 9-19a) $= \sqrt{\dfrac{4(24.8\text{ kips})(1.13\text{ in.})}{0.90(10.0\text{ in.})(36\text{ ksi})[1+0.919(0.548)]}}$ $= 0.480\text{ in.}$ Use a ½-in.-thick end plate, t_1 > 0.480 in., further bolt check for prying not required. Use (4) ¾-in.-diameter Group A bolts.	$t_{\text{min}} = \sqrt{\dfrac{\Omega 4r_{at}b'}{pF_y(1+\delta\alpha')}}$ (from *Manual* Eq. 9-19b) $= \sqrt{\dfrac{1.67(4)(16.5\text{ kips})(1.13\text{ in.})}{(10.0\text{ in.})(36\text{ ksi})[1+0.919(0.568)]}}$ $= 0.477\text{ in.}$ Use a ½-in.-thick end plate, t_1 > 0.477 in., further bolt check for prying not required. Use (4) ¾-in.-diameter Group A bolts.

Required Weld Size

$$R_n = F_{nw}A_{we} \qquad \text{(\textit{Spec}. Eq. J2-4)}$$

$$F_{nw} = 0.60F_{EXX}\left(1.0 + 0.50\sin^{1.5}\theta\right) \qquad \text{(\textit{Spec}. Eq. J2-5)}$$

$$= 0.60(70\text{ ksi})\left(1.0 + 0.50\sin^{1.5}90°\right)$$

$$= 63.0\text{ ksi}$$

$$A_{we} = \left(\frac{\sqrt{2}}{2}\right)\left(\frac{D}{16}\right)l$$

where D is the weld size in sixteenths of an inch (i.e., D is an integer).

$$\begin{aligned} l &= 4(4.00\text{ in.}) \\ &= 16.0\text{ in.} \end{aligned}$$

Note: This weld length is approximate. A more accurate length could be determined by taking into account the curved corners of the HSS.

From AISC *Specification* Table J2.5:

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = \phi F_{nw} A_{we}$ $= 0.75(63.0\text{ ksi})\left(\frac{\sqrt{2}}{2}\right)\left(\frac{D}{16}\right)(16.0\text{ in.})$	$\frac{R_n}{\Omega} = \left(\frac{F_{nw} A_{we}}{\Omega}\right)$ $= \frac{(63.0\text{ ksi})\left(\frac{\sqrt{2}}{2}\right)\left(\frac{D}{16}\right)(16.0\text{ in.})}{2.00}$
Setting $\phi R_n = P_u$ and solving for D,	Setting $\frac{R_n}{\Omega} = P_a$ and solving for D,
$D \geq \frac{(99.2\text{ kips})(16)}{0.75(63.0\text{ ksi})\left(\frac{\sqrt{2}}{2}\right)(16.0\text{ in.})}$ $= 2.97$	$D \geq \frac{2.00(66.0\text{ kips})(16)}{(63.0\text{ ksi})\left(\frac{\sqrt{2}}{2}\right)(16.0\text{ in.})}$ $= 2.96$
$D = 3$ (i.e., a 3⁄16 in. weld)	$D = 3$ (i.e., a 3⁄16 in. weld)

Minimum Weld Size Requirements

For t = ¼ in., the minimum weld size = ⅛ in. from AISC *Specification* Table J2.4.

Summary:

Use a 3⁄16-in. weld with ½-in.-thick end plates and (4) ¾-in.-diameter Group A bolts.

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CHAPTER K DESIGN EXAMPLE REFERENCES

Fisher, J.M. and Kloiber, L.A. (2006), *Base Plate and Anchor Rod Design*, Design Guide 1, 2nd Ed., AISC, Chicago, IL

Packer, J.A., Sherman, D. and Lecce, M. (2010), *Hollow Structural Section Connections*, Design Guide 24, AISC, Chicago, IL.

Willibald, S., Packer, J.A. and Puthli, R.S. (2003), "Design Recommendations for Bolted Rectangular HSS Flange Plate Connections in Axial Tension," *Engineering Journal*, AISC, Vol. 40, No. 1, pp. 15–24.

APPENDIX 6
MEMBER STABILITY BRACING

This Appendix addresses the minimum strength and stiffness necessary to provide a braced point in a column, beam or beam-column.

The governing limit states for column and beam design may include flexural, torsional and flexural-torsional buckling for columns and lateral-torsional buckling for beams. In the absence of other intermediate bracing, column unbraced lengths are defined between points of obviously adequate lateral restraint, such as floor and roof diaphragms that are part of the building’s lateral force-resisting systems. Similarly, beams are often braced against lateral-torsional buckling by relatively strong and stiff bracing elements such as a continuously connected floor slab or roof diaphragm. However, at times, unbraced lengths are bounded by elements that may or may not possess adequate strength and stiffness to provide sufficient bracing. AISC *Specification* Appendix 6 provides equations for determining the required strength and stiffness of braces that have not been included in the second-order analysis of the structural system. It is not intended that the provisions of Appendix 6 apply to bracing that is part of the lateral force-resisting system. Guidance for applying these provisions to stabilize trusses is provided in AISC *Specification* Appendix 6 commentary.

Background for the provisions can be found in references cited in the Commentary including “Fundamentals of Beam Bracing” (Yura, 2001) and the *Guide to Stability Design Criteria for Metal Structures* (Ziemian, 2010). AISC *Manual* Part 2 also provides information on member stability bracing.

6.1 GENERAL PROVISIONS

Lateral column and beam bracing may be either panel or point while torsional beam bracing may be point or continuous. The User Note in AISC *Specification* Appendix 6, Section 6.1 states “A panel brace (formerly referred to as a relative brace) controls the angular deviation of a segment of the braced member between braced points (that is, the lateral displacement of one end of the segment relative to the other). A point brace (formerly referred to as a nodal brace) controls the movement at the braced point without direct interaction with adjacent braced points. A continuous bracing system consists of bracing that is attached along the entire member length.” Panel and point bracing systems are discussed further in AISC *Specification* Commentary Appendix 6, Section 6.1. Examples of each bracing type are shown in AISC *Specification* Commentary Figure C-A-6.1.

In lieu of the requirements of Appendix 6, Sections 6.2, 6.3 and 6.4, alternative provisions are given in Sections 6.1(a), 6.1(b) and 6.1(c).

6.2 COLUMN BRACING

The requirements in this section apply to bracing associated with the limit state of flexural buckling. For columns that could experience torsional or flexural-torsional buckling, as addressed in AISC *Specification* Section E4, the designer must ensure that sufficient bracing to resist the torsional component of buckling is provided. See Helwig and Yura (1999).

Column braces may be panel or point. The type of bracing must be determined before the requirements for strength and stiffness can be determined. The requirements are derived for an infinite number of braces along the column and are thus conservative for most columns as explained in the Commentary. Provision is made in this section for reducing the required brace stiffness for point bracing when the column required strength is less than the available strength of the member. The Commentary also provides an approach to reduce the requirements when a finite number of point braces are provided.

6.3 BEAM BRACING

The requirements in this section apply to bracing of doubly and singly symmetric I-shaped members subject to flexure within a plane of symmetry and zero net axial force. Bracing to resist lateral-torsional buckling may be accomplished by a lateral brace, a torsional brace, or a combination of the two to prevent twist of the section. Lateral bracing should normally be connected near the compression flange. The exception is for the free ends of cantilevers and near inflection points of braced beams subject to double curvature bending. Torsional bracing may be connected anywhere on the cross section in a manner to prevent twist of the section.

According to AISC *Specification* Section F1(b), the design of members for flexure is based on the assumption that points of support are restrained against rotation about their longitudinal axis. The bracing requirements in Appendix 6 are for intermediate braces in addition to those at the support.

In members subject to double curvature, inflection points are not to be considered as braced points unless bracing is provided at that location. In addition, the bracing nearest the inflection point must be attached to prevent twist, either as a torsional brace or as lateral braces attached to both flanges as described in AISC *Specification* Appendix 6, Section 6.3.1(b).

6.3.1 Lateral Bracing

As with column bracing, beam bracing may be panel or point. In addition, it is permissible to provide torsional bracing. This section provides requirements for determining the required lateral brace strength and stiffness for panel and point braces.

For point braces, provision is made in this section to reduce the required brace stiffness when the actual unbraced length is less than the maximum unbraced length for the required flexural strength.

6.3.2 Torsional Bracing

This section provides requirements for determining the required bracing flexural strength and stiffness for point and continuous torsional bracing. Torsional bracing can be connected to the section at any cross-section location. However, if the beam has inadequate distortional (out-of-plane) bending stiffness, torsional bracing will be ineffective. Web stiffeners can be provided when necessary, to increase the web distortional stiffness for point torsional braces.

As is the case for columns and for lateral beam point braces, it is possible to reduce the required brace stiffness when the required strength of the member is less than the available strength for the provided location of bracing.

Provisions for continuous torsional bracing are also provided. A slab connected to the top flange of a beam in double curvature may provide sufficient continuous torsional bracing as discussed in the Commentary. For this condition there is no unbraced length between braces so the unbraced length used in the strength and stiffness equations is the maximum unbraced length permitted to provide the required strength in the beam. In addition, for continuous torsional bracing, stiffeners are not permitted to be used to increase web distortional stiffness.

6.4 BEAM-COLUMN BRACING

For bracing of beam-columns, the required strength and stiffness are to be determined for the column and beam independently as specified in AISC *Specification* Appendix 6, Sections 6.2 and 6.3. These values are then to be combined, depending on the type of bracing provided.

EXAMPLE A-6.1 POINT STABILITY BRACING OF A W-SHAPE COLUMN

Given:

Determine the required strength and the stiffness for intermediate point braces, such that the unbraced length for the column can be taken as 12 ft. The column is an ASTM A992 W12×72 with loading and geometry as shown in Figure A-6.1-1. The column is braced laterally and torsionally at its ends with intermediate lateral braces for the *x*- and *y*-axis provided at the one-third points as shown. Thus, the unbraced length for the limit state of flexural-torsional buckling is 36 ft and the unbraced length for flexural buckling is 12 ft. The column has sufficient strength to support the applied loads with this bracing.

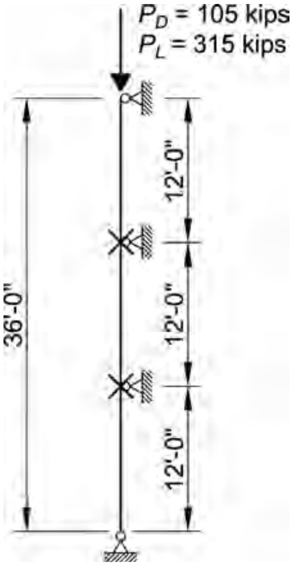


Fig. A-6.1-1. Column bracing geometry for Example A-6.1.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Column
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Required Compressive Strength of Column

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$P_u = 1.2(105\text{ kips}) + 1.6(315\text{ kips})$ $= 630\text{ kips}$	$P_a = 105\text{ kips} + 315\text{ kips}$ $= 420\text{ kips}$

Available Compressive Strength of Column

From AISC *Manual* Table 4-1a at L_{cy} = 12 ft, the available strength of the W12×72 is:

LRFD	ASD
$\phi_c P_n = 806 \text{ kips} > 630 \text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_c} = 536 \text{ kips} > 420 \text{ kips}$ **o.k.**

Required Point Brace Strength

From AISC *Specification* Appendix 6, Section 6.2.2, the required point brace strength is:

LRFD	ASD
$P_r = P_u$ $= 630 \text{ kips}$ $P_{br} = 0.01P_r$ (*Spec*. Eq. A-6-3) $= 0.01(630 \text{ kips})$ $= 6.30 \text{ kips}$	$P_r = P_a$ $= 420 \text{ kips}$ $P_{br} = 0.01P_r$ (*Spec*. Eq. A-6-3) $= 0.01(420 \text{ kips})$ $= 4.20 \text{ kips}$

Required Point Brace Stiffness

From AISC *Specification* Appendix 6, Section 6.2.2, the required point brace stiffness, with an unbraced length adjacent to the point brace L_{br} = 12 ft, is:

LRFD	ASD
$\phi = 0.75$ $P_r = P_u$ $= 630 \text{ kips}$ $\beta_{br} = \dfrac{1}{\phi}\left(\dfrac{8P_r}{L_{br}}\right)$ (*Spec*. Eq. A-6-4a) $= \dfrac{1}{0.75}\left[\dfrac{8(630 \text{ kips})}{(12 \text{ ft})(12 \text{ in./ft})}\right]$ $= 46.7 \text{ kip/in.}$	$\Omega = 2.00$ $P_r = P_a$ $= 420 \text{ kips}$ $\beta_{br} = \Omega\left(\dfrac{8P_r}{L_{br}}\right)$ (*Spec*. Eq. A-6-4b) $= 2.00\left[\dfrac{8(420 \text{ kips})}{(12 \text{ ft})(12 \text{ in./ft})}\right]$ $= 46.7 \text{ kip/in.}$

Determine the maximum permitted unbraced length for the required strength.

Interpolating between values, from AISC *Manual* Table 4-1a:

LRFD	ASD
L_{cy} = 18.9 ft for P_u = 632 kips	L_{cy} = 18.9 ft for P_a = 421 kips

Calculate the required point brace stiffness for this increased unbraced length

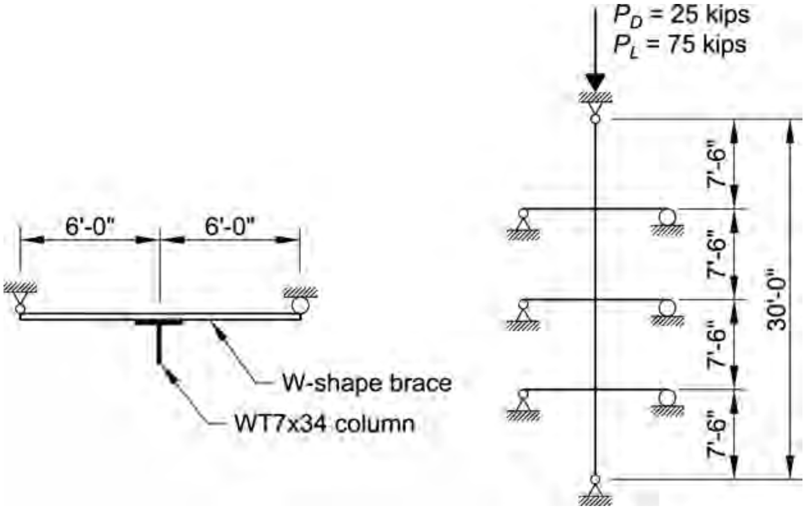
It is permissible to design the braces to provide the lower stiffness determined using the maximum unbraced length permitted to carry the required strength according to AISC *Specification* Appendix 6, Section 6.2.2.

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$P_r = P_u$ $= 630 \text{ kips}$	$P_r = P_a$ $= 420 \text{ kips}$
$\beta_{br} = \dfrac{1}{\phi}\left(\dfrac{8P_r}{L_{br}}\right)$ (*Spec*. Eq. A-6-4a) $= \dfrac{1}{0.75}\left[\dfrac{8(630 \text{ kips})}{(18.9 \text{ ft})(12 \text{ in./ft})}\right]$ $= 29.6 \text{ kip/in.}$	$\beta_{br} = \Omega\left(\dfrac{8P_r}{L_{br}}\right)$ (*Spec*. Eq. A-6-4b) $= 2.00\left[\dfrac{8(420 \text{ kips})}{(18.9 \text{ ft})(12 \text{ in./ft})}\right]$ $= 29.6 \text{ kip/in.}$

EXAMPLE A-6.2 POINT STABILITY BRACING OF A WT-SHAPE COLUMN

Given:

Determine the strength and stiffness requirements for the point braces and select a W-shape brace based on *x*-axis flexural buckling of the ASTM A992 WT7×34 column with loading and geometry as shown in Figure A-6.2-1. The unbraced length for this column is 7.5 ft. Bracing about the *y*-axis is provided by the axial resistance of a W-shape connected to the flange of the WT, while bracing about the *x*-axis is provided by the flexural resistance of the same W-shape loaded at the midpoint of a 12-ft-long simple span beam. Assume that the axial strength and stiffness of the W-shape are adequate to brace the *y*-axis of the WT. Also, assume the column is braced laterally and torsionally at its ends and is torsionally braced at one-quarter points by the W-shape braces.



(a) Plan (b) Elevation

Fig. A-6.2-1. Column bracing geometry for Example A-6.2.

Solution:

From AISC *Manual* Table 2-4, the material properties of the column and brace are as follows:

ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Required Compressive Strength of Column

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$P_u = 1.2(25\text{ kips}) + 1.6(75\text{ kips})$ $= 150\text{ kips}$	$P_a = 25\text{ kips} + 75\text{ kips}$ $= 100\text{ kips}$

Available Compressive Strength of Column

Interpolating between values, from AISC *Manual* Table 4-7, the available axial compressive strength of the WT7×34 with $L_{cx} = 7.5\text{ ft}$ is:

LRFD	ASD
$\phi_c P_n = 357\text{ kips} > 150\text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_c} = 238\text{ kips} > 100\text{ kips}$ **o.k.**

Required Point Brace Size

From AISC *Specification* Appendix 6, Section 6.2.2, the required point brace strength is:

LRFD	ASD
$P_r = P_u$ $= 150\text{ kips}$ $P_{br} = 0.01P_r$ (*Spec*. Eq. A-6-3) $= 0.01(150\text{ kips})$ $= 1.50\text{ kips}$	$P_r = P_a$ $= 100\text{ kips}$ $P_{br} = 0.01P_r$ (*Spec*. Eq. A-6-3) $= 0.01(100\text{ kips})$ $= 1.00\text{ kips}$

From AISC *Specification* Appendix 6, Section 6.2.2, the required point brace stiffness is:

LRFD	ASD
$\phi = 0.75$ $P_r = P_u$ $= 150\text{ kips}$ $\beta_{br} = \dfrac{1}{\phi}\left(\dfrac{8P_r}{L_{br}}\right)$ (*Spec*. Eq. A-6-4a) $= \dfrac{1}{0.75}\left[\dfrac{8(150\text{ kips})}{(7.50\text{ ft})(12\text{ in./ft})}\right]$ $= 17.8\text{ kip/in.}$	$\Omega = 2.00$ $P_r = P_a$ $= 100\text{ kips}$ $\beta_{br} = \Omega\left(\dfrac{8P_r}{L_{br}}\right)$ (*Spec*. Eq. A-6-4b) $= 2.00\left[\dfrac{8(100\text{ kips})}{(7.50\text{ ft})(12\text{ in./ft})}\right]$ $= 17.8\text{ kip/in.}$

The brace is a simple-span beam loaded at its midspan. Thus, its flexural stiffness can be derived from Case 7 of AISC *Manual* Table 3-23 to be $48EI/L^3$, which must be greater than the required point brace stiffness, β_{br}. Also, the flexural strength of the beam, $\phi_b M_p$, for a compact laterally supported beam, must be greater than the moment resulting from the required brace strength over the beam's simple span, $M_{br} = P_{br}L/4$.

Based on brace stiffness, the minimum required moment of inertia of the beam is:

$$\begin{aligned} I_{br} &= \frac{\beta_{br} L^3}{48E} \\ &= \frac{(17.8\text{ kip/in.})(12.0\text{ ft})^3(12\text{ in./ft})^3}{48(29{,}000\text{ ksi})} \\ &= 38.2\text{ in.}^4 \end{aligned}$$

Based on moment strength for a compact laterally supported beam, the minimum required plastic section modulus is:

LRFD	ASD
$Z_{req} = \dfrac{M_{br}}{\phi F_y}$ $= \dfrac{(1.50\text{ kips})(12.0\text{ ft})(12\text{ in./ft})}{0.90(50\text{ ksi})(4)}$ $= 1.20\text{ in.}^3$	$Z_{req} = \dfrac{\Omega M_{br}}{F_y}$ $= \dfrac{1.67(1.00\text{ kip})(12.0\text{ ft})(12\text{ in./ft})}{(50\text{ ksi})(4)}$ $= 1.20\text{ in.}^3$

From AISC *Manual* Table 3-2, select a W8×13 member with $Z_x = 11.4\text{ in.}^3$ and $I_x = 39.6\text{ in.}^4$

Note that because the live-to-dead load ratio is 3, the LRFD and ASD results are identical.

The required stiffness can be reduced if the maximum permitted unbraced length is used as described in AISC *Specification* Appendix 6, Section 6.2, and also if the actual number of braces are considered, as discussed in the Commentary. The following demonstrates how this affects the design.

Interpolating between values in AISC *Manual* Table 4-7, the maximum permitted unbraced length of the WT7×34 for the required strength is as follows:

LRFD	ASD
L_{cx} = 18.6 ft for P_u = 150 kips	L_{cx} = 18.6 ft for P_a = 100 kips

From AISC *Specification* Commentary Appendix 6, Section 6.2, determine the reduction factor for three intermediate braces:

$$\frac{2n-1}{2n} = \frac{2(3)-1}{2(3)}$$
$$= 0.833$$

Determine the required point brace stiffness for the increased unbraced length and number of braces:

LRFD	ASD
$\phi = 0.75$ $P_r = P_u$ $= 150\text{ kips}$ $\beta_{br} = 0.833\left[\dfrac{1}{\phi}\left(\dfrac{8P_r}{L_{br}}\right)\right]$ (*Spec*. Eq. A-6-4a) $= 0.833\left\{\dfrac{1}{0.75}\left[\dfrac{8(150\text{ kips})}{(18.6\text{ ft})(12\text{ in./ft})}\right]\right\}$ $= 5.97\text{ kip/in.}$	$\Omega = 2.00$ $P_r = P_a$ $= 100\text{ kips}$ $\beta_{br} = 0.833\left[\Omega\left(\dfrac{8P_r}{L_{br}}\right)\right]$ (*Spec*. Eq. A-6-4b) $= 0.833\left\{2.00\left[\dfrac{8(100\text{ kips})}{(18.6\text{ ft})(12\text{ in./ft})}\right]\right\}$ $= 5.97\text{ kip/in.}$

Determine the required brace size based on this new stiffness requirement.

Based on brace stiffness, the minimum required moment of inertia of the beam is:

$$
\begin{aligned}
I_{br} &= \frac{\beta_{br} L^3}{48E} \\
&= \frac{(5.97 \text{ kip/in.})(12.0 \text{ ft})^3 (12 \text{ in./ft})^3}{48(29{,}000 \text{ ksi})} \\
&= 12.8 \text{ in.}^4
\end{aligned}
$$

Based on the unchanged flexural strength for a compact laterally supported beam, the minimum required plastic section modulus, Z_x, was determined previously to be 1.20 in.^{3} From AISC *Manual* Table 1-1, select a W6×8.5 noncompact member with Z_x = 5.73 in.^{3} and I_x = 14.9 in.^{4}

EXAMPLE A-6.3 POINT STABILITY BRACING OF A BEAM—CASE I

Given:

A walkway in an industrial facility has a span of 28 ft as shown in Figure A-6.3.1. The walkway has a deck of grating which is not sufficient to brace the beams. The ASTM A992 W12×22 beams along walkway edges are braced against twist at the ends as required by AISC *Specification* Section F1(b) and are connected by an L3×3×¼ strut at midspan. The two diagonal ASTM A36 L5×5×5⁄16 braces are connected to the top flange of the beams at the supports and at the strut at the middle. The strut and the brace connections are welded; therefore, bolt slippage does not need to be accounted for in the stiffness calculation. The dead load on each beam is 0.05 kip/ft and the live load is 0.125 kip/ft. Determine if the diagonal braces are strong enough and stiff enough to brace this walkway.

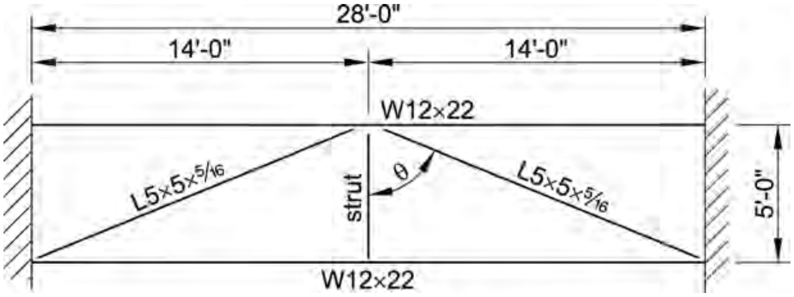


Fig. A-6.3-1. Plan view for Example A-6.3.

Solution:

Because the diagonal braces are connected directly to an unyielding support that is independent of the midspan brace point, they are designed as point braces. The strut will be assumed to be sufficiently strong and stiff to force the two beams to buckle together.

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Diagonal braces
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Tables 1-1 and 1-7, the geometric properties are as follows:

Beam
W12×22
$h_o = 11.9$ in.

Diagonal braces
L5×5×5⁄16
$A = 3.07$ in.^{2}

Required Flexure Strength of Beam

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$w_u = 1.2(0.05 \text{ kip/ft}) + 1.6(0.125 \text{ kip/ft})$ $= 0.260 \text{ kip/ft}$	$w_a = 0.05 \text{ kip/ft} + 0.125 \text{ kip/ft}$ $= 0.175 \text{ kip/ft}$

Determine the required flexural strength for a uniformly loaded simply supported beam using AISC *Manual* Table 3-23, Case 1.

LRFD	ASD
$M_u = \dfrac{w_u L^2}{8}$ $= \dfrac{(0.260 \text{ kip/ft})(28 \text{ ft})^2}{8}$ $= 25.5 \text{ kip-ft}$	$M_a = \dfrac{w_a L^2}{8}$ $= \dfrac{(0.175 \text{ kip/ft})(28 \text{ ft})^2}{8}$ $= 17.2 \text{ kip-ft}$

It can be shown that the W12×22 beams are adequate with the unbraced length of 14 ft. Both beams need bracing in the same direction simultaneously.

Required Brace Strength and Stiffness

From AISC *Specification* Appendix 6, Section 6.3, determine the required point brace strength for each beam as follows, with C_d = 1.0 for bending in single curvature.

LRFD	ASD
$M_r = M_u$ $= 25.5 \text{ kip-ft}$ $P_{br} = 0.02\left(\dfrac{M_r C_d}{h_o}\right)$ (*Spec*. Eq. A-6-7) $= 0.02\left[\dfrac{(25.5 \text{ kip-ft})(12 \text{ in./ft})(1.0)}{11.9 \text{ in.}}\right]$ $= 0.514 \text{ kip}$	$M_r = M_a$ $= 17.2 \text{ kip-ft}$ $P_{br} = 0.02\left(\dfrac{M_r C_d}{h_o}\right)$ (*Spec*. Eq. A-6-7) $= 0.02\left[\dfrac{(17.2 \text{ kip-ft})(12 \text{ in./ft})(1.0)}{11.9 \text{ in.}}\right]$ $= 0.347 \text{ kip}$

Because there are two beams to be braced, the total required brace strength is:

LRFD	ASD
$P_{br} = 2(0.514 \text{ kip})$ $= 1.03 \text{ kips}$	$P_{br} = 2(0.347 \text{ kip})$ $= 0.694 \text{ kip}$

There are two beams to brace and two braces to share the load. The worst case for design of the braces will be when they are in compression.

By geometry, the diagonal bracing length is

$$L = \sqrt{(14 \text{ ft})^2 + (5 \text{ ft})^2}$$
$$= 14.9 \text{ ft}$$

The required brace strength is:

LRFD	ASD
$P_{br}\cos\theta = P_{br}\left(\dfrac{5\text{ ft}}{14.9\text{ ft}}\right)$ $= 1.03\text{ kips}$ Because there are two braces, the required brace strength is: $P_{br} = \dfrac{1.03\text{ kips}}{2(5\text{ ft}/14.9\text{ ft})}$ $= 1.53\text{ kips}$	$P_{br}\cos\theta = P_{br}\left(\dfrac{5\text{ ft}}{14.9\text{ ft}}\right)$ $= 0.694\text{ kip}$ Because there are two braces, the required brace strength is: $P_{br} = \dfrac{0.694\text{ kip}}{2(5\text{ ft}/14.9\text{ ft})}$ $= 1.03\text{ kips}$

The required point brace stiffness, with C_d = 1.0 for bending in single curvature, is determined as follows:

LRFD	ASD
$\phi = 0.75$ $M_r = M_u$ $= 25.5\text{ kip-ft}$ $\beta_{br} = \dfrac{1}{\phi}\left(\dfrac{10M_rC_d}{L_{br}h_o}\right)$ (*Spec*. Eq. A-6-8a) $= \dfrac{1}{0.75}\left[\dfrac{10(25.5\text{ kip-ft})(12\text{ in./ft})(1.0)}{(14\text{ ft})(12\text{ in./ft})(11.9\text{ in.})}\right]$ $= 2.04\text{ kip/in.}$	$\Omega = 2.00$ $M_r = M_a$ $= 17.2\text{ kip-ft}$ $\beta_{br} = \Omega\left(\dfrac{10M_rC_d}{L_{br}h_o}\right)$ (*Spec*. Eq. A-6-8b) $= 2.00\left[\dfrac{10(17.2\text{ kip-ft})(12\text{ in./ft})(1.0)}{(14\text{ ft})(12\text{ in./ft})(11.9\text{ in.})}\right]$ $= 2.06\text{ kip/in.}$

Because there are two beams to be braced, the total required point brace stiffness is:

LRFD	ASD
$\beta_{br} = 2(2.04\text{ kip/in.})$ $= 4.08\text{ kip/in.}$	$\beta_{br} = 2(2.06\text{ kip/in.})$ $= 4.12\text{ kip/in.}$

The beams require bracing in order to have sufficient strength to carry the given load. However, locating that brace at the midspan provides flexural strength greater than the required strength. The maximum unbraced length permitted for the required flexural strength is L_b = 18.2 ft from AISC *Manual* Table 6-2. Thus, according to AISC *Specification* Appendix 6, Section 6.3.1b, this length could be used in place of 14 ft to determine the required stiffness. However, because the required stiffness is so small, the 14 ft length will be used here.

For a single brace, the stiffness is:

$$\beta = \frac{AE\cos^2\theta}{L}$$

$$= \frac{(3.07\text{ in.}^2)(29{,}000\text{ ksi})(5\text{ ft}/14.9\text{ ft})^2}{(14.9\text{ ft})(12\text{ in./ft})}$$

$$= 56.1\text{ kip/in.}$$

Because there are two braces, the system stiffness is twice this. Thus,

$$\begin{aligned}\beta &= 2\left(56.1\text{ kip/in.}\right)\\ &= 112\text{ kip/in.}\end{aligned}$$

LRFD	ASD
$\beta = 112\text{ kip/in.} > 4.08\text{ kip/in.}$ **o.k.**	$\beta = 112\text{ kip/in.} > 4.12\text{ kip/in.}$ **o.k.**

Available Strength of Braces

The braces may be called upon to act in either tension or compression, depending on which transverse direction the system tries to buckle. Brace compression buckling will control over tension yielding. Therefore, determine the compressive strength of the braces assuming they are eccentrically loaded using AISC *Manual* Table 4-12.

LRFD	ASD
Interpolating for L_c = 14.9 ft: $\phi_c P_n = 17.2\text{ kips} > 1.53\text{ kips}$ **o.k.**	Interpolating for L_c = 14.9 ft: $\dfrac{P_n}{\Omega_c} = 11.2\text{ kips} > 1.03\text{ kips}$ **o.k.**

The L5×5×5⁄16 braces have sufficient strength and stiffness to act as the point braces for this system.

EXAMPLE A-6.4 POINT STABILITY BRACING OF A BEAM—CASE II

Given:

A walkway in an industrial facility has a span of 28 ft as shown in Figure A-6.4-1. The walkway has a deck of grating which is not sufficient to brace the beams. The ASTM A992 W12×22 beams are braced against twist at the ends, and they are connected by a strut connected at midspan. At that same point they are braced to an adjacent ASTM A500 Grade C HSS8×8×¼ column by the attachment of a 5-ft-long ASTM A36 2L3×3×¼. The brace connections are all welded; therefore, bolt slippage does not need to be accounted for in the stiffness calculation. The adjacent column is not braced at the walkway level, but is adequately braced 12 ft below and 12 ft above the walkway level. The dead load on each beam is 0.05 kip/ft and the live load is 0.125 kip/ft. Determine if the bracing system has adequate strength and stiffness to brace this walkway.

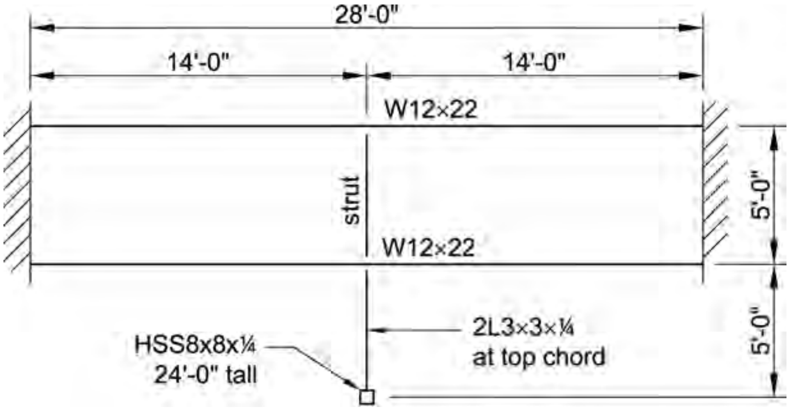


Fig. A-6.4-1. Plan view for Example A-6.4.

Solution:

Because the bracing system does not interact directly with any other braced point on the beam, the double angle and column constitute a point brace system. The strut will be assumed to be sufficiently strong and stiff to force the two beams to buckle together.

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

HSS column
ASTM A500 Grade C
F_y = 50 ksi
F_u = 62 ksi

Double-angle brace
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Tables 1-1, 1-12 and 1-15, the geometric properties are as follows:

Beam
W12×22
h_o = 11.9 in.

HSS column
HSS8×8×¼
I = 70.7 in.^{4}

Double-angle brace
2L3×3×¼
A = 2.88 in.^{2}

Required Flexural Strength of Beam

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$w_u = 1.2(0.05\text{ kip/ft}) + 1.6(0.125\text{ kip/ft})$ $= 0.260\text{ kip/ft}$	$w_a = 0.05\text{ kip/ft} + 0.125\text{ kip/ft}$ $= 0.175\text{ kip/ft}$

Determine the required flexural strength for a uniformly distributed load on the simply supported beam using AISC *Manual* Table 3-23, Case 1, as follows:

LRFD	ASD
$M_u = \dfrac{w_u L^2}{8}$ $= \dfrac{(0.260\text{ kip/ft})(28\text{ ft})^2}{8}$ $= 25.5\text{ kip-ft}$	$M_a = \dfrac{w_a L^2}{8}$ $= \dfrac{(0.175\text{ kip/ft})(28\text{ ft})^2}{8}$ $= 17.2\text{ kip-ft}$

It can be shown that the W12×22 beams are adequate with this unbraced length of 14 ft. Both beams need bracing in the same direction simultaneously.

Required Brace Strength and Stiffness

From AISC *Specification* Appendix 6, Section 6.3.1b, the required brace force for each beam, with C_d = 1.0 for bending in single curvature, is determined as follows:

LRFD	ASD
$M_r = M_u$ $= 25.5\text{ kip-ft}$ $P_{br} = 0.02\left(\dfrac{M_r C_d}{h_o}\right)$ (*Spec.* Eq. A-6-7) $= 0.02\left[\dfrac{(25.5\text{ kip-ft})(12\text{ in./ft})(1.0)}{11.9\text{ in.}}\right]$ $= 0.514\text{ kip}$	$M_r = M_a$ $= 17.2\text{ kip-ft}$ $P_{br} = 0.02\left(\dfrac{M_r C_d}{h_o}\right)$ (*Spec.* Eq. A-6-7) $= 0.02\left[\dfrac{(17.2\text{ kip-ft})(12\text{ in./ft})(1.0)}{11.9\text{ in.}}\right]$ $= 0.347\text{ kip}$

Because there are two beams, the total required brace force is:

LRFD	ASD
$P_{br} = 2(0.514\text{ kip})$ $= 1.03\text{ kips}$	$P_{br} = 2(0.347\text{ kip})$ $= 0.694\text{ kip}$

By inspection, the 2L3×3×¼ can carry the required bracing force. The HSS column can also carry the bracing force through bending on a 24-ft-long span. It will be shown that the change in length of the 2L3×3×¼ is negligible, so the available brace stiffness will come from the flexural stiffness of the column only.

From AISC *Specification* Appendix 6, Section 6.3.1b, with C_d = 1.0 for bending in single curvature, the required brace stiffness is:

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$M_r = M_u$ $= 25.5\text{ kip-ft}$	$M_r = M_a$ $= 17.2\text{ kip-ft}$
$\beta_{br} = \dfrac{1}{\phi}\left(\dfrac{10M_rC_d}{L_{br}h_o}\right)$ (*Spec*. Eq. A-6-8a) $= \dfrac{1}{0.75}\left[\dfrac{10(25.5\text{ kip-ft})(12\text{ in./ft})(1.0)}{(14\text{ ft})(12\text{ in./ft})(11.9\text{ in.})}\right]$ $= 2.04\text{ kip/in.}$	$\beta_{br} = \Omega\left(\dfrac{10M_rC_d}{L_{br}h_o}\right)$ (*Spec*. Eq. A-6-8b) $= 2.00\left[\dfrac{10(17.2\text{ kip-ft})(12\text{ in./ft})(1.0)}{(14\text{ ft})(12\text{ in./ft})(11.9\text{ in.})}\right]$ $= 2.06\text{ kip/in.}$

The beams require one brace in order to have sufficient strength to carry the given load. However, locating that brace at midspan provides flexural strength greater than the required strength. The maximum unbraced length permitted for the required flexural strength is L_b = 18.2 ft from AISC *Manual* Table 6-2. Thus, according to AISC *Specification* Appendix 6, Section 6.3.1b, this length could be used in place of 14 ft to determine the required stiffness.

Available Stiffness of Brace

Because the brace stiffness comes from the combination of the axial stiffness of the double-angle member and the flexural stiffness of the column loaded at its midheight, the individual element stiffness will be determined and then combined.

The axial stiffness of the double angle is:

$$\begin{aligned}\beta &= \frac{AE}{L}\\ &= \frac{(2.88\text{ in.}^2)(29{,}000\text{ ksi})}{(5\text{ ft})(12\text{ in./ft})}\\ &= 1{,}390\text{ kip/in.}\end{aligned}$$

The available flexural stiffness of the HSS column with a point load at midspan using AISC *Manual* Table 3-23, Case 7, is:

$$\begin{aligned}\beta &= \frac{48EI}{L^3}\\ &= \frac{48\left(29{,}000\text{ ksi}\right)\left(70.7\text{ in.}^4\right)}{\left(24.0\text{ ft}\right)^3\left(12\text{ in./ft}\right)^3}\\ &= 4.12\text{ kip/in.}\end{aligned}$$

The combined stiffness is:

$$\begin{aligned}\frac{1}{\beta} &= \frac{1}{\beta_{angles}} + \frac{1}{\beta_{column}}\\ &= \frac{1}{1{,}390\text{ kip/in.}} + \frac{1}{4.12\text{ kip/in.}}\\ &= 0.243\text{ in./kip}\end{aligned}$$

Thus, the system stiffness is:

$$\beta = 4.12\text{ kip/in.}$$

The stiffness of the double-angle member could have reasonably been ignored.

Because the double-angle brace is ultimately bracing two beams, the required stiffness is multiplied by 2:

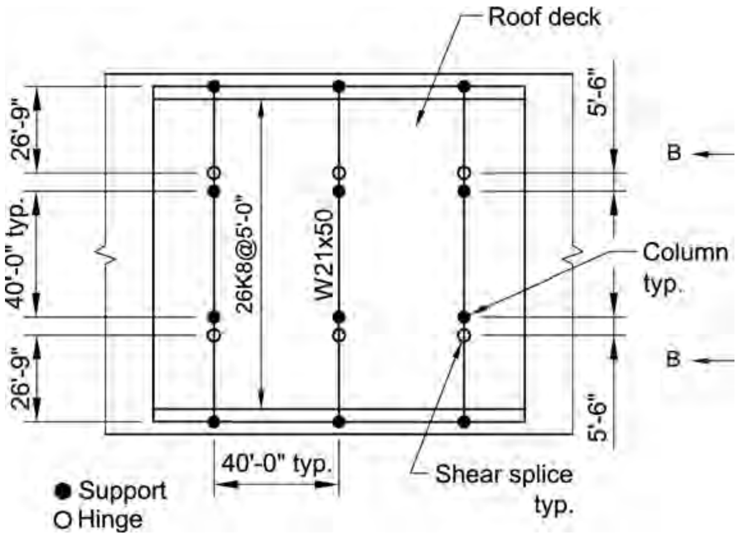
LRFD	ASD
$4.12\text{ kip/in.} \geq 2\left(2.04\text{ kip/in.}\right)$ $4.12\text{ kip/in.} > 4.08\text{ kip/in.}$ **o.k.**	$4.12\text{ kip/in.} \geq 2\left(2.06\text{ kip/in.}\right)$ $4.12\text{ kip/in.} = 4.12\text{ kip/in.}$ **o.k.**

The HSS8×8×¼ column is an adequate brace for the beams. However, if the column also carries an axial force, it must be checked for combined forces.

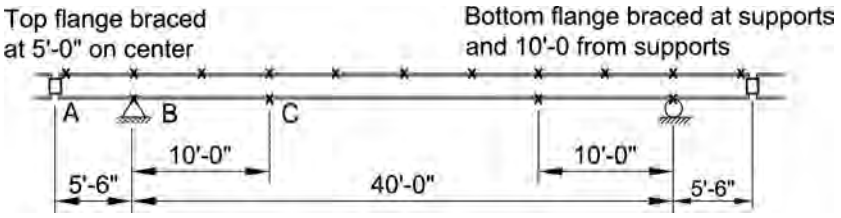
EXAMPLE A-6.5 POINT STABILITY BRACING OF A BEAM WITH REVERSE CURVATURE BENDING

Given:

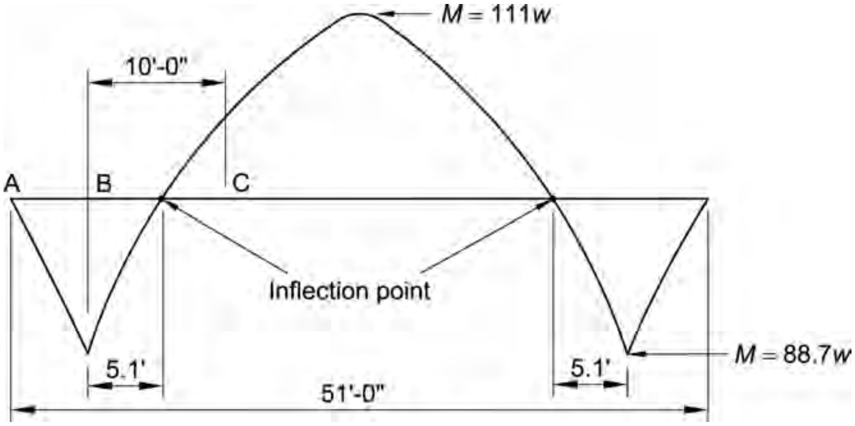
A roof system is composed of 26K8 steel joists spaced at 5-ft intervals and supported on ASTM A992 W21×50 girders as shown in Figure A-6.5-1(a). The roof dead load is 33 psf and the roof live load is 25 psf. Determine the required strength and stiffness of the braces needed to brace the girder at the support and near the inflection point. Bracing for the beam is shown in Figure A-6.5-1(b). Moment diagrams for the beam are shown in Figures A-6.5-1(c) and A-6.5-1(d). Determine the size of single-angle kickers connected to the bottom flange of the girder and the top chord of the joist, as shown in Figure A-6.5-1(e), where the brace force will be taken by a connected rigid diaphragm.



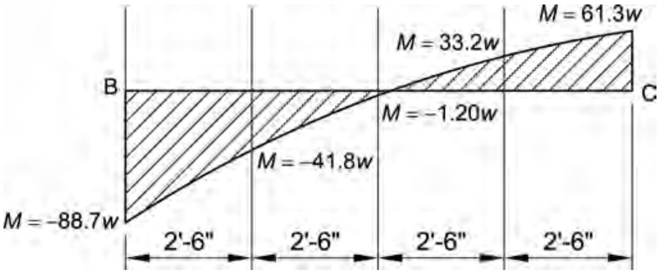
(a) Plan



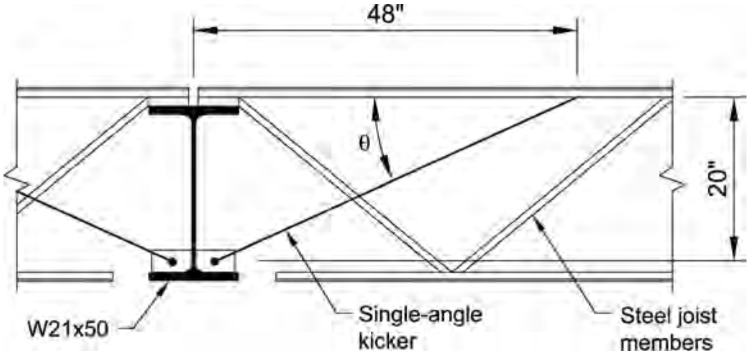
(b) Section B-B: Beam with bracing at top flanges by the steel joists and at the bottom flanges by the single-angle kickers



(c) Moment diagram of beam



(d) Moment diagram between points B and C



(e) Bracing configuration

Fig. A-6.5-1. Example A-6.5 configuration.

Solution:

Since the braces will transfer their force to a rigid roof diaphragm, they will be treated as point braces.

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Single-angle brace
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From the Steel Joist Institute:

Joist
K-Series
F_y = 50 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W21×50
h_o = 20.3 in.

Required Flexural Strength of Beam

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$w_u = 1.2(33\text{ psf}) + 1.6(25\text{ psf})$ $= 79.6\text{ psf}$ $w_u = \dfrac{(79.6\text{ psf})(40\text{ ft})}{1{,}000\text{ lb/kip}}$ $= 3.18\text{ kip/ft}$ From Figure A-6.5-1(d): $M_{uB} = 88.7(3.18\text{ kip/ft})$ $= 282\text{ kip-ft}$	$w_a = 33\text{ psf} + 25\text{ psf}$ $= 58.0\text{ psf}$ $w_a = \dfrac{(58.0\text{ psf})(40\text{ ft})}{1{,}000\text{ lb/kip}}$ $= 2.32\text{ kip/ft}$ From Figure A-6.5-1(d): $M_{aB} = 88.7(2.32\text{ kip/ft})$ $= 206\text{ kip-ft}$

Required Brace Strength and Stiffness

Determine the required force to brace the bottom flange of the girder with a point brace. The braces at points B and C will be determined based on the moment at B. However, because the brace at C is the closest to the inflection point, its strength and stiffness requirements are greater since they are influenced by the variable C_d which will be equal to 2.0.

From AISC *Specification* Appendix 6, Section 6.3.1b, the required brace force is determined as follows:

LRFD	ASD
$M_r = M_{uB}$ $= 282\text{ kip-ft}$	$M_r = M_{aB}$ $= 206\text{ kip-ft}$

LRFD	ASD
$P_{br} = 0.02\left(\frac{M_r C_d}{h_o}\right)$ (*Spec*. Eq. A-6-7) $= 0.02\left[\frac{(282\text{ kip-ft})(12\text{ in./ft})(2.0)}{20.3\text{ in.}}\right]$ $= 6.67\text{ kips}$	$P_{br} = 0.02\left(\frac{M_r C_d}{h_o}\right)$ (*Spec*. Eq. A-6-7) $= 0.02\left[\frac{(206\text{ kip-ft})(12\text{ in./ft})(2.0)}{20.3\text{ in.}}\right]$ $= 4.87\text{ kips}$

Determine the required stiffness of the point brace at point C. The required brace stiffness is a function of the unbraced length. It is permitted to use the maximum unbraced length permitted for the beam based upon the required flexural strength. Thus, determine the maximum unbraced length permitted.

Based on AISC *Specification* Section F1 and the moment diagram shown in Figure A-6.5-1(d), for the beam between points B and C, the lateral-torsional buckling modification factor, C_b, is:

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \qquad (\textit{Spec}.\text{ Eq. F1-1})$$

$$= \frac{12.5(|-88.7w|)}{2.5(|-88.7w|) + 3(|-41.8w|) + 4(|-1.2w|) + 3(|32.2w|)}$$

$$= 2.47$$

The maximum unbraced length for the required flexural strength can be determined by setting the available flexural strength based on AISC *Specification* Equation F2-3 (lateral-torsional buckling) equal to the required strength and solving for L_b (this is assuming that $L_b > L_r$).

LRFD	ASD
For a required flexural strength, M_u = 282 kip-ft, with C_b = 2.47, the unbraced length may be taken as: L_b = 22.0 ft	For a required flexural strength, M_a = 206 kip-ft, with C_b = 2.47, the unbraced length may be taken as: L_b = 20.6 ft

From AISC *Specification* Appendix 6, Section 6.3.1b, the required brace stiffness is:

LRFD	ASD
$\phi = 0.75$ $M_r = M_{uB}$ $= 282\text{ kip-ft}$ $\beta_{br} = \frac{1}{\phi}\left(\frac{10M_r C_d}{L_{br} h_o}\right)$ (*Spec*. Eq. A-6-8a) $= \frac{1}{0.75}\left[\frac{10(282\text{ kip-ft})(12\text{ in./ft})(2.0)}{(22.0\text{ ft})(12\text{ in./ft})(20.3\text{ in.})}\right]$ $= 16.8\text{ kip/in.}$	$\Omega = 2.00$ $M_r = M_{aB}$ $= 206\text{ kip-ft}$ $\beta_{br} = \Omega\left(\frac{10M_r C_d}{L_{br} h_o}\right)$ (*Spec*. Eq. A-6-8b) $= 2.00\left[\frac{10(206\text{ kip-ft})(12\text{ in./ft})(2.0)}{(20.6\text{ ft})(12\text{ in./ft})(20.3\text{ in.})}\right]$ $= 19.7\text{ kip/in.}$

Because no deformation will be considered in the connections, only the brace itself will be used to provide the required stiffness. The brace is oriented with the geometry as shown in Figure A-6.5-1(e). Thus, the force in the brace is $F_{br} = P_{br}/(\cos\theta)$ and the stiffness of the brace is $AE(\cos^2\theta)/L$. There are two braces at each brace point. One would be in tension and one in compression, depending on the direction that the girder attempts to buckle. For simplicity in design, a single brace will be selected that will be assumed to be in tension. Only the limit state of yielding will be considered.

Select a single angle to meet the requirements of strength and stiffness, with a length of:

$$L = \sqrt{(48 \text{ in.})^2 + (20 \text{ in.})^2}$$
$$= 52.0 \text{ in.}$$

Required Brace Force

LRFD	ASD
$F_{br} = \dfrac{P_{br}}{\cos\theta}$ $= \dfrac{6.67 \text{ kips}}{(48.0 \text{ in.}/52.0 \text{ in.})}$ $= 7.23 \text{ kips}$	$F_{br} = \dfrac{P_{br}}{\cos\theta}$ $= \dfrac{4.87 \text{ kips}}{(48.0 \text{ in.}/52.0 \text{ in.})}$ $= 5.28 \text{ kips}$

From AISC *Specification* Section D2(a), the required area based on available tensile strength is determined as follows:

$A_g = \dfrac{F_{br}}{\phi F_y}$ (modified *Spec.* Eq. D2-1) $= \dfrac{7.23 \text{ kips}}{0.90(36 \text{ kips})}$ $= 0.223 \text{ in.}^2$	$A_g = \dfrac{\Omega F_{br}}{F_y}$ (modified *Spec.* Eq. D2-1) $= \dfrac{1.67(5.28 \text{ kips})}{36 \text{ kips}}$ $= 0.245 \text{ in.}^2$

The required area based on stiffness is:

LRFD	ASD
$A_g = \dfrac{\beta_{br} L}{E \cos^2\theta}$ $= \dfrac{(16.8 \text{ kip/in.})(52.0 \text{ in.})}{(29{,}000 \text{ ksi})(48.0 \text{ in.}/52.0 \text{ in.})^2}$ $= 0.0354 \text{ in.}^2$	$A_g = \dfrac{\beta_{br} L}{E \cos^2\theta}$ $= \dfrac{(19.7 \text{ kip/in.})(52.0 \text{ in.})}{(29{,}000 \text{ ksi})(48.0 \text{ in.}/52.0 \text{ in.})^2}$ $= 0.0415 \text{ in.}^2$

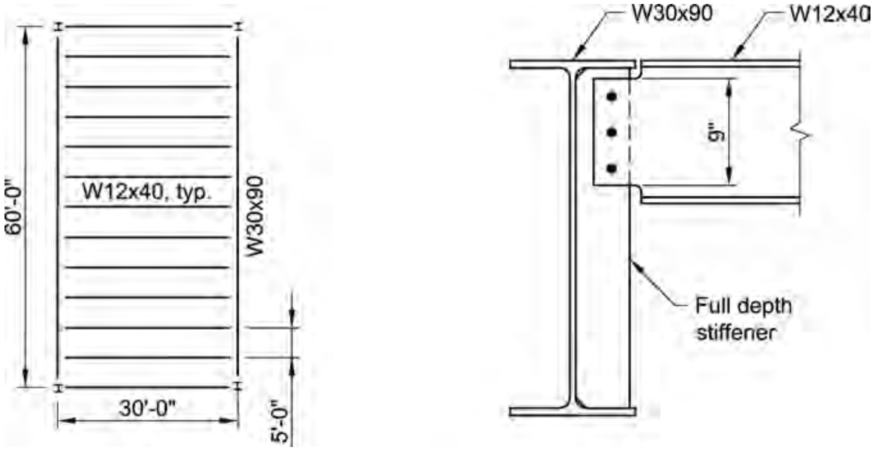
The strength requirement controls, therefore select L2×2×⅛ with $A = 0.491 \text{ in.}^2$

At the column at point B, the required strength would be one-half of that at point C, because $C_d = 1.0$ at point B instead of 2.0. However, since the smallest angle available has been selected for the brace, there is no reason to check further at the column and the same angle will be used there.

EXAMPLE A-6.6 POINT TORSIONAL STABILITY BRACING OF A BEAM

Given:

A roof system is composed of ASTM A992 W12×40 intermediate beams spaced 5 ft on center supporting a connected panel roof system that cannot be used as a diaphragm. As shown in Figure A-6.6-1, the beams span 30 ft and are supported on W30×90 girders spanning 60 ft. This is an isolated roof structure with no connections to other structures that could provide lateral support to the girder compression flanges. Thus, the flexural resistance of the attached beams must be used to provide torsional stability bracing of the girders. The roof dead load is 40 psf and the roof live load is 24 psf. Determine if the beams are sufficient to provide point torsional stability bracing.



(a) Plan *(b) Point torsional brace connection*

Fig. A-6.6-1. Roof system configuration

Solution:

Because the bracing beams are not connected in a way that would permit them to transfer an axial bracing force, they must behave as point torsional braces if they are to effectively brace the girders.

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam and girder
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W12×40
t_w = 0.295 in.
I_x = 307 in.^{4}

Girder
W30×90
t_w = 0.470 in.
h_o = 28.9 in.
I_y = 115 in.^{4}

Required Flexural Strength of Girder

From ASCE/SEI 7, Chapter 2, and using AISC *Manual* Table 3-23, Case 1, the required strength of the girder is:

LRFD	ASD
$w_u = 1.2(40\text{ psf}) + 1.6(24\text{ psf})$ $= 86.4\text{ psf}$ $w_u = \dfrac{(86.4\text{ psf})(15\text{ ft})}{1{,}000\text{ lb/kip}}$ $= 1.30\text{ kip/ft}$ $M_u = \dfrac{w_u L^2}{8}$ $= \dfrac{(1.30\text{ kip/ft})(60\text{ ft})^2}{8}$ $= 585\text{ kip-ft}$	$w_a = 40\text{ psf} + 24\text{ psf}$ $= 64.0\text{ psf}$ $w_a = \dfrac{(64.0\text{ psf})(15\text{ ft})}{1{,}000\text{ lb/kip}}$ $= 0.960\text{ kip/ft}$ $M_a = \dfrac{w_a L^2}{8}$ $= \dfrac{(0.960\text{ kip/ft})(60\text{ ft})^2}{8}$ $= 432\text{ kip-ft}$

With $C_b = 1.0$, from AISC *Manual* Table 3-10, the maximum unbraced length permitted for the W30×90 based upon required flexural strength is:

LRFD	ASD
For $M_{uB} = 585$ kip-ft, $L_b = 22.0$ ft	For $M_{aB} = 432$ kip-ft, $L_b = 20.7$ ft

Point Torsional Brace Design

The required flexural strength for a point torsional brace for the girder is determined from AISC *Specification* Appendix 6, Section 6.3.2a.

LRFD	ASD
$M_r = M_{uB}$ $= 585\text{ kip-ft}$ $M_{br} = 0.02M_r$ (*Spec.* Eq. A-6-9) $= 0.02(585\text{ kip-ft})$ $= 11.7\text{ kip-ft}$	$M_r = M_{aB}$ $= 432\text{ kip-ft}$ $M_{br} = 0.02M_r$ (*Spec.* Eq. A-6-9) $= 0.02(432\text{ kip-ft})$ $= 8.64\text{ kip-ft}$

The required overall point torsional brace stiffness with braces every 5 ft, $n = 11$, and assuming $C_b = 1.0$, is determined in the following. Based on the User Note in *Specification* Section 6.3.2a:

$$\begin{aligned} I_{yeff} &= I_y \\ &= 115\text{ in.}^4 \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\beta_T = \frac{1}{\phi}\frac{2.4L}{nEI_{yeff}}\left(\frac{M_r}{C_b}\right)^2$ (*Spec*. Eq. A-6-11a) $= \frac{1}{0.75}\left[\frac{2.4(60\text{ ft})(12\text{ in./ft})}{11(29{,}000\text{ ksi})(115\text{ in.}^4)}\right]$ $\times\left[\frac{(585\text{ kip-ft})(12\text{ in./ft})}{1.0}\right]^2$ $= 3{,}100\text{ kip-in./rad}$	$\Omega = 3.00$ $\beta_T = \Omega\frac{2.4L}{nEI_{yeff}}\left(\frac{M_r}{C_b}\right)^2$ (*Spec*. Eq. A-6-11b) $= 3.00\left[\frac{2.4(60\text{ ft})(12\text{ in./ft})}{11(29{,}000\text{ ksi})(115\text{ in.}^4)}\right]$ $\times\left[\frac{(432\text{ kip-ft})(12\text{ in./ft})}{1.0}\right]^2$ $= 3{,}800\text{ kip-in./rad}$

The distortional buckling stiffness of the girder web is a function of the web slenderness and the presence of any stiffeners. The web distortional stiffness is:

$$\beta_{sec} = \frac{3.3E}{h_o}\left(\frac{1.5h_o t_w^3}{12} + \frac{t_{st} b_s^3}{12}\right) \qquad \text{(\textit{Spec}. Eq. A-6-12)}$$

Therefore the distortional stiffness of the girder web alone is:

$$\beta_{sec} = \frac{3.3E}{h_o}\left(\frac{1.5h_o t_w^3}{12}\right)$$

$$= \frac{3.3(29{,}000\text{ ksi})}{28.9\text{ in.}}\left[\frac{1.5(28.9\text{ in.})(0.470\text{ in.})^3}{12}\right]$$

$$= 1{,}240\text{ kip-in./rad}$$

For AISC *Specification* Equation A-6-10 to give a nonnegative result, the web distortional stiffness given by Equation A-6-12 must be greater than the required point torsional stiffness given by Equation A-6-11. Because the web distortional stiffness of the girder is less than the required point torsional stiffness for both LRFD and ASD, web stiffeners will be required.

Determine the torsional stiffness contributed by the beams. Both girders will buckle in the same direction forcing the beams to bend in reverse curvature. Thus, the flexural stiffness of the beam using AISC *Manual* Table 3-23, Case 9, is:

$$\beta_{Tb} = \frac{6EI}{L}$$

$$= \frac{6(29{,}000\text{ ksi})(307\text{ in.}^4)}{(30\text{ ft})(12\text{ in./ft})}$$

$$= 148{,}000\text{ kip-in./rad}$$

Determining the required distortional stiffness of the girder will permit determination of the required stiffener size. The total stiffness is determined by summing the inverse of the distortional and flexural stiffnesses. Thus:

$$\frac{1}{\beta_T} = \frac{1}{\beta_{Tb}} + \frac{1}{\beta_{sec}}$$

Determine the minimum web distortional stiffness required to provide bracing for the girder.

LRFD	ASD
$\dfrac{1}{\beta_T} = \dfrac{1}{\beta_{Tb}} + \dfrac{1}{\beta_{sec}}$ $\dfrac{1}{3{,}100} = \dfrac{1}{148{,}000} + \dfrac{1}{\beta_{sec}}$ $\beta_{sec} = 3{,}170 \text{ kip-in./rad}$	$\dfrac{1}{\beta_T} = \dfrac{1}{\beta_{Tb}} + \dfrac{1}{\beta_{sec}}$ $\dfrac{1}{3{,}800} = \dfrac{1}{148{,}000} + \dfrac{1}{\beta_{sec}}$ $\beta_{sec} = 3{,}900 \text{ kip-in./rad}$

Determine the required width, b_s, of ⅜-in.-thick stiffeners.

LRFD	ASD
$\beta_{sec} = \dfrac{3.3E}{h_o}\left(\dfrac{1.5h_o t_w^3}{12} + \dfrac{t_{st} b_s^3}{12}\right)$ (*Spec*. Eq. A-6-12) Using the total required girder web distortional stiffness and the contribution of the girder web distortional stiffness calculated previously, solve for the required width for ⅜-in.-thick stiffeners: $3{,}170 \text{ kip-in./rad} = 1{,}240 \text{ kip-in./rad} + \dfrac{3.3(29{,}000 \text{ ksi})}{28.9 \text{ in.}}\left[\dfrac{(\text{⅜ in.})b_s^3}{12}\right]$ and b_s = 2.65 in.	$\beta_{sec} = \dfrac{3.3E}{h_o}\left(\dfrac{1.5h_o t_w^3}{12} + \dfrac{t_{st} b_s^3}{12}\right)$ (*Spec*. Eq. A-6-12) Using the total required girder web distortional stiffness and the contribution of the girder web distortional stiffness calculated previously, solve for the required width for ⅜-in.-thick stiffeners: $3{,}900 \text{ kip-in./rad} = 1{,}240 \text{ kip-in./rad} + \dfrac{3.3(29{,}000 \text{ ksi})}{28.9 \text{ in.}}\left[\dfrac{(\text{⅜ in.})b_s^3}{12}\right]$ and b_s = 2.95 in.

Therefore, use a 4 in. x ⅜ in. full depth one-sided stiffener at the connection of each beam.

Available Flexural Strength of Beam

Each beam is connected to a girder web stiffener. Thus, each beam will be coped at the top and bottom as shown in Figure A-6.6-1(b) with a depth at the coped section of 9 in. The available flexural strength of the coped beam is determined using the provisions of AISC *Specification* Sections J4.5 and F11.

$$M_n = M_p = F_y Z \le 1.6 F_y S_x \qquad (\textit{Spec}. \text{ Eq. F11-1})$$

For a rectangle, $Z < 1.6S$. Therefore, strength will be controlled by F_yZ and

$$\begin{aligned} Z &= \frac{(0.295 \text{ in.})(9.00 \text{ in.})^2}{4} \\ &= 5.97 \text{ in.}^3 \end{aligned}$$

The nominal flexural strength of the beam is:

$$\begin{aligned} M_n &= F_y Z_x \\ &= \frac{(50 \text{ ksi})(5.97 \text{ in.}^3)}{(12 \text{ in./ft})} \\ &= 24.9 \text{ kip-ft} \end{aligned}$$

LRFD	ASD
$\phi = 0.90$ $\phi M_n = 0.90(24.9\text{ kip-ft})$ $= 22.4\text{ kip-ft} > 11.7\text{ kip-ft}$ **o.k.**	$\Omega = 1.67$ $\dfrac{M_n}{\Omega} = \dfrac{24.9\text{ kip-ft}}{1.67}$ $= 14.9\text{ kip-ft} > 8.64\text{ kip-ft}$ **o.k.**

Neglecting any rotation due to the bolts moving in the holes or any influence of the end moments on the strength of the beams, this system has sufficient strength and stiffness to provide point torsional bracing to the girders.

Additional connection design limit states may also need to be checked.

APPENDIX 6 REFERENCES

Helwig, Todd A. and Yura, J.A. (1999), “Torsional Bracing of Columns,” *Journal of Structural Engineering*, ASCE, Vol. 125, No. 5, pp. 547–555.

Yura, J.A. (2001), “Fundamentals of Beam Bracing,” *Engineering Journal*, AISC, Vol. 38, No. 1, pp. 11–26.

Ziemian, R.D. (ed.) (2010), *Guide to Stability Design Criteria for Metal Structures*, 6th Ed., John Wiley & Sons, Inc., Hoboken, NJ.

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Chapter IIA
Simple Shear Connections

The design of connecting elements are covered in Part 9 of the AISC *Manual*. The design of simple shear connections is covered in Part 10 of the AISC *Manual*.

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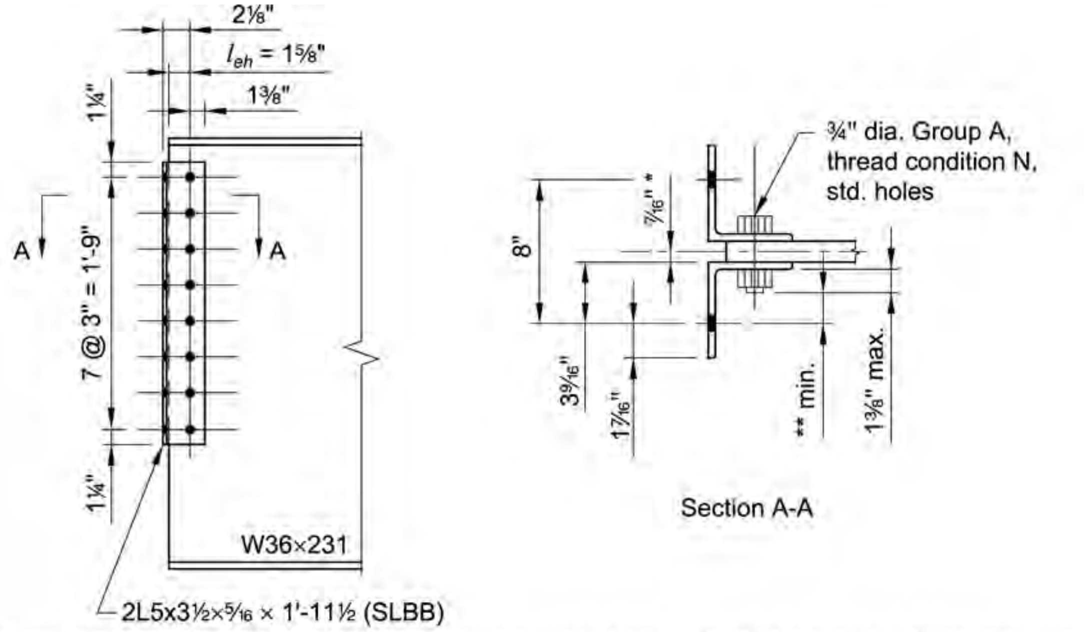
EXAMPLE II.A-1A ALL-BOLTED DOUBLE-ANGLE CONNECTION

Given:

Using the tables in AISC *Manual* Part 10, verify the available strength of an all-bolted double-angle shear connection between an ASTM A992 W36×231 beam and an ASTM A992 W14×90 column flange, as shown in Figure IIA-1A-1, supporting the following beam end reactions:

R_D = 37.5 kips
R_L = 113 kips

Use ASTM A36 angles.



* This dimension (see sketch, Section A) is determined as one-half of the decimal web thickness rounded to the next higher 1⁄16 in. Example: 0.760"/2 = 0.380"; use 7⁄16 in. This will produce spacing of holes in the supporting beam slightly larger than detailed in the angles to permit spreading of angles (angles can be spread but not closed) at time of erection to supporting member. Alternatively, consider using horizontal short slots in the support legs of the angles.
**See AISC *Manual* Tables 7-15 and 7-16 for driving clearance.

Fig. IIA-1A-1. Connection geometry for Example II.A-1A.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam and column
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Angles
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W36×231
$t_w = 0.760$ in.

Column
W14×90
$t_f = 0.710$ in.

From AISC *Specification* Table J3.3, the hole diameter for a ¾-in.-diameter bolt with standard holes is:

$$d_h = \tfrac{13}{16} \text{ in.}$$

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(37.5\text{ kips}) + 1.6(113\text{ kips})$ $= 226\text{ kips}$	$R_a = 37.5\text{ kips} + 113\text{ kips}$ $= 151\text{ kips}$

Connection Selection

AISC *Manual* Table 10-1 includes checks for the limit states of bolt shear, bolt bearing and tearout on the angles, shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles.

Try 8 rows of bolts and 2L5×3½×5⁄16 (SLBB). From AISC *Manual* Table 10-1:

LRFD	ASD
$\phi R_n = 248\text{ kips} > 226\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 165\text{ kips} > 151\text{ kips}$ **o.k.**

Available Beam Web Strength

The available beam web strength is the lesser of the limit states of block shear rupture, shear yielding, shear rupture, and the sum of the effective strengths of the individual fasteners. Because the beam is not coped, the only applicable limit state is the effective strength of the individual fasteners, which is the lesser of the bolt shear strength per AISC *Specification* Section J3.6, and the bolt bearing and tearout strength per AISC *Specification* Section J3.10.

Bolt Shear

From AISC *Manual* Table 7-1, the available shear strength per bolt for ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) in double shear is:

LRFD	ASD
$\phi R_n = 35.8\text{ kips/bolt}$	$\dfrac{R_n}{\Omega} = 23.9\text{ kips/bolt}$

Bolt Bearing on Beam Web

The nominal bearing strength of the beam web per bolt is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration:

$$
\begin{aligned}
r_n &= 2.4dtF_u \\
&= 2.4\left(\tfrac{3}{4}\text{ in.}\right)\left(0.760\text{ in.}\right)\left(65\text{ ksi}\right) \\
&= 88.9\text{ kips/bolt}
\end{aligned}
$$
(*Spec*. Eq. J3-6a)

From AISC *Specification* Section J3.10, the available bearing strength of the beam web per bolt is:

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75\left(88.9\text{ kips/bolt}\right)$ $= 66.7\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{88.9\text{ kips/bolt}}{2.00}$ $= 44.5\text{ kips/bolt}$

Bolt Tearout on Beam Web

The available tearout strength of the beam web per bolt is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration:

$$
\begin{aligned}
l_c &= 3.00\text{ in.} - \tfrac{13}{16}\text{ in.} \\
&= 2.19\text{ in.}
\end{aligned}
$$

The available tearout strength is:

$$
\begin{aligned}
r_n &= 1.2l_ctF_u \\
&= 1.2\left(2.19\text{ in.}\right)\left(0.760\text{ in.}\right)\left(65\text{ ksi}\right) \\
&= 130\text{ kips/bolt}
\end{aligned}
$$
(*Spec*. Eq. J3-6c)

From AISC *Specification* Section J3.10, the available tearout strength of the beam web per bolt is:

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75\left(130\text{ kips/bolt}\right)$ $= 97.5\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{130\text{ kips/bolt}}{2.00}$ $= 65.0\text{ kips/bolt}$

Bolt shear strength is the governing limit state for all bolts at the beam web. Bolt shear strength is one of the limit states included in the capacities shown in Table 10-1 as used above; thus, the effective strength of the fasteners is adequate.

Available Strength at the Column Flange

Since the thickness of the column flange, t_f = 0.710 in., is greater than the thickness of the angles, t = 5⁄16 in., bolt bearing will control for the angles, which was previously checked. The column flange is adequate for the required loading.

Conclusion

The connection is found to be adequate as given for the applied loads.

EXAMPLE II.A-1B ALL-BOLTED DOUBLE-ANGLE CONNECTION SUBJECT TO AXIAL AND SHEAR LOADING

Given:

Verify the available strength of an all-bolted double-angle connection for an ASTM A992 W18×50 beam, as shown in Figure II.A-1B-1, to support the following beam end reactions:

LRFD	ASD
Shear, V_u = 75 kips Axial tension, N_u = 60 kips	Shear, V_a = 50 kips Axial tension, N_a = 40 kips

Use ASTM A36 double angles that will be shop-bolted to the beam.

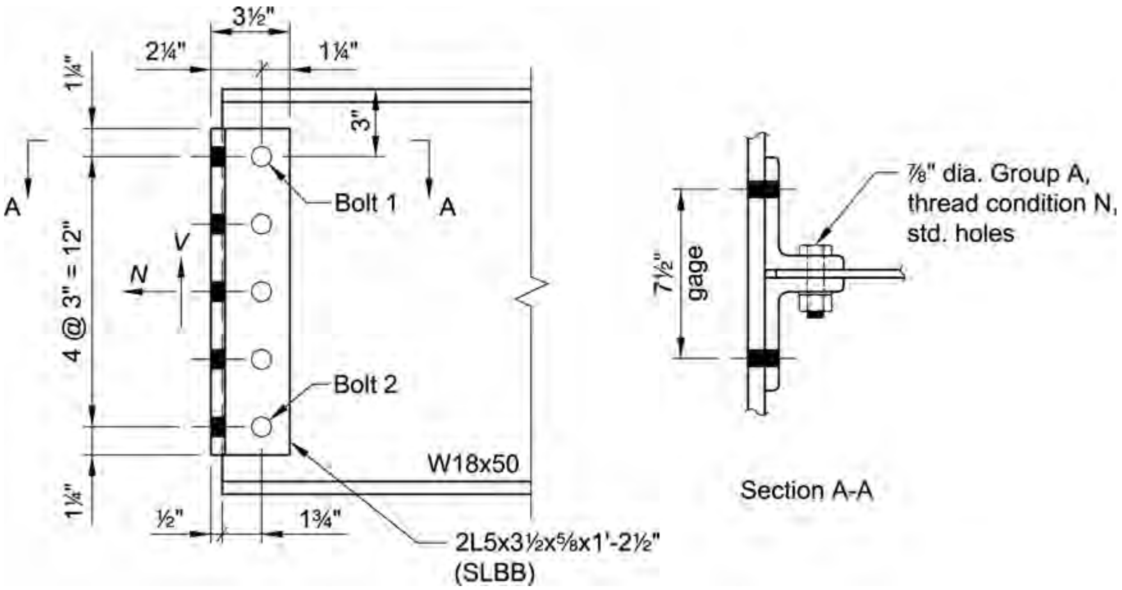


Fig. II.A-1B-1. Connection geometry for Example II.A-1B.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Angles
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W18×50
A_g = 14.7 in.^{2}
d = 18.0 in.
t_w = 0.355 in.
t_f = 0.570 in.

From AISC *Specification* Table J3.3, the hole diameter for ⅞-in.-diameter bolts with standard holes is:

d_h = 15⁄16 in.

The resultant load is:

LRFD	ASD
$R_u = \sqrt{V_u^2 + N_u^2}$ $= \sqrt{(75 \text{ kips})^2 + (60 \text{ kips})^2}$ $= 96.0 \text{ kips}$	$R_a = \sqrt{V_a^2 + N_a^2}$ $= \sqrt{(50 \text{ kips})^2 + (40 \text{ kips})^2}$ $= 64.0 \text{ kips}$

Try 5 rows of bolts and 2L5×3½×⅝ (SLBB).

Strength of the Bolted Connection—Angles

From the Commentary to AISC *Specification* Section J3.6, the strength of the bolt group is taken as the sum of the invividual strengths of the individual fasteners, which may be taken as the lesser of the fastener shear strength per AISC *Specification* Section J3.6, the bearing strength at the bolt hole per AISC *Specification* Section J3.10, or the tearout strength at the bolt hole per AISC *Specification* Section J3.10.

Bolt shear

From AISC *Manual* Table 7-1, the available shear strength for ⅞-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) in double shear (or pair of bolts) is:

LRFD	ASD
$\phi r_n = 48.7$ kips/bolt (or per pair of bolts)	$\dfrac{r_n}{\Omega} = 32.5$ kips/bolt (or per pair of bolts)

Bolt bearing on angles

The available bearing strength of the angles per bolt in double shear is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration:

$$\begin{aligned} r_n &= (2 \text{ angles})2.4dtF_u \qquad \text{(from } Spec. \text{ Eq. J3-6a)} \\ &= (2 \text{ angles})(2.4)(\tfrac{7}{8} \text{ in.})(\tfrac{5}{8} \text{ in.})(58 \text{ ksi}) \\ &= 152 \text{ kips/bolt} \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(152 \text{ kips/bolt})$ $= 114 \text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{152 \text{ kips/bolt}}{2.00}$ $= 76.0 \text{ kips/bolt}$

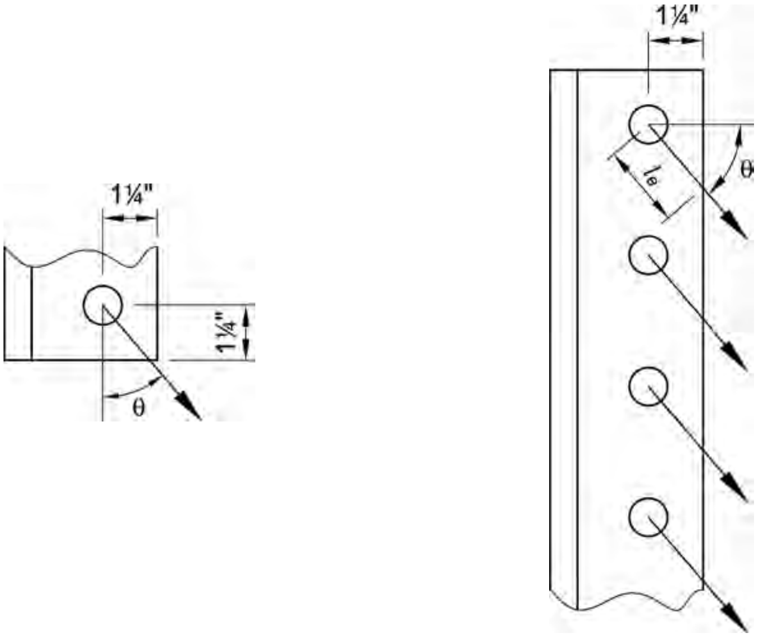
Bolt tearout on angles

From AISC *Specification* Section J3.10, the available tearout strength of the angles per bolt in double shear is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration.

As shown in Figures II.A-1B-2(a) and II.A-1B-2(b), the tearout dimensions on the angle differ between the edge bolt and the other bolts.

The angle θ, as shown in Figure II.A-1B-2(a), of the resultant force on the edge bolt is:

LRFD	ASD
$\theta = \tan^{-1}\left(\dfrac{N_u}{V_u}\right)$ $= \tan^{-1}\left(\dfrac{60 \text{ kips}}{75 \text{ kips}}\right)$ $= 38.7°$	$\theta = \tan^{-1}\left(\dfrac{N_a}{V_a}\right)$ $= \tan^{-1}\left(\dfrac{40 \text{ kips}}{50 \text{ kips}}\right)$ $= 38.7°$



(a) Edge bolt *(b) Other bolts*

Fig. II.A-1B-2. Bolt tearout on angles.

The length from the center of the bolt hole to the edge of the angle along the line of action of the force is:

$$\begin{aligned} l_e &= \frac{1\tfrac{1}{4}\text{ in.}}{\cos 38.7^\circ} \\ &= 1.60\text{ in.} \end{aligned}$$

The clear distance, along the line of action of the force, between the edge of the hole and the edge of the angle is:

$$\begin{aligned} l_c &= l_e - 0.5d_h \\ &= 1.60\text{ in.} - 0.5\left(\tfrac{15}{16}\text{ in.}\right) \\ &= 1.13\text{ in.} \end{aligned}$$

The available tearout strength of the pair of angles at the edge bolt is:

$$\begin{aligned} r_n &= (2\text{ angles})1.2l_ctF_u \\ &= (2\text{ angles})(1.2)(1.13\text{ in.})\left(\tfrac{5}{8}\text{ in.}\right)(58\text{ ksi}) \\ &= 98.3\text{ kips/bolt} \end{aligned}$$ (from *Spec.* Eq. J3-6c)

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(98.3\text{ kips/bolt})$ $= 73.7\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{98.3\text{ kips/bolt}}{2.00}$ $= 49.2\text{ kips/bolt}$

Therefore, bolt shear controls over bearing or tearout of the angles at the edge bolt.

The angle θ, as shown in Figure II.A-1B-2(b), of the resultant force on the other bolts is:

LRFD	ASD
$\theta = \tan^{-1}\left(\dfrac{V_u}{N_u}\right)$ $= \tan^{-1}\left(\dfrac{75\text{ kips}}{60\text{ kips}}\right)$ $= 51.3^\circ$	$\theta = \tan^{-1}\left(\dfrac{V_a}{N_a}\right)$ $= \tan^{-1}\left(\dfrac{50\text{ kips}}{40\text{ kips}}\right)$ $= 51.3^\circ$

The length from the center of the bolt hole to the edge of the angle along the line of action of the force is:

$$\begin{aligned} l_e &= \frac{1\tfrac{1}{4}\text{ in.}}{\cos 51.3^\circ} \\ &= 2.00\text{ in.} \end{aligned}$$

The clear distance, along the line of action of the force, between the edge of the hole and the edge of the angle is:

$$\begin{aligned} l_c &= l_e - 0.5d_h \\ &= 2.00\text{ in.} - 0.5\left(\tfrac{15}{16}\text{ in.}\right) \\ &= 1.53\text{ in.} \end{aligned}$$

The available tearout strength of the pair of angles at the other bolts is:

$$\begin{aligned} r_n &= (2\text{ angles})1.2l_ctF_u \\ &= (2\text{ angles})(1.2)(1.53\text{ in.})(\tfrac{5}{8}\text{ in.})(58\text{ ksi}) \\ &= 133\text{ kips/bolt} \end{aligned}$$ (from *Spec*. Eq. J3-6c)

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(133\text{ kips/bolt})$ $= 99.8\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{133\text{ kips/bolt}}{2.00}$ $= 66.5\text{ kips/bolt}$

Therefore, bolt shear controls over bearing or tearout of the angles at the other bolt.

The effective strength for the bolted connection at the angles is determined by summing the effective strength for each bolt using the minimum available strength calculated for bolt shear, bearing on the angles, and tearout on the angles.

LRFD	ASD
$\phi R_n = n\phi r_n$ $= (5\text{ bolts})(48.7\text{ kips/bolt})$ $= 244\text{ kips} > 96.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = n\dfrac{r_n}{\Omega}$ $= (5\text{ bolts})(32.5\text{ kips/bolt})$ $= 163\text{ kips} > 64.0\text{ kips}$ **o.k.**

Strength of the Bolted Connection—Beam Web

Bolt bearing on beam web

The available bearing strength of the beam web per bolt is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration:

$$\begin{aligned} r_n &= 2.4dtF_u \\ &= 2.4(\tfrac{7}{8}\text{ in.})(0.355\text{ in.})(65\text{ ksi}) \\ &= 48.5\text{ kips/bolt} \end{aligned}$$ (*Spec*. Eq. J3-6a)

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(48.5\text{ kips/bolt})$ $= 36.4\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{48.5\text{ kips/bolt}}{2.00}$ $= 24.3\text{ kips/bolt}$

Bolt tearout on beam web

From AISC *Specification* Section J3.10, the available tearout strength of the beam web is determined from AISC *Specification* Equation J3-6a, assuming deformation at the bolt hole is a design consideration, where the edge distance, l_c, is based on the angle of the resultant load. As shown in Figure II.A-1B-3, a horizontal edge distance of 1½ in. is used which includes a ¼ in. tolerance to account for possible mill underrun.

The angle, θ, of the resultant force is:

LRFD	ASD
$\theta = \tan^{-1}\left(\dfrac{V_u}{N_u}\right)$ $= \tan^{-1}\left(\dfrac{75\text{ kips}}{60\text{ kips}}\right)$ $= 51.3°$	$\theta = \tan^{-1}\left(\dfrac{V_a}{N_a}\right)$ $= \tan^{-1}\left(\dfrac{50\text{ kips}}{40\text{ kips}}\right)$ $= 51.3°$

The length from the center of the bolt hole to the edge of the web along the line of action of the force is:

$$\begin{aligned} l_e &= \frac{1\tfrac{1}{2}\text{ in.}}{\cos 51.3°} \\ &= 2.40\text{ in.} \end{aligned}$$

The clear distance, along the line of action of the force, between the edge of the hole and the edge of the web is:

$$\begin{aligned} l_c &= l_e - 0.5d_h \\ &= 2.40\text{ in.} - 0.5\left(\tfrac{15}{16}\text{ in.}\right) \\ &= 1.93\text{ in.} \end{aligned}$$

The available tearout strength of the beam web is determined as follows:

$$\begin{aligned} r_n &= 1.2 l_c t F_u \\ &= 1.2(1.93\text{ in.})(0.355\text{ in.})(65\text{ ksi}) \\ &= 53.4\text{ kips/bolt} \end{aligned}$$ (*Spec*. Eq. J3-6c)

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(53.4\text{ kips/bolt})$ $= 40.1\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{53.4\text{ kips/bolt}}{2.00}$ $= 26.7\text{ kips/bolt}$

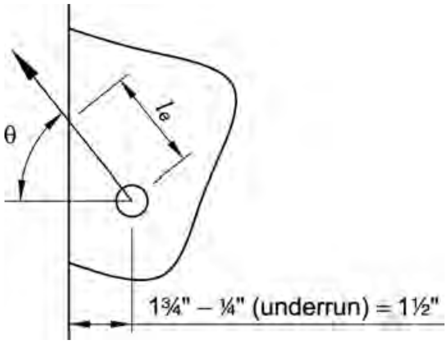


Fig. II.A-1B-3. Bolt tearout on beam web.

Therefore, bolt bearing on the beam web is the controlling limit state for all bolts.

The effective strength for the bolted connection at the beam web is determined by summing the effective strength for each bolt using the minimum available strength calculated for bolt shear, bearing on the beam web, and tearout on the beam web.

LRFD	ASD
$\phi R_n = n\phi r_n$ $= (5\text{ bolts})(36.4\text{ kips/bolt})$ $= 182\text{ kips} > 96.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = n\dfrac{r_n}{\Omega}$ $= (5\text{ bolts})(24.3\text{ kips/bolt})$ $= 122\text{ kips} > 64.0\text{ kips}$ **o.k.**

Bolt Shear and Tension Interaction—Outstanding Angle Legs

The available tensile strength of the bolts due to the effect of combined tension and shear is determined from AISC *Specification* Section J3.7.

The required shear stress is:

$$f_{rv} = \frac{V_r}{nA_b}$$

where

$A_b = 0.601\text{ in.}^2$ (from AISC *Manual* Table 7-1)

$n = 10$

LRFD	ASD
$f_{rv} = \dfrac{V_u}{nA_b}$ $= \dfrac{75\text{ kips}}{10(0.601\text{ in.}^2)}$ $= 12.5\text{ ksi}$	$f_{rv} = \dfrac{V_a}{nA_b}$ $= \dfrac{50\text{ kips}}{10(0.601\text{ in.}^2)}$ $= 8.32\text{ ksi}$

The nominal tensile strength modified to include the effects of shear stress is determined from AISC *Specification* Section J3.7 as follows. From AISC *Specification* Table J3.2:

$F_{nt} = 90\text{ ksi}$

$F_{nv} = 54\text{ ksi}$

LRFD	ASD
$\phi = 0.75$ $F'_{nt} = 1.3F_{nt} - \dfrac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt}$ (*Spec.* Eq. J3-3a) $= 1.3(90\text{ ksi}) - \dfrac{90\text{ ksi}}{0.75(54\text{ ksi})}(12.5\text{ ksi}) < 90\text{ ksi}$ $= 89.2\text{ ksi} < 90\text{ ksi}$	$\Omega = 2.00$ $F'_{nt} = 1.3F_{nt} - \dfrac{\Omega F_{nt}}{F_{nv}} f_{rv} \le F_{nt}$ (*Spec.* Eq. J3-3b) $= 1.3(90\text{ ksi}) - \dfrac{2.00(90\text{ ksi})}{54\text{ ksi}}(8.32\text{ ksi}) < 90\text{ ksi}$ $= 89.3\text{ ksi} < 90\text{ ksi}$

LRFD	ASD
Therefore: $F'_{nt} = 89.2$ ksi	Therefore: $F'_{nt} = 89.3$ ksi

Using the value of F'_{nt} determined for LRFD, the nominal tensile strength of one bolt is:

$$\begin{aligned} r_n &= F'_{nt} A_b \\ &= (89.2 \text{ ksi})(0.601 \text{ in.}^2) \\ &= 53.6 \text{ kips} \end{aligned} \qquad \text{(from } \textit{Spec}\text{. Eq. J3-2)}$$

The available tensile strength of the bolts due to combined tension and shear is:

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(53.6 \text{ kips/bolt})$ $= 40.2$ kips $\phi R_n = n\phi r_n$ $= (10 \text{ bolts})(40.2 \text{ kips/bolt})$ $= 402 \text{ kips} > 60 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{53.6 \text{ kips/bolt}}{2.00}$ $= 26.8$ kips $\dfrac{R_n}{\Omega} = n\dfrac{r_n}{\Omega}$ $= (10 \text{ bolts})(26.8 \text{ kips/bolt})$ $= 268 \text{ kips} > 40 \text{ kips}$ **o.k.**

Prying Action

From AISC *Manual* Part 9, the available tensile strength of the bolts in the outstanding angle legs taking prying action into account is determined as follows:

$$\begin{aligned} a &= \frac{2(angle\ leg) + t_w - gage}{2} \\ &= \frac{2(5 \text{ in.}) + 0.355 \text{ in.} - 7\tfrac{1}{2} \text{ in.}}{2} \\ &= 1.43 \text{ in.} \end{aligned}$$

$$\begin{aligned} b &= \frac{gage - t_w - t}{2} \\ &= \frac{7\tfrac{1}{2} \text{ in.} - 0.355 \text{ in.} - \tfrac{5}{8} \text{ in.}}{2} \\ &= 3.26 \text{ in.} \end{aligned}$$

$$\begin{aligned} a' &= \left(a + \frac{d_b}{2}\right) \le \left(1.25b + \frac{d_b}{2}\right) \\ &= 1.43 \text{ in.} + \frac{\tfrac{7}{8} \text{ in.}}{2} \le 1.25(3.26 \text{ in.}) + \frac{\tfrac{7}{8} \text{ in.}}{2} \\ &= 1.87 \text{ in.} < 4.51 \text{ in.} \quad \textbf{o.k.} \end{aligned} \qquad (\textit{Manual} \text{ Eq. 9-23})$$

$$
\begin{aligned}
b' &= \left(b - \frac{d_b}{2}\right) \\
&= 3.26\text{ in.} - \frac{\tfrac{7}{8}\text{ in.}}{2} \\
&= 2.82\text{ in.}
\end{aligned}
$$

(*Manual* Eq. 9-18)

$$
\begin{aligned}
\rho &= \frac{b'}{a'} \\
&= \frac{2.82\text{ in.}}{1.87\text{ in.}} \\
&= 1.51
\end{aligned}
$$

(*Manual* Eq. 9-22)

Note that end distances of 1¼ in. are used on the angles, so p is the average pitch of the bolts:

$$
\begin{aligned}
p &= \frac{l}{n} \\
&= \frac{14\tfrac{1}{2}\text{ in.}}{5\text{ rows}} \\
&= 2.90\text{ in.}
\end{aligned}
$$

Check:

$p < s = 3.00$ in. **o.k.**

$$
\begin{aligned}
\delta &= 1 - \frac{d'}{p} \\
&= 1 - \frac{\tfrac{15}{16}\text{ in.}}{2.90\text{ in.}} \\
&= 0.677
\end{aligned}
$$

(*Manual* Eq. 9-20)

The angle thickness required to develop the available strength of the bolt with no prying action is determined as follows:

LRFD	ASD
$\phi = 0.90$ $B_c = 40.2$ kips/bolt (calculated previously) $t_c = \sqrt{\dfrac{4B_c b'}{\phi p F_u}}$ (*Manual* Eq. 9-26a) $= \sqrt{\dfrac{4(40.2\text{ kips/bolt})(2.82\text{ in.})}{0.90(2.90\text{ in.})(58\text{ ksi})}}$ $= 1.73$ in.	$\Omega = 1.67$ $B_c = 26.8$ kips/bolt (calculated previously) $t_c = \sqrt{\dfrac{\Omega 4 B_c b'}{p F_u}}$ (*Manual* Eq. 9-26b) $= \sqrt{\dfrac{1.67(4)(26.8\text{ kips/bolt})(2.82\text{ in.})}{(2.90\text{ in.})(58\text{ ksi})}}$ $= 1.73$ in.

$$\alpha' = \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] \qquad \text{(}\textit{Manual}\text{ Eq. 9-28)}$$

$$= \frac{1}{0.677(1+1.51)}\left[\left(\frac{1.73 \text{ in.}}{\frac{5}{8} \text{ in.}}\right)^2 - 1\right]$$

$$= 3.92$$

Because $\alpha' > 1$, the angles have insufficient strength to develop the bolt strength, therefore:

$$Q = \left(\frac{t}{t_c}\right)^2 (1+\delta)$$

$$= \left(\frac{\frac{5}{8} \text{ in.}}{1.73 \text{ in.}}\right)^2 (1+0.677)$$

$$= 0.219$$

The available tensile strength of the bolts, taking prying action into account, is determined using AISC *Manual* Equation 9-27, as follows:

LRFD	ASD
$\phi r_n = B_c Q$ $= (40.2 \text{ kips/bolt})(0.219)$ $= 8.80 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = B_c Q$ $= (26.8 \text{ kips/bolt})(0.219)$ $= 5.87 \text{ kips/bolt}$
$\phi R_n = n\phi r_n$ $= (10 \text{ bolts})(8.80 \text{ kips/bolt})$ $= 88.0 \text{ kips} > 60 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = n\dfrac{r_n}{\Omega}$ $= (10 \text{ bolts})(5.87 \text{ kips/bolt})$ $= 58.7 \text{ kips} > 40 \text{ kips}$ **o.k.**

Shear Strength of Angles

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the angles is determined as follows:

$$A_{gv} = (2 \text{ angles})lt$$

$$= (2 \text{ angles})(14\tfrac{1}{2} \text{ in.})(\tfrac{5}{8} \text{ in.})$$

$$= 18.1 \text{ in.}^2$$

$$R_n = 0.60F_y A_{gv} \qquad \text{(}\textit{Spec}\text{. Eq. J4-3)}$$

$$= 0.60(36 \text{ ksi})(18.1 \text{ in.}^2)$$

$$= 391 \text{ kips}$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(391\text{ kips})$ $= 391\text{ kips} > 96.0\text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{391\text{ kips}}{1.50}$ $= 261\text{ kips} > 64.0\text{ kips}$ **o.k.**

From AISC *Specification* Section J4.2, the available shear rupture strength of the angle is determined using the net area determined in accordance with AISC *Specification* Section B4.3b.

$$\begin{aligned}A_{nv} &= (2\text{ angles})\left[l - n\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t\\ &= (2\text{ angles})\left[14\tfrac{1}{2}\text{ in.} - 5\left(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right]\left(\tfrac{5}{8}\text{ in.}\right)\\ &= 11.9\text{ in.}^2\end{aligned}$$

$$\begin{aligned}R_n &= 0.60F_uA_{nv}\\ &= 0.60(58\text{ ksi})\left(11.9\text{ in.}^2\right)\\ &= 414\text{ kips}\end{aligned}$$ (*Spec.* Eq. J4-4)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(414\text{ kips})$ $= 311\text{ kips} > 96.0\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{414\text{ kips}}{2.00}$ $= 207\text{ kips} > 64.0\text{ kips}$ **o.k.**

Tensile Strength of Angles

From AISC Specification Section J4.1(a), the available tensile yielding strength of the angles is determined as follows:

$$\begin{aligned}A_g &= (2\text{ angles})lt\\ &= (2\text{ angles})\left(14\tfrac{1}{2}\text{ in.}\right)\left(\tfrac{5}{8}\text{ in.}\right)\\ &= 18.1\text{ in.}^2\end{aligned}$$

$$\begin{aligned}R_n &= F_yA_g\\ &= (36\text{ ksi})\left(18.1\text{ in.}^2\right)\\ &= 652\text{ kips}\end{aligned}$$ (*Spec.* Eq. J4-1)

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(652\text{ kips})$ $= 587\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{652\text{ kips}}{1.67}$ $= 390\text{ kips} > 40\text{ kips}$ **o.k.**

From AISC *Specification* Sections J4.1, the available tensile rupture strength of the angles is determined from AISC *Specification* Equation J4-2. Table D3.1, Case 1 applies in this case because the tension load is transmitted directly to the cross-sectional element by fasteners; therefore, $U = 1.00$. With $A_{nt} = A_{nv}$ (calculated previously), the effective net area is:

$$\begin{aligned} A_e &= A_{nt}U \\ &= \left(11.9\text{ in.}^2\right)(1.00) \\ &= 11.9\text{ in.}^2 \end{aligned}$$

(*Spec*. Eq. D3-1)

$$\begin{aligned} R_n &= F_u A_e \\ &= (58\text{ ksi})\left(11.9\text{ in.}^2\right) \\ &= 690\text{ kips} \end{aligned}$$

(*Spec*. Eq. J4-2)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(690\text{ kips})$ $= 518\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{690\text{ kips}}{2.00}$ $= 345\text{ kips} > 40\text{ kips}$ **o.k.**

Block Shear Rupture of Angles—Beam Web Side

The nominal strength for the limit state of block shear rupture of the angles, assuming an L-shaped tearout due the shear load only, is determined as follows. The tearout pattern is shown in Figure II.A-1B-4.

$$R_{bsv} = 0.60F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.60F_y A_{gv} + U_{bs} F_u A_{nt}$$

(*Spec*. Eq. J4-5)

where

$$\begin{aligned} A_{gv} &= (2\text{ angles})(l - l_{ev})t \\ &= (2\text{ angles})(14\tfrac{1}{2}\text{ in.} - 1\tfrac{1}{4}\text{ in.})(\tfrac{5}{8}\text{ in.}) \\ &= 16.6\text{ in.}^2 \end{aligned}$$

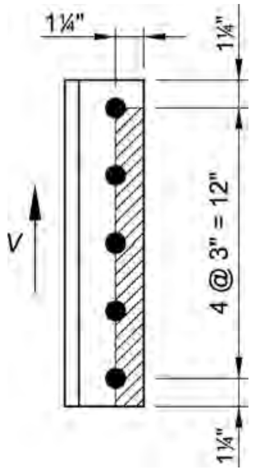


Fig. II.A-1B-4. Block shear rupture of angles for shear load only.

$$\begin{aligned} A_{nv} &= A_{gv} - (2\text{ angles})(n-0.5)(d_h + \tfrac{1}{16}\text{ in.})t \\ &= 16.6\text{ in.}^2 - (2\text{ angles})(5-0.5)(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})(\tfrac{5}{8}\text{ in.}) \\ &= 11.0\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= (2\text{ angles})\left[l_{eh} - 0.5(d_h + \tfrac{1}{16}\text{ in.})\right]t \\ &= (2\text{ angles})\left[1\tfrac{1}{4}\text{ in.} - 0.5(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})\right](\tfrac{5}{8}\text{ in.}) \\ &= 0.938\text{ in.}^2 \end{aligned}$$

$$U_{bs} = 1.0$$

and

$$\begin{aligned} R_{bsv} &= 0.60(58\text{ ksi})(11.0\text{ in.}^2) + 1.0(58\text{ ksi})(0.938\text{ in.}^2) \le 0.60(36\text{ ksi})(16.6\text{ in.}^2) + 1.0(58\text{ ksi})(0.938\text{ in.}^2) \\ &= 437\text{ kips} > 413\text{ kips} \end{aligned}$$

Therefore:

$$R_{bsv} = 413\text{ kips}$$

From AISC *Specification* Section J4.3, the available strength for the limit state of block shear rupture on the angles is:

LRFD	ASD
$\phi = 0.75$ $\phi R_{bsv} = 0.75(413\text{ kips})$ $= 310\text{ kips} > 75\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_{bsv}}{\Omega} = \dfrac{413\text{ kips}}{2.00}$ $= 207\text{ kips} > 50\text{ kips}$ **o.k.**

The block shear rupture failure path due to axial load only could occur as an L- or U-shape. Assuming an L-shaped tearout relative to the axial load on the angles, the nominal block shear rupture strength in the angles is determined as follows. The tearout pattern is shown in Figure II.A-1B-5.

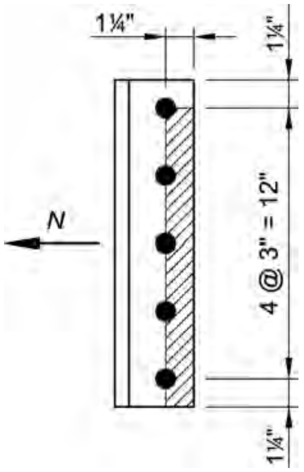


Fig. II.A-1B-5. Block shear rupture of angles for axial load only—L-shape.

$$R_{bsn} = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt}$$ (*Spec*. Eq. J4-5)

where

$$\begin{aligned} A_{gv} &= (2\text{ angles})l_{eh}t \\ &= (2\text{ angles})(1\tfrac{1}{4}\text{ in.})(\tfrac{5}{8}\text{ in.}) \\ &= 1.56\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - (2\text{ angles})(0.5)(d_h + \tfrac{1}{16}\text{ in.})t \\ &= 1.56\text{ in.}^2 - (2\text{ angles})(0.5)(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})(\tfrac{5}{8}\text{ in.}) \\ &= 0.935\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= (2\text{ angles})\left[(l - l_{ev}) - (n - 0.5)(d_h + \tfrac{1}{16}\text{ in.})\right]t \\ &= (2\text{ angles})\left[(14\tfrac{1}{2}\text{ in.} - 1\tfrac{1}{4}\text{ in.}) - (5 - 0.5)(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})\right](\tfrac{5}{8}\text{ in.}) \\ &= 10.9\text{ in.}^2 \end{aligned}$$

$$U_{bs} = 1.0$$

and

$$\begin{aligned} R_{bsn} &= 0.60(58\text{ ksi})(0.935\text{ in.}^2) + 1.0(58\text{ ksi})(10.9\text{ in.}^2) \le 0.60(36\text{ ksi})(1.56\text{ in.}^2) + 1.0(58\text{ ksi})(10.9\text{ in.}^2) \\ &= 665\text{ kips} < 666\text{ kips} \end{aligned}$$

Therefore:

$$R_{bsn} = 665\text{ kips}$$

From AISC *Specification* Section J4.3, the available strength for the limit state of block shear rupture on the angles is:

LRFD	ASD
$\phi = 0.75$ $\phi R_{bsn} = 0.75(665\text{ kips})$ $= 499\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_{bsn}}{\Omega} = \dfrac{665\text{ kips}}{2.00}$ $= 333\text{ kips} > 40\text{ kips}$ **o.k.**

The nominal strength for the limit state of block shear rupture assuming an U-shaped tearout relative to the axial load on the angles is determined as follows. The tearout pattern is shown in Figure II.A-1B-6.

$$R_{bsn} = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt}$$ (*Spec*. Eq. J4-5)

where

$$\begin{aligned} A_{gv} &= (2\text{ angles})(2\text{ planes})l_{eh}t \\ &= (2\text{ angles})(2\text{ planes})(1\tfrac{1}{4}\text{ in.})(\tfrac{5}{8}\text{ in.}) \\ &= 3.13\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= (2\text{ angles})(2\text{ planes})\left[l_{eh} - 0.5\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t \\ &= (2\text{ angles})(2\text{ planes})\left[1\tfrac{1}{4}\text{ in.} - 0.5\left(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right]\left(\tfrac{5}{8}\text{ in.}\right) \\ &= 1.88\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= (2\text{ angles})\left[12.0\text{ in.} - (n-1)\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t \\ &= (2\text{ angles})\left[12.0\text{ in.} - (5-1)\left(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right]\left(\tfrac{5}{8}\text{ in.}\right) \\ &= 10.0\text{ in.}^2 \end{aligned}$$

$$U_{bs} = 1.0$$

and

$$\begin{aligned} R_{bsn} &= 0.60(58\text{ ksi})\left(1.88\text{ in.}^2\right) + 1.0(58\text{ ksi})\left(10.0\text{ in.}^2\right) \le 0.60(36\text{ ksi})\left(3.13\text{ in.}^2\right) + 1.0(58\text{ ksi})\left(10.0\text{ in.}^2\right) \\ &= 645\text{ kips} < 648\text{ kips} \end{aligned}$$

Therefore:

$$R_{bsn} = 645\text{ kips}$$

From AISC *Specification* Section J4.3, the available strength for the limit state of block shear rupture on the angles is:

LRFD	ASD
$\phi = 0.75$ $\begin{aligned} \phi R_{bsn} &= 0.75(645\text{ kips}) \\ &= 484\text{ kips} > 60\text{ kips} \quad \textbf{o.k.} \end{aligned}$	$\Omega = 2.00$ $\begin{aligned} \frac{R_{bsn}}{\Omega} &= \frac{645\text{ kips}}{2.00} \\ &= 323\text{ kips} > 40\text{ kips} \quad \textbf{o.k.} \end{aligned}$

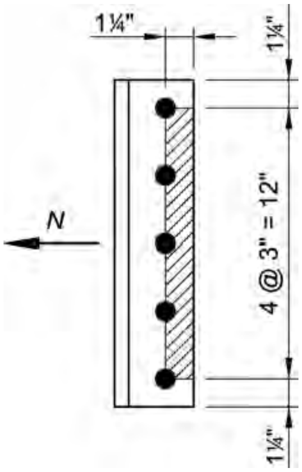


Fig. II.A-1B-6. Block shear rupture of angles for axial load only—U-shape.

Considering the interaction of shear and axial loads, apply a formulation that is similar to AISC *Manual* Equation 10-5:

LRFD	ASD
$\left(\dfrac{V_u}{\phi R_{bsv}}\right)^2+\left(\dfrac{N_u}{\phi R_{bsn}}\right)^2\le 1$	$\left(\dfrac{V_u}{R_{bsv}/\Omega}\right)^2+\left(\dfrac{N_u}{R_{bsn}/\Omega}\right)^2\le 1$
$\left(\dfrac{75\text{ kips}}{310\text{ kips}}\right)^2+\left(\dfrac{60\text{ kips}}{484\text{ kips}}\right)^2=0.0739\le 1$ **o.k.**	$\left(\dfrac{50\text{ kips}}{207\text{ kips}}\right)^2+\left(\dfrac{40\text{ kips}}{323\text{ kips}}\right)^2=0.0737\le 1$ **o.k.**

Block Shear Rupture of Angles–Outstanding Legs

The nominal strength for the limit state of block shear rupture relative to the shear load on the angles is determined as follows. The tearout pattern is shown in Figure II.A-1B-7.

$$R_n=0.60F_uA_{nv}+U_{bs}F_uA_{nt}\le 0.60F_yA_{gv}+U_{bs}F_uA_{nt} \qquad (\textit{Spec}.\text{ Eq. J4-5})$$

where

$$\begin{aligned}A_{gv}&=(2\text{ angles})(l-l_{ev})t\\&=(2\text{ angles})(14\tfrac{1}{2}\text{ in.}-1\tfrac{1}{4}\text{ in.})(\tfrac{5}{8}\text{ in.})\\&=16.6\text{ in.}^2\end{aligned}$$

$$\begin{aligned}A_{nv}&=A_{gv}-(2\text{ angles})(n-0.5)(d_h+\tfrac{1}{16}\text{ in.})t\\&=16.6\text{ in.}^2-(2\text{ angles})(5-0.5)(\tfrac{15}{16}\text{ in.}+\tfrac{1}{16}\text{ in.})(\tfrac{5}{8}\text{ in.})\\&=11.0\text{ in.}^2\end{aligned}$$

$$\begin{aligned}A_{nt}&=(2\text{ angles})\left[l_{eh}-0.5(d_h+\tfrac{1}{16}\text{ in.})\right]t\\&=(2\text{ angles})\left[1\tfrac{7}{16}\text{ in.}-0.5(\tfrac{15}{16}\text{ in.}+\tfrac{1}{16}\text{ in.})\right](\tfrac{5}{8}\text{ in.})\\&=1.17\text{ in.}^2\end{aligned}$$

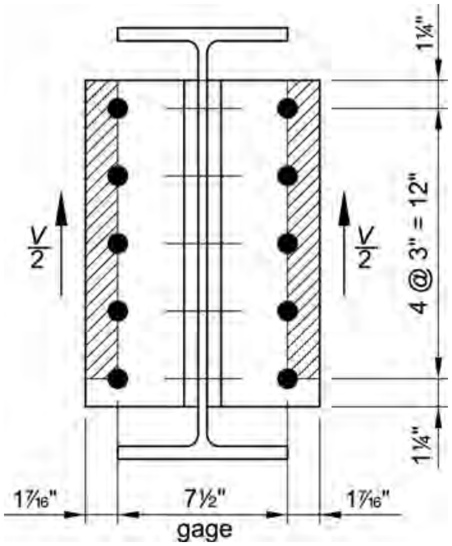


Fig. II.A-1B-7. Block shear rupture of outstanding legs of angles.

$$U_{bs} = 1.0$$

and

$$\begin{aligned} R_n &= 0.60(58\text{ ksi})\left(11.0\text{ in.}^2\right) + 1.0(58\text{ ksi})\left(1.17\text{ in.}^2\right) \le 0.60(36\text{ ksi})\left(16.6\text{ in.}^2\right) + 1.0(58\text{ ksi})\left(1.17\text{ in.}^2\right) \\ &= 451\text{ kips} > 426\text{ kips} \end{aligned}$$

Therefore:

$$R_n = 426\text{ kips}$$

From AISC *Specification* Section J4.3, the available strength for the limit state of block shear rupture on the angles is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(426\text{ kips})$ $= 320\text{ kips} > 75\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{426\text{ kips}}{2.00}$ $= 213\text{ kips} > 50\text{ kips}$ **o.k.**

Shear Strength of Beam Web

From AISC *Specification* Section J4.2(a), the available shear yield strength of the beam web is determined as follows:

$$\begin{aligned} A_{gv} &= dt_w \\ &= (18.0\text{ in.})(0.355\text{ in.}) \\ &= 6.39\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60F_y A_{gv} \\ &= 0.60(50\text{ ksi})\left(6.39\text{ in.}^2\right) \\ &= 192\text{ kips} \end{aligned}$$ (*Spec*. Eq. J4-3)

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(192\text{ kips})$ $= 192\text{ kips} > 75\text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{192\text{ kips}}{1.50}$ $= 128\text{ kips} > 50\text{ kips}$ **o.k.**

The limit state of shear rupture of the beam web does not apply in this example because the beam is uncoped.

Tensile Strength of Beam

From AISC *Specification* Section J4.1(a), the available tensile yielding strength of the beam is determined as follows:

$$
\begin{aligned}
R_n &= F_y A_g \\
&= (50\text{ ksi})(14.7\text{ in.}^2) \\
&= 735\text{ kips}
\end{aligned}
\qquad (\textit{Spec.}\text{ Eq. J4-1})
$$

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(735\text{ kips})$ $= 662\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{735\text{ kips}}{1.67}$ $= 440\text{ kips} > 40\text{ kips}$ **o.k.**

From AISC *Specification* Section J4.1(b), determine the available tensile rupture strength of the beam. The effective net area is $A_e = A_nU$. No cases in AISC *Specification* Table D3.1 apply to this configuration; therefore, U is determined from AISC *Specification* Section D3.

$$
\begin{aligned}
A_n &= A_g - n(d_h + {}^1\!/\!_{16}\text{ in.})(t_w) \\
&= 14.7\text{ in.}^2 - 5({}^{15}\!/\!_{16}\text{ in.} + {}^1\!/\!_{16}\text{ in.})(0.355\text{ in.}) \\
&= 12.9\text{ in.}^2
\end{aligned}
$$

As stated in AISC *Specification* Section D3, the value of U can be determined as the ratio of the gross area of the connected element (beam web) to the member gross area.

$$
\begin{aligned}
U &= \frac{(d - 2t_f)(t_w)}{A_g} \\
&= \frac{[18.0\text{ in.} - 2(0.570\text{ in.})](0.355\text{ in.})}{14.7\text{ in.}^2} \\
&= 0.407
\end{aligned}
$$

$$
\begin{aligned}
A_e &= A_nU \\
&= (12.9\text{ in.}^2)(0.407) \\
&= 5.25\text{ in.}^2
\end{aligned}
\qquad (\textit{Spec.}\text{ Eq. D3-1})
$$

$$
\begin{aligned}
R_n &= F_u A_e \\
&= (65\text{ ksi})(5.25\text{ in.}^2) \\
&= 341\text{ kips}
\end{aligned}
\qquad (\textit{Spec.}\text{ Eq. J4-2})
$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(341\text{ kips})$ $= 256\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{341\text{ kips}}{2.00}$ $= 171\text{ kips} > 40\text{ kips}$ **o.k.**

Block Shear Rupture Strength of Beam Web

Block shear rupture is only applicable in the direction of the axial load, because the beam is uncoped and the limit state is not applicable for an uncoped beam subject to vertical shear. Assuming a U-shaped tearout relative to the axial load, and assuming a horizontal edge distance of l_{eh} = 1¾ in. – ¼ in. = 1½ in. to account for a possible beam underrun of ¼ in., the block shear rupture strength is:

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad \text{(}\textit{Spec}\text{. Eq. J4-5)}$$

where

$$\begin{aligned} A_{gv} &= (2)l_{eh}t_w \\ &= (2)(1\tfrac{1}{2}\text{ in.})(0.355\text{ in.}) \\ &= 1.07\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - (2)(0.5)(d_h + \tfrac{1}{16}\text{ in.})t_w \\ &= 1.07\text{ in.}^2 - (2)(0.5)(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})(0.355\text{ in.}) \\ &= 0.715\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= \left[12.0\text{ in.} - (n-1)(d_h + \tfrac{1}{16}\text{ in.})\right]t_w \\ &= \left[12.0\text{ in.} - (5-1)(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})\right](0.355\text{ in.}) \\ &= 2.84\text{ in.}^2 \end{aligned}$$

$$U_{bs} = 1.0$$

and

$$\begin{aligned} R_n &= 0.60(65\text{ ksi})(0.710\text{ in.}^2) + 1.0(65\text{ ksi})(2.84\text{ in.}^2) \le 0.60(50\text{ ksi})(1.07\text{ in.}^2) + 1.0(65\text{ ksi})(2.84\text{ in.}^2) \\ &= 212\text{ kips} < 217\text{ kips} \end{aligned}$$

Therefore:

$$R_n = 212\text{ kips}$$

From AISC *Specification* Section J4.3, the available strength for the limit state of block shear rupture of the beam web is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(212\text{ kips})$ $= 159\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{212\text{ kips}}{2.00}$ $= 106\text{ kips} > 40\text{ kips}$ **o.k.**

Conclusion

The connection is found to be adequate as given for the applied loads.

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EXAMPLE II.A-1C ALL-BOLTED DOUBLE-ANGLE CONNECTION—STRUCTURAL INTEGRITY CHECK

Given:

Verify the all-bolted double-angle connection from Example II.A-1B, as shown in Figure II.A-1C-1, for the structural integrity provisions of AISC *Specification* Section B3.9. The connection is verified as a beam and girder end connection and as an end connection of a member bracing a column. Note that these checks are necessary when design for structural integrity is required by the applicable building code.

The beam is an ASTM A992 W18×50 and the angles are ASTM A36 material.

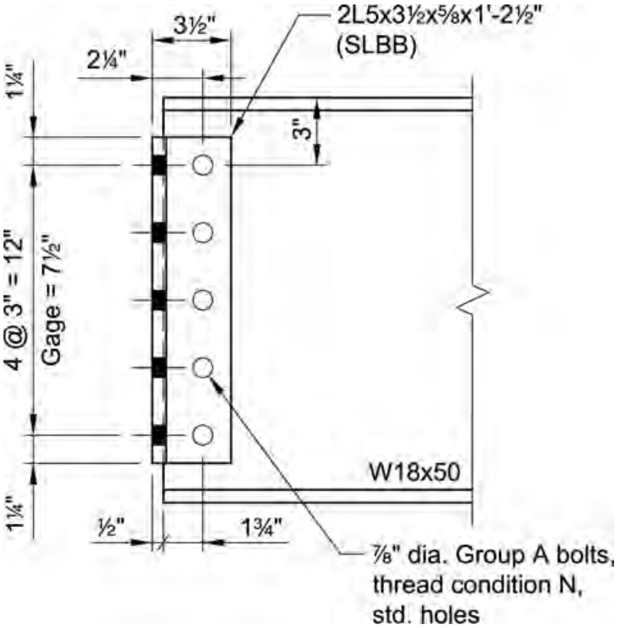

Fig. II.A-1C-1. Connection geometry for Example II.A-1C.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Angle
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W18x50
$t_w = 0.355$ in.

From AISC *Specification* Table J3.3, the hole diameter for ⅞-in.-diameter bolts with standard holes is:

d_h = 15⁄16 in.

Beam and Girder End Connection

From Example II.A-1B, the required shear strength is:

LRFD	ASD
$V_u = 75$ kips	$V_a = 50$ kips

From AISC *Specification* Section B3.9(b), the required axial tensile strength is:

LRFD	ASD
$T_u = \frac{2}{3}V_u \geq 10\text{ kips}$ $= \frac{2}{3}(75\text{ kips}) > 10\text{ kips}$ $= 50\text{ kips} > 10\text{ kips}$ Therefore: $T_u = 50$ kips	$T_a = V_a \geq 10\text{ kips}$ $= 50\text{ kips} > 10\text{ kips}$ Therefore: $T_a = 50$ kips

From AISC *Specification* Section B3.9, these strength requirements are evaluated independently from other strength requirements.

Bolt Shear

From AISC *Specification* Section J3.6, the nominal bolt shear strength is:

F_{nv} = 54 ksi, from AISC *Specification* Table J3.2

$$
\begin{aligned}
T_n &= nF_{nv}A_b\left(2\text{ shear planes}\right) \\
&= (5\text{ bolts})(54\text{ ksi})\left(0.601\text{ in.}^2\right)(2\text{ shear planes}) \\
&= 325\text{ kips}
\end{aligned}
$$

(from *Spec.* Eq. J3-1)

Bolt Tension

From AISC *Specification* Section J3.6, the nominal bolt tensile strength is:

F_{nt} = 90 ksi, from AISC *Specification* Table J3.2

$$
\begin{aligned}
T_n &= nF_{nt}A_b \\
&= (10\text{ bolts})(90\text{ ksi})\left(0.601\text{ in.}^2\right) \\
&= 541\text{ kips}
\end{aligned}
$$

(from *Spec.* Eq. J3-1)

Bolt Bearing and Tearout

From AISC *Specification* Section B3.9, for the purpose of satisfying structural integrity requirements, inelastic deformations of the connection are permitted; therefore, AISC *Specification* Equations J3-6b and J3-6d are used to determine the nominal bearing and tearout strength. By inspection the beam web will control. For bolt bearing on the beam web:

$$\begin{aligned} T_n &= (5\text{ bolts})3.0dt_wF_u \\ &= (5\text{ bolts})(3.0)(⅞\text{ in.})(0.355\text{ in.})(65\text{ ksi}) \\ &= 303\text{ kips} \end{aligned}$$

(from *Spec*. Eq. J3-6b)

For bolt tearout on the beam web (including a ¼-in. tolerance to account for possible beam underrun):

$$\begin{aligned} l_c &= l_{eh} - 0.5d_h \\ &= (1¾\text{ in.} - ¼\text{ in.}) - 0.5(^{15}\!/_{16}\text{ in.}) \\ &= 1.03\text{ in.} \end{aligned}$$

$$\begin{aligned} T_n &= (5\text{ bolts})1.5l_ct_wF_u \\ &= (5\text{ bolts})(1.5)(1.03\text{ in.})(0.355\text{ in.})(65\text{ ksi}) \\ &= 178\text{ kips} \end{aligned}$$

(from *Spec*. Eq. J3-6d)

Angle Bending and Prying Action

From AISC *Manual* Part 9, the nominal strength of the angles accounting for prying action is determined as follows:

$$\begin{aligned} a &= \frac{2(angle\ leg) + t_w - gage}{2} \\ &= \frac{2(5\text{ in.}) + 0.355\text{ in.} - 7½\text{ in.}}{2} \\ &= 1.43\text{ in.} \end{aligned}$$

$$\begin{aligned} b &= \frac{gage - t_w - t}{2} \\ &= \frac{7½\text{ in.} - 0.355\text{ in.} - ⅝\text{ in.}}{2} \\ &= 3.26\text{ in.} \end{aligned}$$

$$\begin{aligned} a' &= a + \frac{d_b}{2} \le 1.25b + \frac{d_b}{2} \\ &= 1.43\text{ in.} + \frac{⅞\text{ in.}}{2} \le 1.25(3.26\text{ in.}) + \frac{⅞\text{ in.}}{2} \\ &= 1.87\text{ in.} < 4.51\text{ in.} \\ &= 1.87\text{ in.} \end{aligned}$$

(*Manual* Eq. 9-23)

$$\begin{aligned} b' &= \left(b - \frac{d_b}{2}\right) \\ &= 3.26\text{ in.} - \frac{⅞\text{ in.}}{2} \\ &= 2.82\text{ in.} \end{aligned}$$

(*Manual* Eq. 9-18)

$$\begin{aligned}\rho &= \frac{b'}{a'} \\ &= \frac{2.82\text{ in.}}{1.87\text{ in.}} \\ &= 1.51\end{aligned}$$ (*Manual* Eq. 9-22)

Note that end distances of 1¼ in. are used on the angles, so p is the average pitch of the bolts:

$$\begin{aligned}p &= \frac{l}{n} \\ &= \frac{14\tfrac{1}{2}\text{ in.}}{5\text{ bolts}} \\ &= 2.90\text{ in.}\end{aligned}$$

Check:

$p < s = 3.00$ in. **o.k.**

$$\begin{aligned}d' &= d_h \\ &= \tfrac{15}{16}\text{ in.}\end{aligned}$$

$$\begin{aligned}\delta &= 1 - \frac{d'}{p} \\ &= 1 - \frac{\tfrac{15}{16}\text{ in.}}{2.90\text{ in.}} \\ &= 0.677\end{aligned}$$ (*Manual* Eq. 9-20)

$$\begin{aligned}B_n &= F_{nt} A_b \\ &= (90\text{ ksi})\left(0.601\text{ in.}^2\right) \\ &= 54.1\text{ kips/bolt}\end{aligned}$$

$$\begin{aligned}t_c &= \sqrt{\frac{4B_n b'}{pF_u}} \\ &= \sqrt{\frac{4(54.1\text{ kips/bolt})(2.82\text{ in.})}{(2.90\text{ in.})(58\text{ ksi})}} \\ &= 1.90\text{ in.}\end{aligned}$$ (from *Manual* Eq. 9-26)

$$\begin{aligned}\alpha' &= \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] \\ &= \frac{1}{0.677(1+1.51)}\left[\left(\frac{1.90\text{ in.}}{\tfrac{5}{8}\text{ in.}}\right)^2 - 1\right] \\ &= 4.85\end{aligned}$$ (*Manual* Eq. 9-28)

Because $\alpha' > 1$, the angles have insufficient strength to develop the bolt strength, therefore:

$$
\begin{aligned}
Q &= \left(\frac{t}{t_c}\right)^2 (1+\delta) \\
&= \left(\frac{\text{5⁄8 in.}}{\text{1.90 in.}}\right)^2 (1+0.677) \\
&= 0.181
\end{aligned}
$$

$$
\begin{aligned}
T_n &= nB_nQ && \text{(from } \textit{Manual} \text{ Eq. 9-27)} \\
&= (10\text{ bolts})(54.1\text{ kips/bolt})(0.181) \\
&= 97.9\text{ kips}
\end{aligned}
$$

Note: The 97.9 kips includes any prying forces so there is no need to calculate the prying force per bolt, q_r.

Tensile Yielding of Angles

From AISC *Specification* Section J4.1, the nominal tensile yielding strength of the angles is determined as follows:

$$
\begin{aligned}
A_g &= (2\text{ angles})lt \\
&= (2\text{ angles})(14\tfrac{1}{2}\text{ in.})(\tfrac{5}{8}\text{ in.}) \\
&= 18.1\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
T_n &= F_yA_g && \text{(from } \textit{Spec.} \text{ Eq. J4-1)} \\
&= (36\text{ ksi})(18.1\text{ in.}^2) \\
&= 652\text{ kips}
\end{aligned}
$$

Tensile Rupture of Angles

From AISC *Specification* Section J4.1, the nominal tensile rupture strength of the angles is determined as follows:

$$
\begin{aligned}
A_n &= (2\text{ angles})\left[l - n\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t \\
&= (2\text{ angles})\left[14\tfrac{1}{2}\text{ in.} - 5\left(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right](\tfrac{5}{8}\text{ in.}) \\
&= 11.9\text{ in.}^2
\end{aligned}
$$

AISC *Specification* Table D3.1, Case 1 applies in this case because tension load is transmitted directly to the cross-section element by fasteners; therefore, $U = 1.0$.

$$
\begin{aligned}
A_e &= A_nU && (\textit{Spec.} \text{ Eq. D3-1)} \\
&= (11.9\text{ in.}^2)(1.0) \\
&= 11.9\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
T_n &= F_uA_e && \text{(from } \textit{Spec.} \text{ Eq. J4-2)} \\
&= (58\text{ ksi})(11.9\text{ in.}^2) \\
&= 690\text{ kips}
\end{aligned}
$$

Block Shear Rupture

By inspection, block shear rupture of the beam web will control. From AISC *Specification* Section J4.3, the available block shear rupture strength of the beam web is determined as follows (account for possible ¼-in. beam underrun):

$$T_n = 0.60F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.60F_y A_{gv} + U_{bs} F_u A_{nt} \qquad \text{(from } \textit{Spec.} \text{ Eq. J4-5)}$$

where

$$\begin{aligned} A_{gv} &= 2l_{eh}t_w \\ &= 2\left(1\tfrac{3}{4}\text{ in.} - \tfrac{1}{4}\text{ in.}\right)(0.355\text{ in.}) \\ &= 1.07\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= 2\left[l_{eh} - 0.5\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t_w \\ &= 2\left[\left(1\tfrac{3}{4}\text{ in.} - \tfrac{1}{4}\text{ in.}\right) - 0.5\left(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right](0.355\text{ in.}) \\ &= 0.710\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= \left[12.0\text{ in.} - 4\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t_w \\ &= \left[12.0\text{ in.} - 4\left(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right](0.355\text{ in.}) \\ &= 2.84\text{ in.}^2 \end{aligned}$$

$$U_{bs} = 1.0$$

and

$$\begin{aligned} T_n &= 0.60(65\text{ ksi})\left(0.710\text{ in.}^2\right) + 1.0(65\text{ ksi})\left(2.84\text{ in.}^2\right) \le 0.60(50\text{ ksi})\left(1.07\text{ in.}^2\right) + 1.0(65\text{ ksi})\left(2.84\text{ in.}^2\right) \\ &= 212\text{ kips} < 217\text{ kips} \end{aligned}$$

Therefore:

$$T_n = 212\text{ kips}$$

Nominal Tensile Strength

The controlling nominal tensile strength, T_n, is the least of those previously calculated:

$$\begin{aligned} T_n &= \min\{325\text{ kips},\ 541\text{ kips},\ 97.9\text{ kips},\ 652\text{ kips},\ 690\text{ kips},\ 212\text{ kips}\} \\ &= 97.9\text{ kips} \end{aligned}$$

LRFD	ASD
$T_n = 97.9\text{ kips} > 50\text{ kips}$ **o.k.**	$T_n = 97.9\text{ kips} > 50\text{ kips}$ **o.k.**

Column Bracing

From AISC *Specification* Section B3.9(c), the minimum nominal tensile strength for the connection of a member bracing a column is equal to 1% of two-thirds of the required column axial strength for LRFD and equal to 1% of the required column axial for ASD. These requirements are evaluated independently from other strength requirements.

The maximum column axial force this connection is able to brace is determined as follows:

LRFD	ASD
$T_n \geq 0.01\left(\frac{2}{3}\right)P_u$ Solving for the column axial force: $P_u \leq 100\left(\frac{3}{2}\right)T_n$ $= 100\left(\frac{3}{2}\right)(97.9 \text{ kips})$ $= 14{,}700 \text{ kips}$	$T_n \geq 0.01P_a$ Solving for the column axial force: $P_a \leq 100T_n$ $= 100(97.9 \text{ kips})$ $= 9{,}790 \text{ kips}$

As long as the required column axial strength is less than P_u = 14,700 kips or P_a = 9,790 kips, this connection is an adequate column brace.

EXAMPLE II.A-2A BOLTED/WELDED DOUBLE-ANGLE CONNECTION

Given:

Using the tables in AISC *Manual* Part 10, verify the available strength of a double-angle shear connection with welds in the support legs (welds B) and bolts in the supported-beam-web legs, as shown in Figure II.A-2A-1. The ASTM A992 W36×231 beam is attached to an ASTM A992 W14×90 column flange supporting the following beam end reactions:

$R_D = 37.5$ kips
$R_L = 113$ kips

Use ASTM A36 angles and 70-ksi weld electrodes.

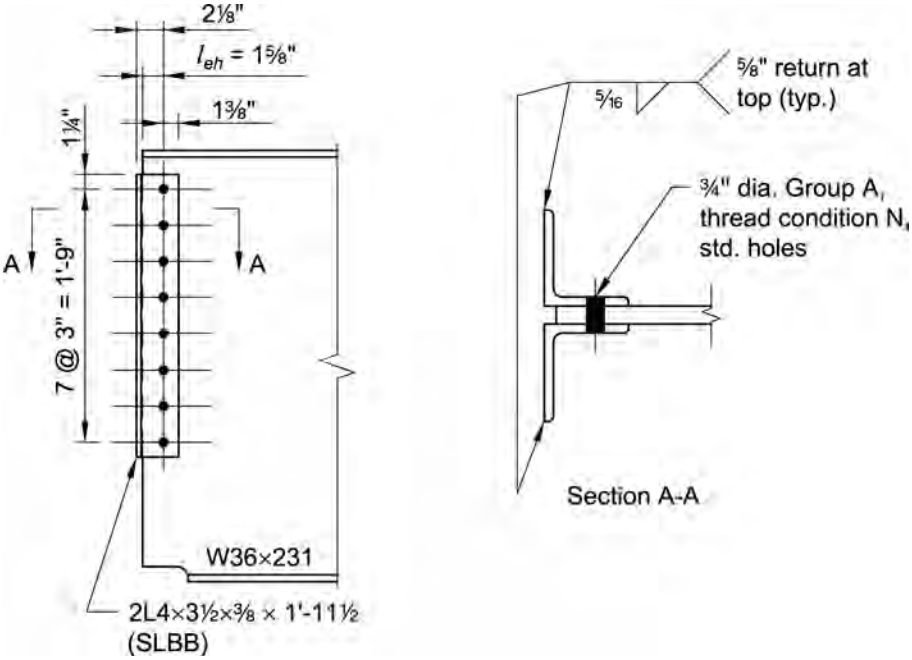

Fig. II.A-2A-1. Connection geometry for Example II.A-2A.

Note: Bottom flange coped for erection.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam and column
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Angles
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W36×231
$t_w = 0.760$ in.

Column
W14×90
$t_f = 0.710$ in.

From AISC *Specification* Table J3.3, the hole diameter for ¾-in.-diameter bolts with standard holes is:

$d_h = {}^{13}\!/_{16}$ in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(37.5 \text{ kips}) + 1.6(113 \text{ kips})$ $= 226 \text{ kips}$	$R_a = 37.5 \text{ kips} + 113 \text{ kips}$ $= 151 \text{ kips}$

Weld Design

Use AISC *Manual* Table 10-2 (welds B) with n = 8. Try ⁵⁄₁₆-in. weld size, l = 23½ in. From AISC *Manual* Table 10-2, the minimum support thickness is:

$t_{min} = 0.238 \text{ in.} < 0.710 \text{ in.}$ **o.k.**

LRFD	ASD
$\phi R_n = 279 \text{ kips} > 226 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 186 \text{ kips} > 151 \text{ kips}$ **o.k.**

Angle Thickness

From AISC *Specification* Section J2.2b, the minimum angle thickness for a ⁵⁄₁₆-in. fillet weld is:

$$\begin{aligned} t &= w + {}^{1}\!/_{16} \text{ in.} \\ &= {}^{5}\!/_{16} \text{ in.} + {}^{1}\!/_{16} \text{ in.} \\ &= {}^{3}\!/_{8} \text{ in.} \end{aligned}$$

Try 2L4×3½×⅜ (SLBB).

Angle and Bolt Design

AISC *Manual* Table 10-1 includes checks for bolt shear, bolt bearing and tearout on the angles, shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles.

Check 8 rows of bolts and ⅜-in. angle thickness.

LRFD	ASD
$\phi R_n = 284 \text{ kips} > 226 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 189 \text{ kips} > 151 \text{ kips}$ **o.k.**

Beam Web Strength

The available beam web strength is the lesser of the limit states of block shear rupture, shear yielding, shear rupture, and the sum of the effective strengths of the individual fasteners. In this example, because of the relative size of the cope to the overall beam size, the coped section will not control, therefore, the strength of the bolt group will control (When this cannot be determined by inspection, see AISC *Manual* Part 9 for the design of the coped section). From the Commentary to AISC *Specification* Section J3.6, the strength of the bolt group is taken as the sum of the effective strengths of the individual fasteners. The effective strength of an individual fastener is the lesser of the shear strength, the bearing strength at the bolt holes, and the tearout strength at the bolt holes.

Bolt Shear

From AISC *Manual* Table 7-1, the available shear strength per bolt for ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) in double shear is:

LRFD	ASD
$\phi R_n = 35.8\text{ kips/bolt}$	$\dfrac{R_n}{\Omega} = 23.9\text{ kips/bolt}$

Bolt Bearing on Beam Web

The nominal bearing strength of the beam web per bolt is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration:

$$\begin{aligned} r_n &= 2.4dtF_u \\ &= 2.4(\tfrac{3}{4}\text{ in.})(0.760\text{ in.})(65\text{ ksi}) \\ &= 88.9\text{ kips/bolt} \end{aligned}$$

(*Spec.* Eq. J3-6a)

From AISC *Specification* Section J3.10, the available bearing strength of the beam web per bolt is:

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(88.9\text{ kips/bolt})$ $= 66.7\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{88.9\text{ kips/bolt}}{2.00}$ $= 44.5\text{ kips/bolt}$

Bolt Tearout on Beam Web

The available tearout strength of the beam web per bolt is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration:

$$\begin{aligned} l_c &= 3.00\text{ in.} - \tfrac{13}{16}\text{ in.} \\ &= 2.19\text{ in.} \end{aligned}$$

$$\begin{aligned} r_n &= 1.2l_ctF_u \\ &= 1.2(2.19\text{ in.})(0.760\text{ in.})(65\text{ ksi}) \\ &= 130\text{ kips/bolt} \end{aligned}$$

(*Spec.* Eq. J3-6c)

From AISC *Specification* Section J3.10, the available tearout strength of the beam web per bolt is:

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(130 \text{ kips/bolt})$ $= 97.5 \text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{130 \text{ kips/bolt}}{2.00}$ $= 65.0 \text{ kips/bolt}$

Bolt shear strength is the governing limit state for all bolts at the beam web. Bolt shear strength is one of the limit states included in the capacities shown in Table 10-1 as used above; thus, the effective strength of the fasteners is adequate.

Available strength at the column flange

Since the thickness of the column flange, t_f = 0.710 in., is greater than the thickness of the angles, t = ⅜ in., shear will control for the angles. The column flange is adequate for the required loading.

Summary

The connection is found to be adequate as given for the applied loads.

EXAMPLE II.A-2B BOLTED/WELDED DOUBLE-ANGLE CONNECTION SUBJECT TO AXIAL AND SHEAR LOADING

Given:

Verify the available strength of a double-angle connection with welds in the supported-beam-web legs and bolts in the outstanding legs for an ASTM A992 W18×50 beam, as showin in Figure II.A-2B-1, to support the following beam end reactions:

LRFD	ASD
Shear, V_u = 75 kips Axial tension, N_u = 60 kips	Shear, V_a = 50 kips Axial tension, N_a = 40 kips

Use ASTM A36 angles and 70-ksi electrodes.

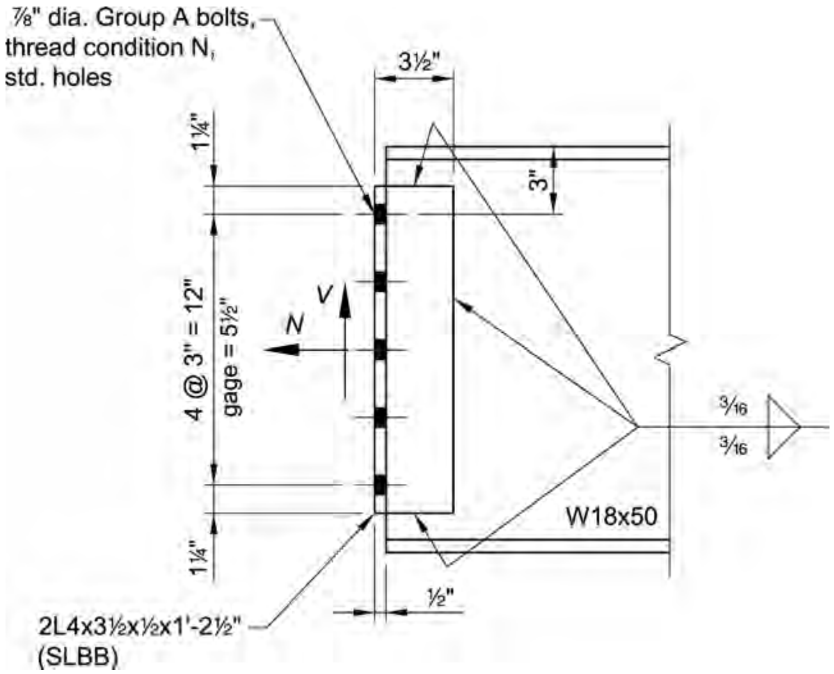

Fig. II.A-2B-1. Connection geometry for Example II.A-2B.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Angles
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W18×50
$A_g = 14.7 \text{ in.}^2$
$d = 18.0 \text{ in.}$
$t_w = 0.355 \text{ in.}$
$b_f = 7.50 \text{ in.}$
$t_f = 0.570 \text{ in.}$

From AISC *Specification* Table J3.3, the hole diameter for ⅞-in.-diameter bolts with standard holes is:

$d_h = {}^{15}\!/_{16} \text{ in.}$

The resultant load is:

LRFD	ASD
$R_u = \sqrt{V_u^2 + N_u^2}$ $= \sqrt{(75 \text{ kips})^2 + (60 \text{ kips})^2}$ $= 96.0 \text{ kips}$	$R_a = \sqrt{V_a^2 + N_a^2}$ $= \sqrt{(50 \text{ kips})^2 + (40 \text{ kips})^2}$ $= 64.0 \text{ kips}$

The following bolt shear, bearing and tearout calculations are for a pair of bolts.

Bolt Shear

From AISC *Manual* Table 7-1, the available shear strength for ⅞-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) in double shear (or pair of bolts):

LRFD	ASD
$\phi r_n = 48.7 \text{ kips (for pair of bolts)}$	$\dfrac{r_n}{\Omega} = 32.5 \text{ kips (for pair of bolts)}$

Bolt Bearing on Angles

The available bearing strength of the double angle is determined from AISC *Specification* Section J3.10, assuming deformation at the bolt hole is a design consideration:

$$\begin{aligned} r_n &= (2 \text{ bolts})2.4dtF_u \\ &= (2 \text{ bolts})(2.4)(⅞ \text{ in.})(½ \text{ in.})(58 \text{ ksi}) \\ &= 122 \text{ kips (for pair of bolts)} \end{aligned}$$ (from *Spec*. Eq. J3-6a)

The available bearing strength for a pair of bolts is:

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(122 \text{ kips})$ $= 91.5 \text{ kips (for pair of bolts)}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{122 \text{ kips}}{2.00}$ $= 61.0 \text{ kips (for pair of bolts)}$

The bolt shear strength controls over bearing in the angles.

Bolt Tearout on Angles

The available tearout strength of the angle is determined from AISC *Specification* Section J3.10, assuming deformation at the bolt hole is a design consideration:

For the edge bolt:

$$\begin{aligned} l_c &= l_e - 0.5d_h \\ &= 1\tfrac{1}{4}\text{ in.} - 0.5\left(\tfrac{15}{16}\text{ in.}\right) \\ &= 0.781\text{ in.} \end{aligned}$$

$$\begin{aligned} r_n &= (2\text{ bolts})1.2l_c tF_u \\ &= (2\text{ bolts})(1.2)(0.781\text{ in.})(\tfrac{1}{2}\text{ in.})(58\text{ ksi}) \\ &= 54.4\text{ kips (for pair of bolts)} \end{aligned}$$ (from *Spec*. Eq. J3-6c)

The available tearout strength of the angles for a pair of edge bolts is:

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi r_n = 0.75(54.4\text{ kips})$ $= 40.8\text{ kips}$	$\dfrac{r_n}{\Omega} = \dfrac{54.4\text{ kips}}{2.00}$ $= 27.2\text{ kips}$

The tearout strength controls over bolt shear and bearing for the edge bolts in the angles.

For the other bolts:

$$\begin{aligned} l_c &= s - d_h \\ &= 3\text{ in.} - \tfrac{15}{16}\text{ in.} \\ &= 2.06\text{ in.} \end{aligned}$$

$$\begin{aligned} r_n &= (2\text{ bolts})1.2l_c tF_u \\ &= (2\text{ bolts})(1.2)(2.06\text{ in.})(\tfrac{1}{2}\text{ in.})(58\text{ ksi}) \\ &= 143\text{ kips (for pair of bolts)} \end{aligned}$$ (*Spec*. Eq. J3-6c)

The available tearout strength for a pair of other bolts is:

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi r_n = 0.75(143\text{ kips})$ $= 107\text{ kips (for pair of bolts)}$	$\dfrac{r_n}{\Omega} = \dfrac{143\text{ kips}}{2.00}$ $= 71.5\text{ kips (for pair of bolts)}$

Bolt shear strength controls over tearout and bearing strength for the other bolts in the angles.

Strength of Bolted Connection

The effective strength for the bolted connection at the angles is determined by summing the effective strength for each bolt using the minimum available strength calculated for bolt shear, bearing on the angles, and tearout on the angles.

LRFD	ASD
$\phi R_n = (1\text{ bolt})(40.8\text{ kips})$ $+(4\text{ bolts})(48.7\text{ kips})$ $= 236\text{ kips} > 75\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = (1\text{ bolt})(27.2\text{ kips})$ $+(4\text{ bolts})(32.5\text{ kips})$ $= 157\text{ kips} > 50\text{ kips}$ **o.k.**

Shear and Tension Interaction in Bolts

The required shear stress for each bolt is determined as follows:

$$f_{rv} = \frac{V_r}{nA_b}$$

where

$A_b = 0.601\text{ in.}^2$ (from AISC *Manual* Table 7-1)
$n = 10\text{ bolts}$

LRFD	ASD
$f_{rv} = \dfrac{75\text{ kips}}{(10\text{ bolts})(0.601\text{ in.}^2)}$ $= 12.5\text{ ksi}$	$f_{rv} = \dfrac{50\text{ kips}}{(10\text{ bolts})(0.601\text{ in.}^2)}$ $= 8.32\text{ ksi}$

The nominal tensile stress modified to include the effects of shear stress is determined from AISC *Specification* Section J3.7 as follows. From AISC *Specification* Table J3.2:

$F_{nt} = 90\text{ ksi}$
$F_{nv} = 54\text{ ksi}$

LRFD	ASD
$\phi = 0.75$ $F'_{nt} = 1.3F_{nt} - \dfrac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt}$ (*Spec*. Eq. J3-3a) $= 1.3(90\text{ ksi}) - \dfrac{90\text{ ksi}}{0.75(54\text{ ksi})}(12.5\text{ ksi}) \le 90\text{ ksi}$ $= 89.2\text{ ksi} < 90\text{ ksi}$ **o.k.**	$\Omega = 2.00$ $F'_{nt} = 1.3F_{nt} - \dfrac{\Omega F_{nt}}{F_{nv}} f_{rv} \le F_{nt}$ (*Spec*. Eq. J3-3b) $= 1.3(90\text{ ksi}) - \dfrac{2.00(90\text{ ksi})}{54\text{ ksi}}(8.32\text{ ksi}) \le 90\text{ ksi}$ $= 89.3\text{ ksi} < 90\text{ ksi}$ **o.k.**

Using the value of $F'_{nt} = 89.2$ ksi determined for LRFD, the nominal tensile strength of one bolt is:

$$\begin{aligned} r_n &= F'_{nt}A_b \\ &= (89.2\text{ ksi})(0.601\text{ in.}^2) \\ &= 53.6\text{ kips} \end{aligned}$$

(*Spec*. Eq. J3-2)

The available tensile strength due to combined tension and shear is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = n\phi r_n$ $= (10\text{ bolts})(0.75)(53.6\text{ kips})$ $= 402\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = n\dfrac{r_n}{\Omega}$ $= (10\text{ bolts})\left(\dfrac{53.6\text{ kips}}{2.00}\right)$ $= 268\text{ kips} > 40\text{ kips}$ **o.k.**

Prying Action on Bolts

From AISC *Manual* Part 9, the available tensile strength of the bolts in the outstanding angle legs taking prying action into account is determined as follows:

$$\begin{aligned} a &= \frac{angle\ leg\,(2) + t_w - gage}{2} \\ &= \frac{(4.00\text{ in.})(2) + 0.355\text{ in.} - 5\tfrac{1}{2}\text{ in.}}{2} \\ &= 1.43\text{ in.} \end{aligned}$$

Note: If the distance from the bolt centerline to the edge of the supporting element is smaller than a = 1.43 in., use the smaller a in the following calculation.

$$\begin{aligned} b &= \frac{gage - t_w - t}{2} \\ &= \frac{5\tfrac{1}{2}\text{ in.} - 0.355\text{ in.} - \tfrac{1}{2}\text{ in.}}{2} \\ &= 2.32\text{ in.} \end{aligned}$$

$$\begin{aligned} a' &= \left(a + \frac{d_b}{2}\right) \le \left(1.25b + \frac{d_b}{2}\right) \\ &= 1.43\text{ in.} + \frac{\tfrac{7}{8}\text{ in.}}{2} \le 1.25(2.32\text{ in.}) + \frac{\tfrac{7}{8}\text{ in.}}{2} \\ &= 1.87\text{ in.} < 3.34\text{ in.} \\ &= 1.87\text{ in.} \end{aligned}$$ (*Manual* Eq. 9-23)

$$\begin{aligned} b' &= \left(b - \frac{d_b}{2}\right) \\ &= 2.32\text{ in.} - \frac{\tfrac{7}{8}\text{ in.}}{2} \\ &= 1.88\text{ in.} \end{aligned}$$ (*Manual* Eq. 9-18)

$$\begin{aligned} \rho &= \frac{b'}{a'} \\ &= \frac{1.88\text{ in.}}{1.87\text{ in.}} \\ &= 1.01 \end{aligned}$$ (*Manual* Eq. 9-22)

Note that end distances of 1¼ in. are used on the angles, so *p* is the average pitch of the bolts:

$$\begin{aligned} p &= \frac{l}{n} \\ &= \frac{14\tfrac{1}{2}\text{ in.}}{5} \\ &= 2.90\text{ in.} \end{aligned}$$

Check:

$p \le s$

$2.90\text{ in.} < 3\text{ in.}$ **o.k.**

$$\begin{aligned} d' &= d_h \\ &= \tfrac{15}{16}\text{ in.} \end{aligned}$$

$$\begin{aligned} \delta &= 1 - \frac{d'}{p} \\ &= 1 - \frac{\tfrac{15}{16}\text{ in.}}{2.90\text{ in.}} \\ &= 0.677 \end{aligned}$$ (*Manual* Eq. 9-20)

The angle thickness required to develop the available strength of the bolt with no prying action as follows:

LRFD	ASD
$B_c = 40.2$ kips/bolt (calculated previously) $\phi = 0.90$ $t_c = \sqrt{\dfrac{4B_c b'}{\phi p F_u}}$ (*Manual* Eq. 9-26a) $= \sqrt{\dfrac{4(40.2\text{ kips/bolt})(1.88\text{ in.})}{0.90(2.90\text{ in.})(58\text{ ksi})}}$ $= 1.41\text{ in.}$	$B_c = 26.8$ kips/bolt (calculated previously) $\Omega = 1.67$ $t_c = \sqrt{\dfrac{\Omega 4B_c b'}{p F_u}}$ (*Manual* Eq. 9-26b) $= \sqrt{\dfrac{1.67(4)(26.8\text{ kips/bolt})(1.88\text{ in.})}{(2.90\text{ in.})(58\text{ ksi})}}$ $= 1.41\text{ in.}$

$$\begin{aligned} \alpha' &= \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] \\ &= \frac{1}{0.677(1+1.01)}\left[\left(\frac{1.41\text{ in.}}{\tfrac{1}{2}\text{ in.}}\right)^2 - 1\right] \\ &= 5.11 \end{aligned}$$ (*Manual* Eq. 9-28)

Because $\alpha' > 1$, the angles have insufficient strength to develop the bolt strength, therefore:

$$\begin{aligned} Q &= \left(\frac{t}{t_c}\right)^2 (1+\delta) \\ &= \left(\frac{\tfrac{1}{2}\text{ in.}}{1.41\text{ in.}}\right)^2 (1+0.677) \\ &= 0.211 \end{aligned}$$

The available tensile strength of the bolts, taking prying action into account is determined from AISC *Manual* Equation 9-27, as follows:

LRFD	ASD
$\phi r_n = B_c Q$ $= (40.2 \text{ kips/bolt})(0.211)$ $= 8.48 \text{ kips/bolt}$ $\phi R_n = n\phi r_n$ $= (10 \text{ bolts})(8.48 \text{ kips/bolt})$ $= 84.8 \text{ kips} > 60 \text{ kips}$ **o.k.**	$\dfrac{r_n}{\Omega} = B_c Q$ $= (26.8 \text{ kips/bolt})(0.211)$ $= 5.65 \text{ kips/bolt}$ $\dfrac{R_n}{\Omega} = n\dfrac{r_n}{\Omega}$ $= (10 \text{ bolts})(5.65 \text{ kips/bolt})$ $= 56.5 \text{ kips} > 40 \text{ kips}$ **o.k.**

Weld Design

The resultant load angle on the weld is:

LRFD	ASD
$\theta = \tan^{-1}\left(\dfrac{N_u}{V_u}\right)$ $= \tan^{-1}\left(\dfrac{60 \text{ kips}}{75 \text{ kips}}\right)$ $= 38.7°$	$\theta = \tan^{-1}\left(\dfrac{N_a}{V_a}\right)$ $= \tan^{-1}\left(\dfrac{40 \text{ kips}}{50 \text{ kips}}\right)$ $= 38.7°$

From AISC *Manual* Table 8-8 for Angle = 30° (which will lead to a conservative result), using total beam setback of ½ in. + ¼ in. = ¾ in. (the ¼ in. is included to account for mill underrun):

$$l = 14\tfrac{1}{2} \text{ in.}$$

$$\begin{aligned} kl &= 3\tfrac{1}{2} \text{ in.} - \tfrac{3}{4} \text{ in.} \\ &= 2.75 \text{ in.} \end{aligned}$$

$$\begin{aligned} k &= \frac{kl}{l} \\ &= \frac{2.75 \text{ in.}}{14\tfrac{1}{2} \text{ in.}} \\ &= 0.190 \end{aligned}$$

$x = 0.027$ by interpolation

$$\begin{aligned} al &= 3\tfrac{1}{2} \text{ in.} - xl \\ &= 3\tfrac{1}{2} \text{ in.} - 0.027(14\tfrac{1}{2} \text{ in.}) \\ &= 3.11 \text{ in.} \end{aligned}$$

$$\begin{aligned} a &= \frac{al}{l} \\ &= \frac{3.11\text{ in.}}{14\tfrac{1}{2}\text{ in.}} \\ &= 0.214 \end{aligned}$$

$C = 2.69$ by interpolation

The required weld size is determined using AISC *Manual* Equation 8-21, as follows:

LRFD	ASD
$D_{min} = \dfrac{R_u}{\phi C C_1 l}$ $= \dfrac{96.0\text{ kips}}{0.75(2.69)(1)(14\tfrac{1}{2}\text{ in.})(2\text{ sides})}$ $= 1.64$ sixteenths	$D_{min} = \dfrac{\Omega R_a}{C C_1 l}$ $= \dfrac{2.00(64.0\text{ kips})}{2.69(1)(14\tfrac{1}{2}\text{ in.})(2\text{ sides})}$ $= 1.64$ sixteenths

Use a 3⁄16-in. fillet weld (minimum size from AISC *Specification* Table J2.4).

Beam Web Strength at Fillet Weld

The minimum beam web thickness required to match the shear rupture strength of a weld both sides to that of the base metal is:

$$\begin{aligned} t_{min} &= \frac{6.19 D_{min}}{F_u} \\ &= \frac{6.19(1.64)}{65\text{ ksi}} \\ &= 0.156\text{ in.} < 0.355\text{ in.} \quad \textbf{o.k.} \end{aligned}$$ (from *Manual* Eq. 9-3)

Shear Strength of Angles

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the angles is determined as follows:

$$\begin{aligned} A_{gv} &= (2\text{ angles})lt \\ &= (2\text{ angles})(14\tfrac{1}{2}\text{ in.})(\tfrac{1}{2}\text{ in.}) \\ &= 14.5\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60 F_y A_{gv} \\ &= 0.60(36\text{ ksi})(14.5\text{ in.}^2) \\ &= 313\text{ kips} \end{aligned}$$ (*Spec*. Eq. J4-3)

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(313 \text{ kips})$ $= 313 \text{ kips} > 96.0 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{313 \text{ kips}}{1.50}$ $= 209 \text{ kips} > 64.0 \text{ kips}$ **o.k.**

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the angle is determined as follows. The effective net area is determined in accordance with AISC *Specification* Section B4.3b.

$$\begin{aligned} A_{nv} &= (2 \text{ angles})\left[l - n\left(d_h + \tfrac{1}{16} \text{ in.}\right)\right]t \\ &= (2 \text{ angles})\left[14\tfrac{1}{2} \text{ in.} - 5\left(\tfrac{15}{16} \text{ in.} + \tfrac{1}{16} \text{ in.}\right)\right]\left(\tfrac{1}{2} \text{ in.}\right) \\ &= 9.50 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60 F_u A_{nv} \\ &= 0.60(58 \text{ ksi})\left(9.50 \text{ in.}^2\right) \\ &= 331 \text{ kips} \end{aligned}$$

(*Spec*. Eq. J4-4)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(331 \text{ kips})$ $= 248 \text{ kips} > 96.0 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{331 \text{ kips}}{2.00}$ $= 166 \text{ kips} > 64.0 \text{ kips}$ **o.k.**

Tensile Strength of Angles—Beam Web Side

From AISC *Specification* Section J4.1(a), the available tensile yielding strength of the angles is determined as follows:

$$\begin{aligned} A_g &= (2 \text{ angles})lt \\ &= (2 \text{ angles})\left(14\tfrac{1}{2} \text{ in.}\right)\left(\tfrac{1}{2} \text{ in.}\right) \\ &= 14.5 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= F_y A_g \\ &= (36 \text{ ksi})\left(14.5 \text{ in.}^2\right) \\ &= 522 \text{ kips} \end{aligned}$$

(*Spec*. Eq. J4-1)

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(522 \text{ kips})$ $= 470 \text{ kips} > 60 \text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{522 \text{ kips}}{1.67}$ $= 313 \text{ kips} > 40 \text{ kips}$ **o.k.**

From AISC *Specification* Sections J4.1(b), the available tensile rupture strength of the angles is determined as follows:

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$$R_n = F_u A_e \qquad \text{(}\textit{Spec}\text{. Eq. J4-2)}$$

Because the angle legs are welded to the beam web there is no bolt hole reduction and $A_e = A_g$; therefore, tensile rupture will not control.

Block Shear Rupture Strength of Angles–Outstanding Legs

The nominal strength for the limit state of block shear rupture of the angles assuming an L-shaped tearout relative to shear load, is determined as follows. The tearout pattern is shown in Figure II.A-2B-2.

$$R_n = 0.60F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.60F_y A_{gv} + U_{bs} F_u A_{nt} \qquad \text{(}\textit{Spec}\text{. Eq. J4-5)}$$

where

$$\begin{aligned} l_{eh} &= \frac{2(\textit{angle leg}) + t_w - \textit{gage}}{2} \\ &= \frac{2(4 \text{ in.}) + 0.355 \text{ in.} - 5\tfrac{1}{2} \text{ in.}}{2} \\ &= 1.43 \text{ in.} \end{aligned}$$

$$\begin{aligned} A_{nt} &= (2 \text{ angles})\left[l_{eh} - 0.5\left(d_h + \tfrac{1}{16} \text{ in.}\right)\right](t) \\ &= (2 \text{ angles})\left[1.43 \text{ in.} - 0.5\left(\tfrac{15}{16} \text{ in.} + \tfrac{1}{16} \text{ in.}\right)\right]\left(\tfrac{1}{2} \text{ in.}\right) \\ &= 0.930 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{gv} &= (2 \text{ angles})\left[l_{ev} + (n-1)s\right](t) \\ &= (2 \text{ angles})\left[1\tfrac{1}{4} \text{ in.} + (5-1)(3 \text{ in.})\right]\left(\tfrac{1}{2} \text{ in.}\right) \\ &= 13.3 \text{ in.}^2 \end{aligned}$$

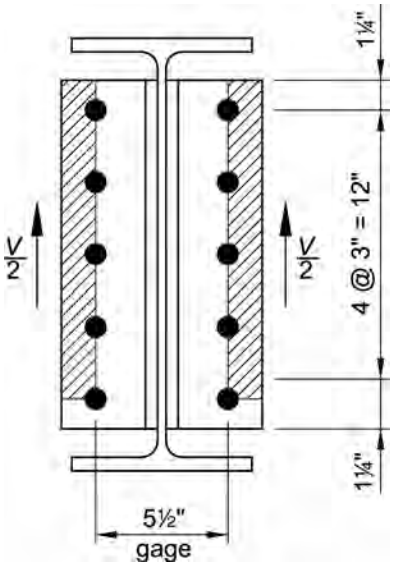


Fig. II.A-2B-2. Block shear rupture of outstanding legs of angles.

$$
\begin{aligned}
A_{nv} &= A_{gv} - (2 \text{ angles})(n - 0.5)(d_h + \tfrac{1}{16}\text{ in.})(t) \\
&= 13.3 \text{ in.}^2 - (2 \text{ angles})(5 - 0.5)(\tfrac{15}{16} \text{ in.} + \tfrac{1}{16} \text{ in.})(\tfrac{1}{2} \text{ in.}) \\
&= 8.80 \text{ in.}^2
\end{aligned}
$$

$$U_{bs} = 1.0$$

and

$$
\begin{aligned}
R_n &= 0.60(58 \text{ ksi})(8.80 \text{ in.}^2) + 1.0(58 \text{ ksi})(0.930 \text{ in.}^2) \le 0.60(36 \text{ ksi})(13.3 \text{ in.}^2) + 1.0(58 \text{ ksi})(0.930 \text{ in.}^2) \\
&= 360 \text{ kips} > 341 \text{ kips}
\end{aligned}
$$

Therefore:

$$R_n = 341 \text{ kips}$$

The available block shear rupture strength of the angles is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(341 \text{ kips})$ $= 256 \text{ kips} > 75 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{341 \text{ kips}}{2.00}$ $= 171 \text{ kips} > 50 \text{ kips}$ **o.k.**

Shear Strength of Beam

From AISC *Specification* Section J4.2(a), the available shear yield strength of the beam web is determined as follows:

$$
\begin{aligned}
A_{gv} &= dt_w \\
&= (18.0 \text{ in.})(0.355 \text{ in.}) \\
&= 6.39 \text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
R_n &= 0.60 F_y A_{gv} \qquad \text{(}\textit{Spec}\text{. Eq. J4-3)} \\
&= 0.60(50 \text{ ksi})(6.39 \text{ in.}^2) \\
&= 192 \text{ kips}
\end{aligned}
$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(192 \text{ kips})$ $= 192 \text{ kips} > 75 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{192 \text{ kips}}{1.50}$ $= 128 \text{ kips} > 50 \text{ kips}$ **o.k.**

The limit state of shear rupture of the beam web does not apply in this example because the beam is uncoped.

Block Shear Rupture Strength of Beam Web

Assuming a U-shaped tearout along the weld relative to the axial load, and a total beam setback of ¾ in. (includes ¼ in. tolerance to account for possible mill underrun), the nominal block shear rupture strength is determined as follows.

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad (Spec.\ \text{Eq. J4-5})$$

where

$$\begin{aligned} A_{nt} &= lt_w \\ &= (14\tfrac{1}{2}\text{ in.})(0.355\text{ in.}) \\ &= 5.15\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{gv} &= (2)(3\tfrac{1}{2}\text{ in.} - setback)t_w \\ &= (2)(3\tfrac{1}{2}\text{ in.} - \tfrac{3}{4}\text{ in.})(0.355\text{ in.}) \\ &= 1.95\text{ in.}^2 \end{aligned}$$

Because the angles are welded and there is no reduction for bolt holes:

$$\begin{aligned} A_{nv} &= A_{gv} \\ &= 1.95\text{ in.}^2 \end{aligned}$$

$$U_{bs} = 1$$

and

$$\begin{aligned} R_n &= 0.60(65\text{ ksi})(1.95\text{ in.}^2) + 1.0(65\text{ ksi})(5.15\text{ in.}^2) \le 0.60(50\text{ ksi})(1.95\text{ in.}^2) + 1.0(65\text{ ksi})(5.15\text{ in.}^2) \\ &= 411\text{ kips} > 393\text{ kips} \end{aligned}$$

Therefore:

$$R_n = 393\text{ kips}$$

The available block shear rupture strength of the web is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(393\text{ kips})$ $= 295\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{393\text{ kips}}{2.00}$ $= 197\text{ kips} > 40\text{ kips}$ **o.k.**

Tensile Strength of Beam

From AISC *Specification* Section J4.1(a), the available tensile yielding strength of the beam is determined from AISC *Specification* Equation J4-1:

$$\begin{aligned} R_n &= F_yA_g \qquad (Spec.\ \text{Eq. J4-1}) \\ &= (50\text{ ksi})(14.7\text{ in.}^2) \\ &= 735\text{ kips} \end{aligned}$$

The available tensile yielding strength of the beam is:

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(735 \text{ kips})$ $= 662 \text{ kips} > 60 \text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{735 \text{ kips}}{1.67}$ $= 440 \text{ kips} > 40 \text{ kips}$ **o.k.**

From AISC *Specification* Section J4.1(b), determine the available tensile rupture strength of the beam. The effective net area is $A_e = A_nU$, where U is determined from AISC *Specification* Table D3.1, Case 2. The value of $\bar{x}$ is determined by treating the W-shape as two channels back-to-back and finding the horizontal distance to the center of gravity of one of the channels from the centerline of the beam. (Note that the fillets are ignored.)

$$\begin{aligned}
\bar{x} &= \frac{\Sigma(A\bar{x})}{\Sigma A} \\
&= \frac{(0.178 \text{ in.})[18.0 \text{ in.} - 2(0.570 \text{ in.})]\left(\dfrac{0.178 \text{ in.}}{2}\right) + 2(0.570 \text{ in.})\left(\dfrac{7.50 \text{ in.}}{2}\right)\left(\dfrac{7.50 \text{ in.}/2}{2}\right)}{\left(\dfrac{14.7 \text{ in.}^2}{2}\right)} \\
&= 1.13 \text{ in.}
\end{aligned}$$

The connection length, l, used in the determination of U will be reduced by ¼ in. to account for possible mill underrun. The shear lag factor, U, is:

$$\begin{aligned}
U &= 1 - \frac{\bar{x}}{l} \\
&= 1 - \frac{1.13 \text{ in.}}{(3 \text{ in.} - \tfrac{1}{4} \text{ in.})} \\
&= 0.589
\end{aligned}$$

The minimum value of U can be determined from AISC *Specification* Section D3, where U is the ratio of the gross area of the connected element to the member gross area.

$$\begin{aligned}
U &= \frac{A_{nt}}{A_g} \\
&= \frac{(d - 2t_f)t_w}{A_g} \\
&= \frac{[18.0 \text{ in.} - 2(0.570 \text{ in.})](0.355 \text{ in.})}{14.7 \text{ in.}^2} \\
&= 0.407
\end{aligned}$$

AISC *Specification* Table D3.1, Case 2 controls, use U = 0.589. Because the angles are welded and there is no reduction for bolt holes:

$$\begin{aligned}
A_n &= A_g \\
&= 14.7 \text{ in.}^2
\end{aligned}$$

$$A_e = A_n U \qquad \text{(\textit{Spec}. Eq. D3-1)}$$
$$= (14.7\text{ in.}^2)(0.589)$$
$$= 8.66\text{ in.}^2$$

$$R_n = F_u A_e \qquad \text{(\textit{Spec}. Eq. J4-2)}$$
$$= (65\text{ ksi})(8.66\text{ in.}^2)$$
$$= 563\text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(563\text{ kips})$ $= 422\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{563\text{ kips}}{2.00}$ $= 282\text{ kips} > 40\text{ kips}$ **o.k.**

Conclusion

The connection is found to be adequate as given for the applied loads.

EXAMPLE II.A-3 ALL-WELDED DOUBLE-ANGLE CONNECTION

Given:

Repeat Example II.A-1A using AISC *Manual* Table 10-3 and applicable provisions from the AISC *Specification* to verify the strength of an all-welded double-angle connection between an ASTM A992 W36×231 beam and an ASTM A992 W14×90 column flange, as shown in Figure II.A-3-1. Use 70-ksi electrodes and ASTM A36 angles.

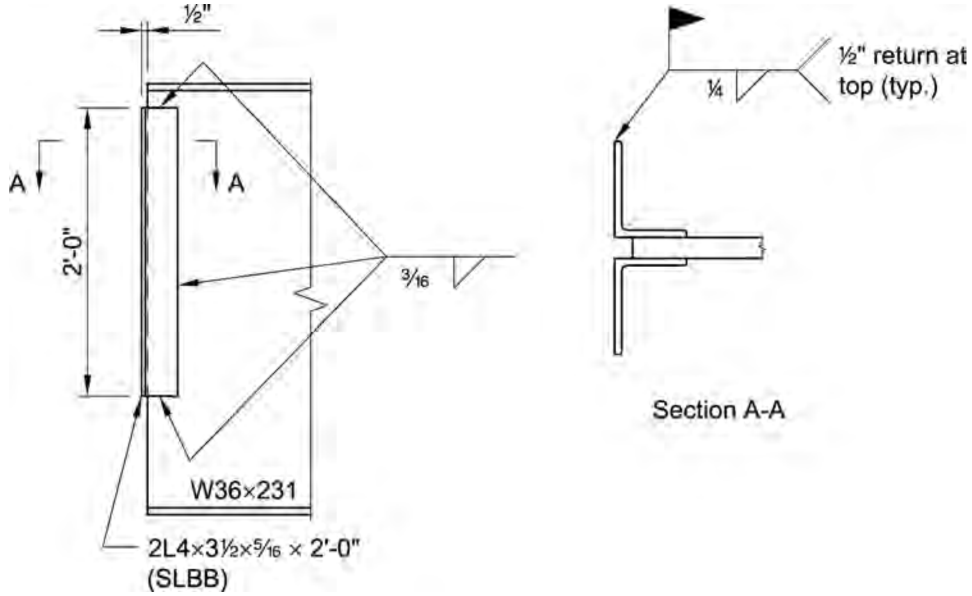


Fig. II.A-3-1. Connection geometry for Example II.A-3.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam and column
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Angles
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W36×231
t_w = 0.760 in.

Column
W14×90
t_f = 0.710 in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(37.5\text{ kips}) + 1.6(113\text{ kips})$ $= 226\text{ kips}$	$R_a = 37.5\text{ kips} + 113\text{ kips}$ $= 151\text{ kips}$

Design of Weld between Beam Web and Angles

Use AISC *Manual* Table 10-3 (Welds A). Try $\frac{3}{16}$-in. weld size, l = 24 in.

LRFD	ASD
$\phi R_n = 257\text{ kips} > 226\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 171\text{ kips} > 151\text{ kips}$ **o.k.**

From AISC *Manual* Table 10-3, the minimum beam web thickness is:

$$t_{w\,min} = 0.286\text{ in.} < 0.760\text{ in.} \quad \textbf{o.k.}$$

Design of Weld between Column Flange and Angles

Use AISC *Manual* Table 10-3 (Welds B). Try ¼-in. weld size, l = 24 in.

LRFD	ASD
$\phi R_n = 229\text{ kips} > 226\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 153\text{ kips} > 151\text{ kips}$ **o.k.**

From AISC *Manual* Table 10-3, the minimum column flange thickness is:

$$t_{f\,min} = 0.190\text{ in.} < 0.710\text{ in.} \quad \textbf{o.k.}$$

Angle Thickness

Minimum angle thickness for weld from AISC *Specification* Section J2.2b:

$$\begin{aligned} t_{min} &= w + \tfrac{1}{16}\text{ in.} \\ &= \tfrac{1}{4}\text{ in.} + \tfrac{1}{16}\text{ in.} \\ &= \tfrac{5}{16}\text{ in.} \end{aligned}$$

Try 2L4×3½×⅝16 (SLBB).

Shear Strength of Angles

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the angles is determined as follows:

$$\begin{aligned} A_{gv} &= (2\text{ angles})lt \\ &= (2\text{ angles})(24\text{ in.})(\tfrac{5}{16}\text{ in.}) \\ &= 15.0\text{ in.}^2 \end{aligned}$$

$$R_n = 0.60F_yA_{gv}$$ (*Spec*. Eq. J4-3)

$$= 0.60(36\text{ ksi})(15.0\text{ in.}^2)$$

$$= 324\text{ kips}$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(324\text{ kips})$ $= 324\text{ kips} > 226\text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{324\text{ kips}}{1.50}$ $= 216\text{ kips} > 151\text{ kips}$ **o.k.**

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the angles is determined as follows:

$$A_{nv} = (2\text{ angles})lt$$

$$= (2\text{ angles})(24\text{ in.})(5/16\text{ in.})$$

$$= 15.0\text{ in.}^2$$

$$R_n = 0.60F_uA_{nv}$$ (*Spec*. Eq. J4-4)

$$= 0.60(58\text{ ksi})(15.0\text{ in.}^2)$$

$$= 522\text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(522\text{ kips})$ $= 392\text{ kips} > 226\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{522\text{ kips}}{2.00}$ $= 261\text{ kips} > 151\text{ kips}$ **o.k.**

Conclusion

The connection is found to be adequate as given for the applied loads.

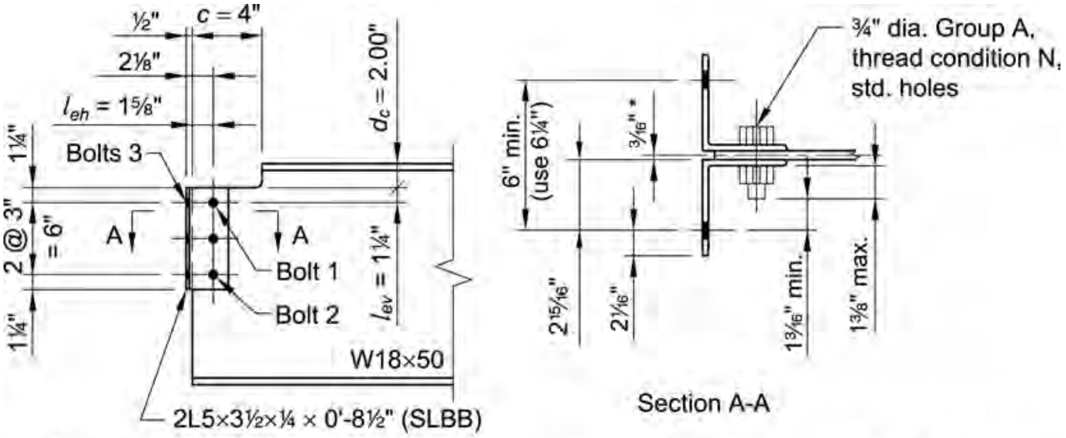
EXAMPLE II.A-4 ALL-BOLTED DOUBLE-ANGLE CONNECTION IN A COPED BEAM

Given:

Use AISC *Manual* Table 10-1 to verify the available strength of an all-bolted double-angle connection between an ASTM A992 W18×50 beam and an ASTM A992 W21×62 girder web, as shown in Figure II.A-4-1, to support the following beam end reactions:

$R_D = 10$ kips
$R_L = 30$ kips

The beam top flange is coped 2 in. deep by 4 in. long, $l_{ev} = 1\frac{1}{4}$ in., $l_{eh} = 1\frac{5}{8}$ in. Use ASTM A36 angles.



* This dimension is one-half decimal web thickness rounded to the next higher ¹⁄₁₆ in., as in Example II.A-1.

Note: The given dimensions for entering and tightening clearances are from AISC *Manual* Table 7-15.

Fig. II.A-4-1. Connection geometry for Example II.A-4.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam and girder
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Angles
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Tables 1-1 the geometric properties are as follows:

Beam
W18×50
$d = 18.0$ in.
$t_w = 0.355$ in.

Girder
W21×62
$t_w = 0.400$ in.

From AISC *Specification* Table J3.3, the hole diameter of a ¾-in.-diameter bolt in a standard hole is:

$d_h = {}^{13}\!/_{16}$ in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(10\text{ kips}) + 1.6(30\text{ kips})$ $= 60.0\text{ kips}$	$R_a = 10\text{ kips} + 30\text{ kips}$ $= 40.0\text{ kips}$

Connection Design

Tabulated values in AISC *Manual* Table 10-1 consider the limit states of bolt shear, bolt bearing and tearout on the angles, shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles.

Try 3 rows of bolts and 2L5×3½×¼ (SLBB).

LRFD	ASD
$\phi R_n = 76.7\text{ kips} > 60.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 51.1\text{ kips} > 40.0\text{ kips}$ **o.k.**

Coped Beam Strength

From AISC *Manual* Part 9, the available coped beam web strength is the lesser of the limit states of flexural local web buckling, shear yielding, shear rupture, block shear rupture, and the sum of the effective strengths of the individual fasteners. From the Commentary to AISC *Specification* Section J3.6, the effective strength of an individual fastener is the lesser of the fastener shear strength, the bearing strength at the bolt holes and the tearout strength at the bolt holes.

Flexural local web buckling of beam web

As shown in AISC *Manual* Figure 9-2, the cope dimensions are:

$c = 4$ in.

$d_c = 2.00$ in.

$$\begin{aligned} e &= c + setback \\ &= 4\text{ in.} + ½\text{ in.} \\ &= 4.50\text{ in.} \end{aligned}$$

$$\begin{aligned} h_o &= d - d_c \\ &= 18.0\text{ in.} - 2.00\text{ in.} \\ &= 16.0\text{ in.} \end{aligned}$$

$$\begin{aligned} \frac{c}{d} &= \frac{4\text{ in.}}{18.0\text{ in.}} \\ &= 0.222 \end{aligned}$$

$$\begin{aligned}\frac{c}{h_o} &= \frac{4 \text{ in.}}{16.0 \text{ in.}}\\ &= 0.250\end{aligned}$$

Because $\frac{c}{d} \leq 1.0$:

$$\begin{aligned}f &= 2\left(\frac{c}{d}\right)\\ &= 2(0.222)\\ &= 0.444\end{aligned} \qquad \text{(\textit{Manual} Eq. 9-14a)}$$

Because $\frac{c}{h_o} \leq 1.0$:

$$\begin{aligned}k &= 2.2\left(\frac{h_o}{c}\right)^{1.65}\\ &= 2.2\left(\frac{16.0 \text{ in.}}{4 \text{ in.}}\right)^{1.65}\\ &= 21.7\end{aligned} \qquad \text{(\textit{Manual} Eq. 9-13a)}$$

$$\begin{aligned}\lambda &= \frac{h_o}{t_w}\\ &= \frac{16.0 \text{ in.}}{0.355 \text{ in.}}\\ &= 45.1\end{aligned} \qquad \text{(\textit{Manual} Eq. 9-11)}$$

$$\begin{aligned}k_1 &= fk \geq 1.61\\ &= (0.444)(21.7) \geq 1.61\\ &= 9.63\end{aligned} \qquad \text{(\textit{Manual} Eq. 9-10)}$$

$$\begin{aligned}\lambda_p &= 0.475\sqrt{\frac{k_1 E}{F_y}}\\ &= 0.475\sqrt{\frac{(9.63)(29{,}000 \text{ ksi})}{50 \text{ ksi}}}\\ &= 35.5\end{aligned} \qquad \text{(\textit{Manual} Eq. 9-12)}$$

$$\begin{aligned}2\lambda_p &= 2(35.5)\\ &= 71.0\end{aligned}$$

Because $\lambda_p < \lambda \leq 2\lambda_p$, calculate the nominal flexural strength using AISC *Manual* Equation 9-7.

The plastic section modulus of the coped section, Z_{net}, is determined from Table IV-11 (included in Part IV of this document).

$$Z_{net} = 42.5 \text{ in.}^3$$

$$\begin{aligned} M_p &= F_y Z_{net} \\ &= (50\text{ ksi})\left(42.5\text{ in.}^3\right) \\ &= 2{,}130\text{ kip-in.} \end{aligned}$$

From AISC *Manual* Table 9-2:

$$S_{net} = 23.4\text{ in.}^3$$

$$\begin{aligned} M_y &= F_y S_{net} \\ &= (50\text{ ksi})\left(23.4\text{ in.}^3\right) \\ &= 1{,}170\text{ kip-in.} \end{aligned}$$

$$M_n = M_p - \left(M_p - M_y\right)\left(\frac{\lambda}{\lambda_p} - 1\right) \qquad (\textit{Manual}\text{ Eq. 9-7})$$

$$\begin{aligned} &= (2{,}130\text{ kip-in.}) - (2{,}130\text{ kip-in.} - 1{,}170\text{ kip-in.})\left(\frac{45.1}{35.5} - 1\right) \\ &= 1{,}870\text{ kip-in.} \end{aligned}$$

$$\begin{aligned} R_n &= \frac{M_n}{e} \\ &= \frac{1{,}870\text{ kip-in.}}{4.50\text{ in.}} \\ &= 416\text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(416\text{ kips})$ $= 374\text{ kips} > 60.0\text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{416\text{ kips}}{1.67}$ $= 249\text{ kips} > 40.0\text{ kips}$ **o.k.**

Shear Strength of Beam Web

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the beam web is determined as follows:

$$\begin{aligned} A_{gv} &= h_o t_w \\ &= (16.0\text{ in.})(0.355\text{ in.}) \\ &= 5.68\text{ in.}^2 \end{aligned}$$

$$R_n = 0.60 F_y A_{gv} \qquad (\textit{Spec.}\text{ Eq. J4-3})$$

$$\begin{aligned} &= 0.60(50\text{ ksi})\left(5.68\text{ in.}^2\right) \\ &= 170\text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(170 \text{ kips})$ $= 170 \text{ kips} > 60.0 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{170 \text{ kips}}{1.50}$ $= 113 \text{ kips} > 40.0 \text{ kips}$ **o.k.**

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the beam web is determined as follows:

$$
\begin{aligned}
A_{nv} &= \left[h_o - 3\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t_w \\
&= \left[16.0\text{ in.} - 3\left(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right](0.355\text{ in.}) \\
&= 4.75\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
R_n &= 0.60F_uA_{nv} \\
&= 0.60(65\text{ ksi})\left(4.75\text{ in.}^2\right) \\
&= 185\text{ kips}
\end{aligned}
$$
(*Spec*. Eq. J4-4)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(185 \text{ kips})$ $= 139 \text{ kips} > 60.0 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{185 \text{ kips}}{2.00}$ $= 92.5 \text{ kips} > 40.0 \text{ kips}$ **o.k.**

Block Shear Rupture of Beam Web

From AISC *Specification* Section J4.3, the block shear rupture strength of the beam web, assuming a total beam setback of ¾ in. (includes ¼ in. tolerance to account for possible mill underrun), is determined as follows.

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt}$$
(*Spec*. Eq. J4-5)

where

$$
\begin{aligned}
A_{gv} &= \left(l_{ev} + 2s\right)t_w \\
&= \left[1\tfrac{1}{4}\text{ in.} + 2(3.00\text{ in.})\right](0.355\text{ in.}) \\
&= 2.57\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
A_{nv} &= A_{gv} - 2.5\left(d_h + \tfrac{1}{16}\text{ in.}\right)t_w \\
&= 2.57\text{ in.}^2 - 2.5\left(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)(0.355\text{ in.}) \\
&= 1.79\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
A_{nt} &= \left[l_{eh} - \tfrac{1}{4}\text{ in.(underrun)} - 0.5\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t_w \\
&= \left[1\tfrac{5}{8}\text{ in.} - \tfrac{1}{4}\text{ in.(underrun)} - 0.5\left(\tfrac{13}{16} + \tfrac{1}{16}\text{ in.}\right)\right](0.355\text{ in.}) \\
&= 0.333\text{ in.}^2
\end{aligned}
$$

The block shear reduction coefficient, U_{bs}, is 1.0 for a single row beam end connection as illustrated in AISC *Specification* Commentary Figure C-J4.2.

$$R_n = 0.60(65\text{ ksi})(1.79\text{ in.}^2) + 1.0(65\text{ ksi})(0.333\text{ in.}^2) < 0.60(50\text{ ksi})(2.57\text{ in.}^2) + 1.0(65\text{ ksi})(0.333\text{ in.}^2)$$
$$= 91.5\text{ kips} \le 98.7\text{ kips}$$

Therefore:

$$R_n = 91.5\text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(91.5\text{ kips})$ $= 68.6\text{ kips} > 60.0\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{91.5\text{ kips}}{2.00}$ $= 45.8\text{ kips} > 40.0\text{ kips}$ **o.k.**

Strength of the Bolted Connection—Beam Web Side

From the Commentary to AISC *Specification* Section J3.6, the strength of the bolt group is taken as the sum of the invividual strengths of the individual fasteners, which may be taken as the lesser of the fastener shear strength per AISC *Specification* Section J3.6, the bearing strength at the bolt hole per AISC *Specification* Section J3.10, or the tearout strength at the bolt hole per AISC *Specification* Section J3.10.

From AISC *Manual* Table 7-1, the available shear strength per bolt for ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) in double shear (or pair of bolts) is:

LRFD	ASD
$\phi r_n = 35.8\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 23.9\text{ kips/bolt}$

The available bearing and tearout strength of the beam web at Bolt 1, as shown in Figure II.A-4-1, is determine using AISC *Manual* Table 7-5 with l_e = 1¼ in.

LRFD	ASD
$\phi r_n = (49.4\text{ kip/in.})(0.355\text{ in.})$ $= 17.5\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (32.9\text{ kip/in.})(0.355\text{ in.})$ $= 11.7\text{ kips/bolt}$

Therefore, bearing or tearout of the beam web controls over bolt shear for Bolt 1.

The available bearing and tearout strength of the beam web at the other bolts is determine using AISC *Manual* Table 7-4 with s = 3 in.

LRFD	ASD
$\phi r_n = (87.8\text{ kip/in.})(0.355\text{ in.})$ $= 31.2\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (58.5\text{ kip/in.})(0.355\text{ in.})$ $= 20.8\text{ kips/bolt}$

Therefore, bearing or tearout of the beam web controls over bolt shear for the other bolts.

The strength of the bolt group in the beam web is determined by summing the strength of the individual fasteners as follows:

LRFD	ASD
$\phi R_n = (1\text{ bolt})(17.5\text{ kips/bolt})$ $+(2\text{ bolts})(31.2\text{ kips/bolt})$ $= 79.9\text{ kips/bolt} > 60.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = (1\text{ bolt})(11.7\text{ kips/bolt})$ $+(2\text{ bolts})(20.8\text{ kips/bolt})$ $= 53.3\text{ kips/bolt} > 40.0\text{ kips}$ **o.k.**

Strength of the Bolted Connection—Support Side

From AISC *Manual* Table 7-1, the available shear strength per bolt for ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) in single shear is:

LRFD	ASD
$\phi r_n = 17.9\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 11.9\text{ kips/bolt}$

Because the girder is not coped, the available bearing and tearout strength of the girder web at all bolts is determined using AISC *Manual* Table 7-4 with $s = 3$ in.

LRFD	ASD
$\phi r_n = (87.8\text{ kip/in.})(0.400\text{ in.})$ $= 35.1\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (58.5\text{ kip/in.})(0.400\text{ in.})$ $= 23.4\text{ kips/bolt}$

Therefore, bolt shear shear controls over bearing and tearout. Bolt shear strength is one of the limit states checked in previous calculations; thus, the effective strength of the fasteners is adequate.

Conclusion

The connection is found to be adequate as given for the applied loads.

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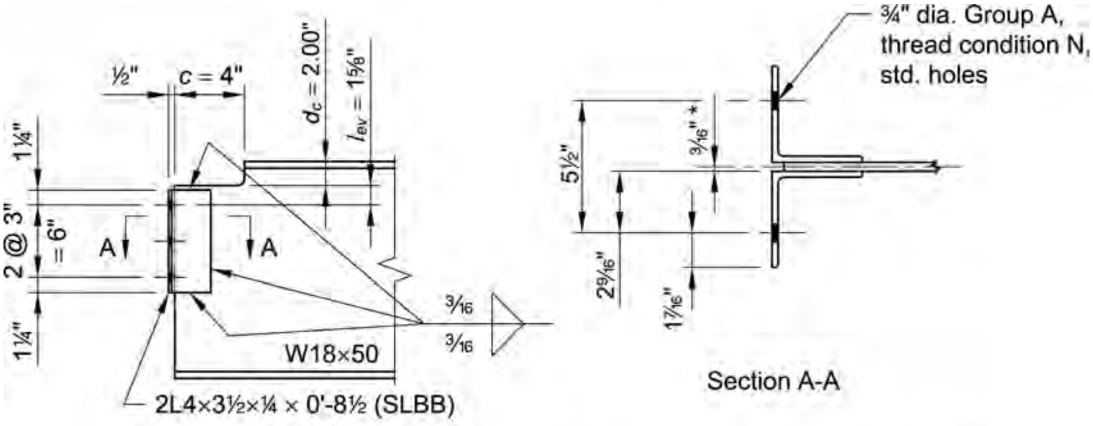
EXAMPLE II.A-5 WELDED/BOLTED DOUBLE-ANGLE CONNECTION IN A COPED BEAM

Given:

Use AISC *Manual* Table 10-2 to verify the available strength of a double angle shear connection welded to an ASTM A992 W18×50 beam and bolted to an ASTM A992 W21×62 girder web, as shown in Figure II.A-5-1. Use 70-ksi electrodes and ASTM A36 angles.



* This dimension is one-half decimal web thickness rounded to the next higher 1⁄16 in., as in Example II.A-1.

Fig. II.A-5-1. Connection geometry for Example II.A-5.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam and girder
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Angles
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Tables 1-1 the geometric properties are as follows:

Beam
W18×50
d = 18.0 in.
t_w = 0.355 in.

Girder
W21×62
t_w = 0.400 in.

From AISC *Specification* Table J3.3, the hole diameter of a ¾-in.-diameter bolt in a standard hole is:

d_h = 13⁄16 in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(10\text{ kips}) + 1.6(30\text{ kips})$ $= 60.0\text{ kips}$	$R_a = 10\text{ kips} + 30\text{ kips}$ $= 40.0\text{ kips}$

Weld Design

Use AISC *Manual* Table 10-2 (Welds A). Try 3⁄16-in. weld size, l = 8½ in.

LRFD	ASD
$\phi R_n = 110\text{ kips} > 60.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 73.5\text{ kips} > 40.0\text{ kips}$ **o.k.**

From AISC *Manual* Table 10-2, the minimum beam web thickness is:

$t_{w\ min} = 0.286\text{ in.} < 0.355\text{ in.}$ **o.k.**

Minimum Angle Thickness for Weld

From AISC *Specification* Section J2.2b, the minimum angle thickness is:

$$\begin{aligned} t_{min} &= w + 1/16\text{ in.} \\ &= 3/16\text{ in.} + 1/16\text{ in.} \\ &= 1/4\text{ in.} \end{aligned}$$

Angle and Bolt Design

Tabulated values in AISC *Manual* Table 10-1 consider the limit states of bolt shear, bolt bearing and tearout on the angles, shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles.

Try 3 rows of bolts and 2L4×3½×¼ (SLBB).

LRFD	ASD
$\phi R_n = 76.7\text{ kips} > 60.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 51.1\text{ kips} > 40.0\text{ kips}$ **o.k.**

Coped Beam Strength

The available flexural local web buckling strength of the coped beam is verified in Example II.A-4.

Block Shear Rupture of Beam Web

From AISC *Specification* Section J4.3, the block shear rupture strength of the beam web, assuming a total beam setback of ¾ in. (includes ¼ in. tolerance to account for possible mill underrun), is determined as follows.

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad \text{(}\textit{Spec.}\text{ Eq. J4-5)}$$

where

$$A_{gv} = (l + 3/8 \text{ in.})t_w$$
$$= (8\tfrac{1}{2} \text{ in.} + 3/8 \text{ in.})(0.355 \text{ in.})$$
$$= 3.15 \text{ in.}^2$$

$$A_{nv} = A_{gv}$$
$$= 3.15 \text{ in.}^2$$

$$A_{nt} = (3\tfrac{1}{2} \text{ in.} - 3/4 \text{ in.})t_w$$
$$= (3\tfrac{1}{2} \text{ in.} - 3/4 \text{ in.})(0.355 \text{ in.})$$
$$= 0.976 \text{ in.}^2$$

$$U_{bs} = 1.0$$

and

$$R_n = 0.60(65 \text{ ksi})(3.15 \text{ in.}^2) + 1.0(65 \text{ ksi})(0.976 \text{ in.}^2) < 0.60(50 \text{ ksi})(3.15 \text{ in.}^2) + 1.0(65 \text{ ksi})(0.976 \text{ in.}^2)$$
$$= 186 \text{ kips} > 158 \text{ kips}$$

Therefore:

$$R_n = 158 \text{ kips}$$

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75(158 \text{ kips})$ $= 119 \text{ kips} > 60.0 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{158 \text{ kips}}{2.00}$ $= 79.0 \text{ kips} > 40.0 \text{ kips}$ **o.k.**

Shear Strength of Beam Web

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the beam web is determined as follows:

$$A_{gv} = (d - d_c)t_w$$
$$= (18.0 \text{ in.} - 2.00 \text{ in.})(0.355 \text{ in.})$$
$$= 5.68 \text{ in.}^2$$

$$R_n = 0.60F_y A_{gv} \qquad (\textit{Spec}. \text{ Eq. J4-3})$$
$$= 0.60(50 \text{ ksi})(5.68 \text{ in.}^2)$$
$$= 170 \text{ kips}$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(170 \text{ kips})$ $= 170 \text{ kips} > 60.0 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{170 \text{ kips}}{1.50}$ $= 113 \text{ kips} > 40.0 \text{ kips}$ **o.k.**

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the beam web is determined as follows. Because the angle is welded to the beam web, there is no reduction for bolt holes, therefore:

$$A_{nv} = A_{gv}$$
$$= 5.68 \text{ in.}^2$$

$$R_n = 0.60F_u A_{nv}$$ (*Spec*. Eq. J4-4)
$$= 0.60(65 \text{ ksi})(5.68 \text{ in.}^2)$$
$$= 222 \text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(222 \text{ kips})$ $= 167 \text{ kips} > 60.0 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{222 \text{ kips}}{2.00}$ $= 111 \text{ kips} > 40.0 \text{ kips}$ **o.k.**

Effective Strength of the Fasteners to the Girder Web

The effective strength of the fasteners to the girder web is verified in Example II.A-4.

Summary

The connection is found to be adequate as given for the applied loads.

EXAMPLE II.A-6 BEAM END COPED AT THE TOP FLANGE ONLY

Given:

For an ASTM A992 W21×62 beam coped 8 in. deep by 9 in. long at the top flange only, assuming a ½ in. setback (e = 9½ in.) and using an ASTM A572 Grade 50 plate for the stiffeners and doubler:

A. Calculate the available strength of the beam end, as shown in Figure II.A-6-1(a), considering the limit states of flexural yielding, flexural local buckling, shear yielding and shear rupture.

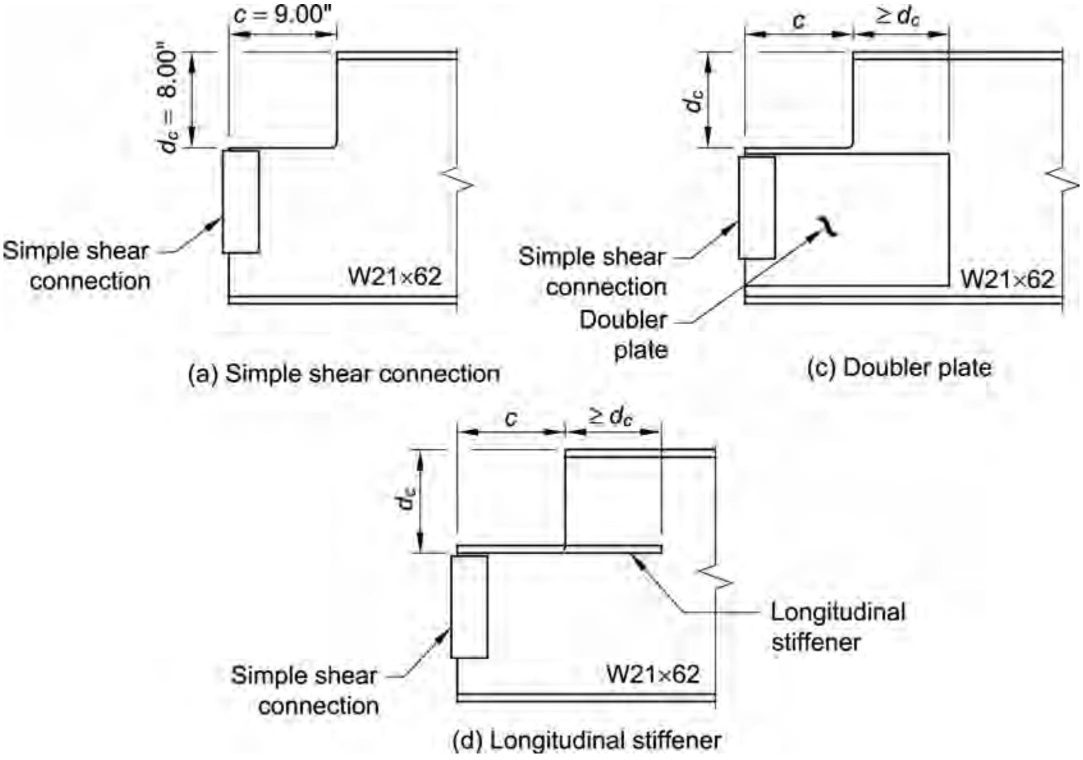
B. Choose an alternate ASTM A992 W21 shape to eliminate the need for stiffening for the following end reactions:

R_D = 23 kips
R_L = 67 kips

C. Determine the size of doubler plate needed to reinforce the W21×62, as shown in Figure II.A-6-1(c), for the given end reaction in Solution B.

D. Determine the size of longitudinal stiffeners needed to stiffen the W21, as shown in Figure II.A-6-1(d), for the given end reaction in Solution B.

Assume the shear connection is welded to the beam web.



(a) Simple shear connection

(c) Doubler plate

(d) Longitudinal stiffener

Fig. II.A-6-1. Connection geometry for Example II.A-6.

Solution A:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Beam
W21×62
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Plate
ASTM A572 Grade 50
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 1-1 the geometric properties are as follows:

Beam
W21×62
d = 21.0 in.
t_w = 0.400 in.
b_f = 8.24 in.
t_f = 0.615 in.

Coped Beam Strength

The beam is assumed to be braced at the end of the uncoped section. Such bracing can be provided by a bracing member or by a slab or other suitable means.

Flexural Local Buckling of Beam Web

The limit state of flexural yielding and local web buckling of the coped beam web are checked using AISC *Manual* Part 9 as follows.

$$\begin{aligned} h_o &= d - d_c \text{ (from AISC } \textit{Manual} \text{ Figure 9-2)} \\ &= 21.0 \text{ in.} - 8.00 \text{ in.} \\ &= 13.0 \text{ in.} \end{aligned}$$

$$\begin{aligned} \frac{c}{d} &= \frac{9.00 \text{ in.}}{21.0 \text{ in.}} \\ &= 0.429 \end{aligned}$$

$$\begin{aligned} \frac{c}{h_o} &= \frac{9.00 \text{ in.}}{13.0 \text{ in.}} \\ &= 0.692 \end{aligned}$$

Because $\frac{c}{d} \le 1.0,$ the buckling adjustment factor, f, is calculated as:

$$\begin{aligned} f &= 2\left(\frac{c}{d}\right) \\ &= 2(0.429) \\ &= 0.858 \end{aligned}$$ (*Manual* Eq. 9-14a)

Because $\frac{c}{h_o} \le 1.0$, the plate buckling coefficient, k, is calculated as:

$$k = 2.2\left(\frac{h_o}{c}\right)^{1.65}$$ (*Manual* Eq. 9-13a)

$$= 2.2\left(\frac{13.0\text{ in.}}{9.00\text{ in.}}\right)^{1.65}$$

$$= 4.04$$

The modified plate buckling coefficient, k_1, is calculated as:

$$k_1 = fk \geq 1.61$$ (*Manual* Eq. 9-10)

$$= (0.858)(4.04) > 1.61$$

$$= 3.47$$

The plastic section modulus, Z_{net}, is determined from Table IV-11 (included in Part IV of this document):

$$Z_{net} = 32.2\text{ in.}^3$$

The plastic moment capacity, M_p, is:

$$M_p = F_y Z_{net}$$

$$= (50\text{ ksi})\left(32.2\text{ in.}^3\right)$$

$$= 1{,}610\text{ kip-in.}$$

The elastic section modulus, S_{net}, is determined from AISC *Manual* Table 9-2:

$$S_{net} = 17.8\text{ in.}^3$$

The flexural yield moment, M_y, is:

$$M_y = F_y S_{net}$$

$$= (50\text{ ksi})\left(17.8\text{ in.}^3\right)$$

$$= 890\text{ kip-in.}$$

$$\lambda = \frac{h_o}{t_w}$$ (*Manual* Eq. 9-11)

$$= \frac{13.0\text{ in.}}{0.400\text{ in.}}$$

$$= 32.5$$

$$\lambda_p = 0.475\sqrt{\frac{k_1 E}{F_y}}$$ (*Manual* Eq. 9-12)

$$= 0.475\sqrt{\frac{(3.47)(29{,}000\text{ ksi})}{50\text{ ksi}}}$$

$$= 21.3$$

$$\begin{aligned}2\lambda_p &= 2(21.3)\\ &= 42.6\end{aligned}$$

Because $\lambda_p < \lambda \le 2\lambda_p$, the nominal flexural strength is:

$$M_n = M_p - (M_p - M_y)\left(\frac{\lambda}{\lambda_p} - 1\right) \qquad \text{(\textit{Manual} Eq. 9-7)}$$

$$\begin{aligned}&= 1{,}610 \text{ kip-in.} - (1{,}610 \text{ kip-in.} - 890 \text{ kip-in.})\left(\frac{32.5}{21.3} - 1\right)\\ &= 1{,}230 \text{ kip-in.}\end{aligned}$$

The nominal strength of the coped section is:

$$\begin{aligned}R_n &= \frac{M_n}{e}\\ &= \frac{1{,}230 \text{ kip-in.}}{9.50 \text{ in.}}\\ &= 129 \text{ kips}\end{aligned}$$

The available strength of the coped section is:

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(129 \text{ kips})$ $= 116 \text{ kips}$	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{129 \text{ kips}}{1.67}$ $= 77.2 \text{ kips}$

Shear Strength of Beam Web

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the beam web is determined as follows:

$$\begin{aligned}A_{gv} &= (d - d_c)t_w\\ &= (21.0 \text{ in.} - 8.00 \text{ in.})(0.400 \text{ in.})\\ &= 5.20 \text{ in.}^2\end{aligned}$$

$$R_n = 0.60F_yA_{gv} \qquad \text{(\textit{Spec}. Eq. J4-3)}$$

$$\begin{aligned}&= 0.60(50 \text{ ksi})(5.20 \text{ in.}^2)\\ &= 156 \text{ kips}\end{aligned}$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(156 \text{ kips})$ $= 156 \text{ kips}$	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{156 \text{ kips}}{1.50}$ $= 104 \text{ kips}$

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the beam web is determined as follows. Because the connection is welded to the beam web there is no reduction for bolt holes, therefore:

$$\begin{aligned} A_{nv} &= A_{gv} \\ &= 5.20 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60 F_u A_{nv} \\ &= 0.60(65 \text{ ksi})(5.20 \text{ in.}^2) \\ &= 203 \text{ kips} \end{aligned}$$

(*Spec*. Eq. J4-4)

LRFD	ASD
$\phi = 0.75$ $\begin{aligned} \phi R_n &= 0.75(203 \text{ kips}) \\ &= 152 \text{ kips} \end{aligned}$	$\Omega = 2.00$ $\begin{aligned} \frac{R_n}{\Omega} &= \frac{203 \text{ kips}}{2.00} \\ &= 102 \text{ kips} \end{aligned}$

Thus, the available strength of the beam is controlled by the coped section.

LRFD	ASD
$\phi R_n = 116 \text{ kips}$	$\frac{R_n}{\Omega} = 77.2 \text{ kips}$

Solution B:

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$\begin{aligned} R_u &= 1.2(23 \text{ kips}) + 1.6(67 \text{ kips}) \\ &= 135 \text{ kips} \end{aligned}$	$\begin{aligned} R_a &= 23 \text{ kips} + 67 \text{ kips} \\ &= 90.0 \text{ kips} \end{aligned}$

Try a W21×73.

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam
W21×73
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

From AISC *Manual* Table 1-1 the geometric properties are as follows:

Beam
W21×73
$d = 21.2$ in.
$t_w = 0.455$ in.
$b_f = 8.30$ in.
$t_f = 0.740$ in.

Flexural Local Buckling of Beam Web

The limit state of flexural yielding and local web buckling of the coped beam web are checked using AISC *Manual* Part 9 as follows.

$$\begin{aligned} h_o &= d - d_c \text{ (from AISC } \textit{Manual} \text{ Figure 9-2)} \\ &= 21.2 \text{ in.} - 8.00 \text{ in.} \\ &= 13.2 \text{ in.} \end{aligned}$$

$$\begin{aligned} \frac{c}{d} &= \frac{9.00 \text{ in.}}{21.2 \text{ in.}} \\ &= 0.425 \end{aligned}$$

$$\begin{aligned} \frac{c}{h_o} &= \frac{9.00 \text{ in.}}{13.2 \text{ in.}} \\ &= 0.682 \end{aligned}$$

Because $\frac{c}{d} \le 1.0$, the buckling adjustment factor, f, is calculated as:

$$\begin{aligned} f &= 2\left(\frac{c}{d}\right) \\ &= 2(0.425) \\ &= 0.850 \end{aligned}$$

(*Manual* Eq. 9-14a)

Because $\frac{c}{h_o} \le 1.0$, the plate buckling coefficient, k, is calculated as:

$$\begin{aligned} k &= 2.2\left(\frac{h_o}{c}\right)^{1.65} \\ &= 2.2\left(\frac{13.2 \text{ in.}}{9.00 \text{ in.}}\right)^{1.65} \\ &= 4.14 \end{aligned}$$

(*Manual* Eq. 9-13a)

The modified plate buckling coefficient, k_1, is calculated as:

$$\begin{aligned} k_1 &= fk \ge 1.61 \\ &= (0.850)(4.14) > 1.61 \\ &= 3.52 \end{aligned}$$

(*Manual* Eq. 9-10)

The plastic section modulus, Z_{net}, is determined from Table IV-11 (included in Part IV of this document):

$$Z_{net} = 37.6 \text{ in.}^3$$

The plastic moment capacity, M_p, is:

$$\begin{aligned} M_p &= F_y Z_{net} \\ &= (50 \text{ ksi})(37.6 \text{ in.}^3) \\ &= 1{,}880 \text{ kip-in.} \end{aligned}$$

The elastic section modulus, S_{net}, is determined from AISC *Manual* Table 9-2:

$$S_{net} = 21.0 \text{ in.}^3$$

The flexural yield moment, M_y, is:

$$\begin{aligned} M_y &= F_y S_{net} \\ &= (50 \text{ ksi})\left(21.0 \text{ in.}^3\right) \\ &= 1{,}050 \text{ kip-in.} \end{aligned}$$

$$\begin{aligned} \lambda &= \frac{h_o}{t_w} \\ &= \frac{13.2 \text{ in.}}{0.455 \text{ in.}} \\ &= 29.0 \end{aligned} \qquad \text{(}\textit{Manual}\text{ Eq. 9-11)}$$

$$\begin{aligned} \lambda_p &= 0.475\sqrt{\frac{k_1 E}{F_y}} \\ &= 0.475\sqrt{\frac{(3.52)(29{,}000 \text{ ksi})}{50 \text{ ksi}}} \\ &= 21.5 \end{aligned} \qquad \text{(}\textit{Manual}\text{ Eq. 9-11)}$$

$$\begin{aligned} 2\lambda_p &= 2(21.5) \\ &= 43.0 \end{aligned}$$

Since $\lambda_p < \lambda \le 2\lambda_p$, the nominal flexural strength is:

$$\begin{aligned} M_n &= M_p - \left(M_p - M_y\right)\left(\frac{\lambda}{\lambda_p} - 1\right) \\ &= 1{,}880 \text{ kip-in.} - (1{,}880 \text{ kip-in.} - 1{,}050 \text{ kip-in.})\left(\frac{29.0}{21.5} - 1\right) \\ &= 1{,}590 \text{ kip-in.} \end{aligned} \qquad \text{(}\textit{Manual}\text{ Eq. 9-7)}$$

The nominal strength of the coped section is:

$$\begin{aligned} R_n &= \frac{M_n}{e} \\ &= \frac{1{,}590 \text{ kip-in.}}{9.50 \text{ in.}} \\ &= 167 \text{ kips} \end{aligned}$$

The available strength of the coped section is:

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(167\text{ kips})$ $= 150\text{ kips}$	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{167\text{ kips}}{1.67}$ $= 100\text{ kips}$

Shear Strength of Beam Web

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the beam web is determined as follows:

$$\begin{aligned} A_{gv} &= (d - d_c)t_w \\ &= (21.2\text{ in.} - 8.00\text{ in.})(0.455\text{ in.}) \\ &= 6.01\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60F_yA_{gv} \\ &= 0.60(50\text{ ksi})(6.01\text{ in.}^2) \\ &= 180\text{ kips} \end{aligned} \qquad (Spec.\text{ Eq. J4-3})$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(180\text{ kips})$ $= 180\text{ kips}$	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{180\text{ kips}}{1.50}$ $= 120\text{ kips}$

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the beam web is determined as follows. Because the connection is welded to the beam web, there is no reduction for bolt holes, therefore:

$$\begin{aligned} A_{nv} &= A_{gv} \\ &= 6.01\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60F_uA_{nv} \\ &= 0.60(65\text{ ksi})(6.01\text{ in.}^2) \\ &= 234\text{ kips} \end{aligned} \qquad (Spec.\text{ Eq. J4-4})$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(234\text{ kips})$ $= 176\text{ kips}$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{234\text{ kips}}{2.00}$ $= 117\text{ kips}$

Thus, the available strength is controlled by the coped section, therefore the available strength of the beam is:

LRFD	ASD
$\phi R_n = 150 \text{ kips} > 135 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 100 \text{ kips} > 90.0 \text{ kips}$ **o.k.**

Solution C:

Doubler Plate Design

The doubler plate is designed using AISC *Manual* Part 9. An ASTM A572 Grade 50 plate is recommended in order to match the beam yield strength. A ¼-in. minimum plate thickness will be used in order to allow the use of a 3⁄16-in. fillet weld. The depth of the plate will be set so that a compact *b/t* ratio from AISC *Specification* Table B4.1b will be satisfied. This is a conservative criterion that will allow local buckling of the doubler to be neglected.

$$\frac{d_p}{t_p} \le 1.12\sqrt{\frac{E}{F_y}}$$

Solving for d_p:

$$\begin{aligned} d_p &\le 1.12 t_p \sqrt{\frac{E}{F_y}} \\ &\le 1.12(0.250 \text{ in.})\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} \\ &\le 6.74 \text{ in.} \end{aligned}$$

A 6.50 in. doubler plate will be used.

Using principles of mechanics, the elastic section modulus, S_{net}, and plastic section modulus, Z_{net}, are calculated neglecting the fillets and assuming the doubler plate is placed ½-in. down from the top of the cope.

$$S_{net} = 25.5 \text{ in.}^3$$
$$Z_{net} = 44.8 \text{ in.}^3$$

The plastic bending moment, M_p, of the reinforced section is:

$$\begin{aligned} M_p &= F_y Z_{net} \\ &= (50 \text{ ksi})(44.8 \text{ in.}^3) \\ &= 2{,}240 \text{ kip-in.} \end{aligned}$$

The flexural yield moment, M_y, of the reinforced section is:

$$\begin{aligned} M_y &= F_y S_{net} \\ &= (50 \text{ ksi})(25.5 \text{ in.}^3) \\ &= 1{,}280 \text{ kip-in.} \end{aligned}$$

Because $\lambda_p < \lambda \le 2\lambda_p$ for the unreinforced section, the nominal flexural strength is:

$$M_n = M_p - \left(M_p - M_y\right)\left(\frac{\lambda}{\lambda_p} - 1\right)$$ (*Manual* Eq. 9-7)

$$= 2{,}240 \text{ kip-in.} - \left(2{,}240 \text{ kip-in.} - 1{,}280 \text{ kip-in.}\right)\left(\frac{32.5}{21.3} - 1\right)$$
$$= 1{,}740 \text{ kip-in.}$$

The available strength of the coped section is determined as follows:

$$R_n = \frac{M_n}{e}$$
$$= \frac{1{,}740 \text{ kip-in.}}{9.50 \text{ in.}}$$
$$= 183 \text{ kips}$$

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(183 \text{ kips})$ $= 165 \text{ kips}$	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{183 \text{ kips}}{1.67}$ $= 110 \text{ kips}$

Shear Strength of Beam Web

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the beam web reinforced with the doubler plate is determined as follows:

$$A_{gv-web} = \left(d - d_c\right)t_w$$
$$= \left(21.0 \text{ in.} - 8.00 \text{ in.}\right)\left(0.400 \text{ in.}\right)$$
$$= 5.20 \text{ in.}^2$$

$$A_{gv-plate} = d_p t_p$$
$$= \left(6.50 \text{ in.}\right)\left(\tfrac{1}{4} \text{ in.}\right)$$
$$= 1.63 \text{ in.}^2$$

$$R_n = 0.60F_y A_{gv-web} + 0.60F_y A_{gv-plate}$$ (from *Spec*. Eq. J4-3)

$$= 0.60\left(50 \text{ ksi}\right)\left(5.20 \text{ in.}^2\right) + 0.60\left(50 \text{ ksi}\right)\left(1.63 \text{ in.}^2\right)$$
$$= 205 \text{ kips}$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(205 \text{ kips})$ $= 205 \text{ kips}$	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{205 \text{ kips}}{1.50}$ $= 137 \text{ kips}$

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the beam web reinforced with the doubler plate is determined as follows. Because the connection is welded, there is no reduction for bolt holes, therefore:

$$A_{nv-web} = A_{gv-web}$$
$$= 5.20 \text{ in.}^2$$

$$A_{nv-plate} = A_{gv-plate}$$
$$= 1.63 \text{ in.}^2$$

$$R_n = 0.60F_u A_{nv-web} + 0.60F_u A_{nv-plate}$$ (from *Spec*. Eq. J4-4)
$$= 0.60(65 \text{ ksi})(5.20 \text{ in.}^2) + 0.60(65 \text{ ksi})(1.63 \text{ in.}^2)$$
$$= 266 \text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(266 \text{ kips})$ $= 200 \text{ kips}$	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{266 \text{ kips}}{2.00}$ $= 133 \text{ kips}$

Thus, the available strength of the beam is controlled by the coped section.

LRFD	ASD
$\phi R_n = 165 \text{ kips} > 135 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 110 \text{ kips} > 90.0 \text{ kips}$ **o.k.**

Weld Design

Determine the length of weld required to transfer the force into and out of the doubler plate. From Solution A, the available strength of the beam web is:

LRFD	ASD
$\phi R_n = 116 \text{ kips}$	$\dfrac{R_n}{\Omega} = 77.2 \text{ kips}$

The available strength of the beam web reinforced with the doubler plate is:

LRFD	ASD
$\phi R_n = 165 \text{ kips}$	$\dfrac{R_n}{\Omega} = 110 \text{ kips}$

The force in the doubler plate is determined as follows:

LRFD	ASD
$\phi = 0.90$ $F_d = 0.90(50 \text{ ksi})(¼ \text{ in.})(6.50 \text{ in.})\left(\dfrac{116 \text{ kips}}{165 \text{ kips}}\right)$ $= 51.4 \text{ kips}$	$\Omega = 1.67$ $F_d = \dfrac{(50 \text{ ksi})(¼ \text{ in.})(6.50 \text{ in.})\left(\dfrac{77.2 \text{ kips}}{110 \text{ kips}}\right)}{1.67}$ $= 34.1 \text{ kips}$

From AISC *Specification* Section J2.4, the doubler plate weld is designed as follows:

$$R_n = 0.85R_{nwl} + 1.5R_{nwt}$$ (*Spec*. Eq. J2-6b)

LRFD	ASD
From AISC *Manual* Equation 8-2a: $R_{nw} = 1.392Dl$ From AISC *Specification* Equation J2-6b: $51.4\text{ kips} = \begin{bmatrix}(2\text{ welds})(0.85)(1.392\text{ kips/in.})\\ \times(3\text{ sixteenths})l_w\end{bmatrix} + \begin{bmatrix}(1.5)(1.392\text{ kips/in.})(3\text{ sixteenths})\\ \times(6.50\text{ in.})\end{bmatrix}$ Solving for l_w: l_w = 1.50 in.	From AISC *Manual* Equation 8-2b: $R_{nw} = 0.928Dl$ From AISC *Specification* Equation J2-6b: $34.1\text{ kips} = \begin{bmatrix}(2\text{ welds})(0.85)(0.928\text{ kips/in.})\\ \times(3\text{ sixteenths})l_w\end{bmatrix} + \begin{bmatrix}1.5(0.928\text{ kips/in.})(3\text{ sixteenths})\\ \times(6.50\text{ in.})\end{bmatrix}$ Solving for l_w: l_w = 1.47 in..

Use 1.50 in. of 3⁄16-in. fillet weld, minimum.

The doubler plate must extend at least d_c beyond the cope. Use a PL¼ in. × 6½ in. × 1ft 5 in. with 3⁄16-in. welds all around.

Solution D:

Longitudinal Stiffener Design

Try PL¼ in.×4 in. slotted to fit over the beam web.

Determine Z_x for the stiffened section:

$$\begin{aligned}A_w &= (d - d_c - t_f)t_w\\ &= (21.0\text{ in.} - 8.00\text{ in.} - 0.615\text{ in.})(0.400\text{ in.})\\ &= 4.95\text{ in.}^2\end{aligned}$$

$$\begin{aligned}A_f &= b_f t_f\\ &= (8.24\text{ in.})(0.615\text{ in.})\\ &= 5.07\text{ in.}^2\end{aligned}$$

$$\begin{aligned}A_{rp} &= b_p t_p\\ &= (4.00\text{ in.})(\tfrac{1}{4}\text{ in.})\\ &= 1.00\text{ in.}^2\end{aligned}$$

$$\begin{aligned}A_t &= A_w + A_f + A_{rp}\\ &= 4.95\text{ in.}^2 + 5.07\text{ in.}^2 + 1.00\text{ in.}^2\\ &= 11.0\text{ in.}^2\end{aligned}$$

The location of the plastic neutral axis (neglecting fillets) from the inside of the flange is:

$$(0.615\text{ in.})(8.24\text{ in.})+y_p(0.400\text{ in.})=(\tfrac{1}{4}\text{ in.})(4.00\text{ in.})+(12.4\text{ in.}-y_p)(0.400\text{ in.})$$

$$y_p=1.12\text{ in.}$$

From elementary mechanics, the section properties are as follows:

$Z_x = 44.3\text{ in.}^3$
$I_x = 253\text{ in.}^4$
$S_{xc} = 28.6\text{ in.}^3$
$S_{xt} = 57.7\text{ in.}^3$

$$\begin{aligned}h_c&=2(13.0\text{ in.}-4.39\text{ in.})\\&=17.2\text{ in.}\end{aligned}$$

$$\begin{aligned}h_p&=2(13.0\text{ in.}-1.12\text{ in.}-0.615\text{ in.})\\&=22.5\text{ in.}\end{aligned}$$

Compact section properties for the longitudinal stiffener and the web are determined from AISC *Specification* Table B4.1b, Cases 11 and 16.

$$\begin{aligned}\lambda_p&=0.38\sqrt{\frac{E}{F_y}}\\&=0.38\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}\\&=9.15\end{aligned}$$

(*Spec.* Table B4.1b, Case 11)

$$\begin{aligned}\lambda&=\frac{b}{t}\\&=\frac{(4.00\text{ in.}/2)}{\tfrac{1}{4}\text{ in.}}\\&=8.00\end{aligned}$$

Because $\lambda < \lambda_p$, the stiffener is compact in flexure.

$$\begin{aligned}\lambda_r&=5.70\sqrt{\frac{E}{F_y}}\\&=5.70\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}\\&=137\end{aligned}$$

(*Spec.* Table B4.1b, Case 16)

$$\begin{aligned}\lambda&=\frac{h_c}{t_w}\\&=\frac{17.2\text{ in.}}{0.400\text{ in.}}\\&=43.0\end{aligned}$$

Because $\lambda < \lambda_r$, the web is not slender, therefore AISC *Specification* Section F4 applies.

Determine if lateral-torsional buckling is a design consideration.

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} \qquad \text{(}\textit{Spec}\text{. Eq. F4-12)}$$

$$= \frac{(17.2\text{ in.})(0.400\text{ in.})}{(4.00\text{ in.})(¼\text{ in.})}$$

$$= 6.88$$

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{6}a_w\right)}} \qquad \text{(}\textit{Spec}\text{. Eq. F4-11)}$$

$$= \frac{4.00\text{ in.}}{\sqrt{12\left[1+\frac{1}{6}(6.88)\right]}}$$

$$= 0.788\text{ in.}$$

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} \qquad \text{(}\textit{Spec}\text{. Eq. F4-7)}$$

$$= 1.1(0.788\text{ in.})\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}$$

$$= 20.9\text{ in.}$$

The stiffener will not reach a length of 20.9 in. Lateral-torsional buckling is not a design consideration.

Determine if the web of the singly-symmetric shape is compact. AISC *Specification* Table B4.1b, Case 16, applies.

$$\lambda_p = \frac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y} - 0.09\right)^2} \le 5.70\sqrt{\frac{E}{F_y}}$$

$$= \frac{\frac{17.2\text{ in.}}{22.5\text{ in.}}\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}}{\left[0.54\left(\frac{2{,}220\text{ kip-in.}}{1{,}430\text{ kip-in.}}\right) - 0.09\right]^2} \le 5.70\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}$$

$$= 32.9 < 137$$

$$= 32.9$$

$$\lambda = \frac{h_c}{t_w}$$

$$= \frac{17.2\text{ in.}}{0.400\text{ in.}}$$

$$= 43.0$$

Because $\lambda < \lambda_p$, the web is non-compact, therefore AISC *Specification* Section F4 applies.

Since $S_{xt} > S_{xc}$, tension flange yielding does not govern. Determine flexural strength based on compression flange yielding.

$$\begin{aligned} M_{yc} &= S_{xc}F_y \\ &= \left(28.6 \text{ in.}^3\right)(50 \text{ ksi}) \\ &= 1{,}430 \text{ kip-in.} \end{aligned}$$

$$\begin{aligned} I_{yc} &= \frac{(\text{¼ in.})(4.00 \text{ in.})^3}{12} \\ &= 1.33 \text{ in.}^4 \end{aligned}$$

$$\begin{aligned} I_y &= 1.33 \text{ in.}^4 + \frac{(0.615 \text{ in.})(8.24 \text{ in.})^3}{12} + \frac{(12.4 \text{ in.})(0.400 \text{ in.})^3}{12} \\ &= 30.1 \text{ in.}^4 \end{aligned}$$

$$\begin{aligned} \frac{I_{yc}}{I_y} &= \frac{1.33 \text{ in.}^4}{30.1 \text{ in.}^4} \\ &= 0.0442 \end{aligned}$$

Since $\dfrac{I_{yc}}{I_y} < 0.23$, $R_{pc} = 1.0$. Thus:

$$\begin{aligned} M_n &= R_{pc}M_{yc} \\ &= 1.0(1{,}430 \text{ kip-in.}) \\ &= 1{,}430 \text{ kip-in.} \end{aligned}$$

The nominal strength of the reinforced section is:

$$\begin{aligned} R_n &= \frac{M_n}{e} \\ &= \frac{1{,}430 \text{ kip-in.}}{9.50 \text{ in.}} \\ &= 151 \text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(151 \text{ kips})$ $= 136 \text{ kips} > 135 \text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{151 \text{ kips}}{1.67}$ $= 90.4 \text{ kips} > 90.0 \text{ kips}$ **o.k.**

Plate Dimensions

Since the longitudinal stiffening must extend at least d_c beyond the cope, use PL¼ in.×4 in.×1 ft 5 in. with ¼-in. welds.

Weld Strength

By calculations not shown, the moment of inertia of the reinforced section and distance from the centroid to the bottom of the reinforcement plate are:

$$I_{net} = 253 \text{ in.}^4$$

$$\bar{y} = 8.61 \text{ in.}$$

The first moment of the reinforcement plate is:

$$\begin{aligned} Q &= A_p y \\ &= (\text{¼ in.})(4.00 \text{ in.})\left[8.61 \text{ in.} + 0.5(\text{¼ in.})\right] \\ &= 8.74 \text{ in.}^3 \end{aligned}$$

where A_p is the area of the reinforcement plate and y is the distance from the centroid of the reinforced section to the centroid of the reinforcement plate.

From mechanics of materials and shear flow, the force per length that the weld must resist in the area of the cope is:

LRFD	ASD
$r_u = \dfrac{V_u Q}{I_{net}(2 \text{ welds})}$ $= \dfrac{(135 \text{ kips})(8.74 \text{ in.}^3)}{(253 \text{ in.}^4)(2 \text{ welds})}$ $= 2.33 \text{ kip/in.}$	$r_a = \dfrac{V_a Q}{I_{net}(2 \text{ welds})}$ $= \dfrac{(90.0 \text{ kips})(8.74 \text{ in.}^3)}{(253 \text{ in.}^4)(2 \text{ welds})}$ $= 1.55 \text{ kip/in.}$

From mechanics of materials, the force per length that the weld must resist to transfer the force in the reinforcement plate to the beam web is:

LRFD	ASD
$r_u = \dfrac{V_u e Q}{I_{net}(2 \text{ welds})(l - c)}$ $= \dfrac{(135 \text{ kips})(9.50 \text{ in.})(8.74 \text{ in.}^3)}{(253 \text{ in.}^4)(2 \text{ welds})(17.0 \text{ in.} - 9.00 \text{ in.})}$ $= 2.77 \text{ kip/in.}$ **controls**	$r_a = \dfrac{V_a e Q}{I_{net}(2 \text{ welds})(l - c)}$ $= \dfrac{(90.0 \text{ kips})(9.50 \text{ in.})(8.74 \text{ in.}^3)}{(253 \text{ in.}^4)(2 \text{ welds})(17.0 \text{ in.} - 9.00 \text{ in.})}$ $= 1.85 \text{ kip/in.}$ **controls**

The weld capacity from AISC *Manual* Part 8:

LRFD	ASD
$\phi r_n = (1.392 \text{ kip/in.})D$ (from *Manual* Eq. 8-2a) $= (1.392 \text{ kip/in.})(4 \text{ sixteenths})$ $= 5.57 \text{ kip/in.} > 2.77 \text{ kip/in.}$ **o.k.**	$\dfrac{r_n}{\Omega} = (0.928 \text{ kip/in.})D$ (from *Manual* Eq. 8-2b) $= (0.928 \text{ kip/in.})(4 \text{ sixteenths})$ $= 3.71 \text{ kip/in.} > 1.85 \text{ kip/in.}$ **o.k.**

Determine if the web has adequate shear rupture capacity:

LRFD	ASD
$\phi = 0.75$ $\phi r_n = \phi 0.60 F_u A_{nv}$ (from *Spec*. Eq. J4-4) $= \dfrac{0.75(0.60)(65\text{ ksi})(0.400\text{ in.})}{2\text{ welds}}$ $= 5.85\text{ kip/in.} > 2.77\text{ kip/in.}$ **o.k.**	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{0.60 F_u A_{nv}}{\Omega}$ (from *Spec*. Eq. J4-4) $= \dfrac{0.60(65\text{ ksi})(0.400\text{ in.})}{2.00(2\text{ welds})}$ $= 5.85\text{ kip/in.} > 1.85\text{ kip/in.}$ **o.k.**

EXAMPLE II.A-7 BEAM END COPED AT THE TOP AND BOTTOM FLANGES

Given:

Determine the available strength for an ASTM A992 W16×40 coped 3½ in. deep by 9½ in. wide at the top flange and 2 in. deep by 9½ in. wide at the bottom flange, as shown in Figure II.A-7-1, considering the limit states of flexural yielding and local buckling. Assume a ½-in. setback from the face of the support to the end of the beam.

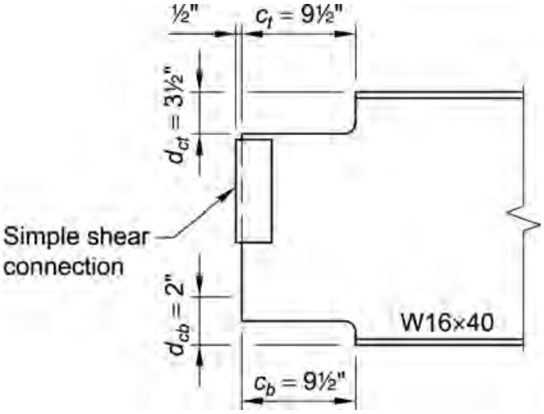


Fig. II.A-7-1. Connection geometry for Example II.A-7.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam
W16×40
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

From AISC *Manual* Table 1-1 and AISC *Manual* Figure 9-3, the geometric properties are as follows:

Beam
W16×40
$d = 16.0$ in.
$t_w = 0.305$ in.
$t_f = 0.505$ in.
$b_f = 7.00$ in.
$c_t = 9\frac{1}{2}$in.
$d_{ct} = 3\frac{1}{2}$ in.
$c_b = 9\frac{1}{2}$ in.
$d_{cb} = 2$ in.

$$\begin{aligned} e &= 9\tfrac{1}{2}\text{ in.} + \tfrac{1}{2}\text{ in.} \\ &= 10.0\text{ in.} \end{aligned}$$

$$\begin{aligned} h_o &= d - d_{ct} - d_{cb} \\ &= 16.0\text{ in.} - 3\tfrac{1}{2}\text{ in.} - 2\text{ in.} \\ &= 10.5\text{ in.} \end{aligned}$$

For a beam that is coped at both flanges, the local flexural strength is determined in accordance with AISC *Specification* Section F11.

Available Strength at Coped Section

The cope at the tension side of the beam is equal to the cope length at the compression side. From AISC *Manual* Part 9, $L_b = c_t$ and d_{ct} is the depth of the cope at the top flange.

$$C_b = \left[3 + \ln\left(\frac{L_b}{d}\right)\right]\left(1 - \frac{d_{ct}}{d}\right) \le 1.84 \qquad \text{(\textit{Manual} Eq. 9-15)}$$

$$= \left[3 + \ln\left(\frac{9\tfrac{1}{2}\text{ in.}}{16.0\text{ in.}}\right)\right]\left(1 - \frac{3\tfrac{1}{2}\text{ in.}}{16.0\text{ in.}}\right) \le 1.84$$

$$= 1.94 > 1.84$$

Use $C_b = 1.84$.

The available strength of the coped section is determined using AISC *Specification* Section F11, with $d = h_o = 10.5$ in. and unbraced length $L_b = c_t = 9\tfrac{1}{2}$ in.

$$\frac{L_b d}{t^2} = \frac{(9\tfrac{1}{2}\text{ in.})(10.5\text{ in.})}{(0.305\text{ in.})^2}$$

$$= 1{,}070$$

$$\frac{0.08E}{F_y} = \frac{0.08(29{,}000\text{ ksi})}{50\text{ ksi}}$$

$$= 46.4$$

$$\frac{1.9E}{F_y} = \frac{1.9(29{,}000\text{ ksi})}{50\text{ ksi}}$$

$$= 1{,}100$$

Since $\frac{0.08E}{F_y} < \frac{L_b d}{t^2} \le \frac{1.9E}{F_y}$, the limit state of lateral-torsional buckling applies. The nominal flexural strength of the coped portion of the web is determined using AISC *Specification* Section F11.2(b).

Determine the net elastic and plastic section moduli:

$$S_{net} = \frac{t_w h_o^2}{6}$$

$$= \frac{(0.305\text{ in.})(10.5\text{ in.})^2}{6}$$

$$= 5.60\text{ in.}^3$$

$$
\begin{aligned}
Z_{net} &= \frac{t_w h_o{}^2}{4} \\
&= \frac{(0.305\text{ in.})(10.5\text{ in.})^2}{4} \\
&= 8.41\text{ in.}^3
\end{aligned}
$$

$$
\begin{aligned}
M_y &= F_y S_{net} \\
&= (50\text{ ksi})(5.60\text{ in.}^3) \\
&= 280\text{ kip-in.}
\end{aligned}
$$

$$
\begin{aligned}
M_p &= F_y Z_{net} \\
&= (50\text{ ksi})(8.41\text{ in.}^3) \\
&= 421\text{ kip-in.}
\end{aligned}
$$

$$
\begin{aligned}
M_n &= C_b\left[1.52 - 0.274\left(\frac{L_b d}{t^2}\right)\frac{F_y}{E}\right]M_y \le M_p \qquad \text{(\textit{Spec.} Eq. F11-2)} \\
&= 1.84\left[1.52 - 0.274(1{,}070)\left(\frac{50\text{ ksi}}{29{,}000\text{ ksi}}\right)\right](280\text{ kip-in.}) \le 421\text{ kip-in.} \\
&= 523\text{ kip-in.} > 421\text{ kip-in.}
\end{aligned}
$$

The nominal moment capacity of the reduced section is 421 kip-in. The nominal strength of the coped section is:

$$
\begin{aligned}
R_n &= \frac{M_n}{e} \\
&= \frac{421\text{ kip-in.}}{10.0\text{ in.}} \\
&= 42.1\text{ kips}
\end{aligned}
$$

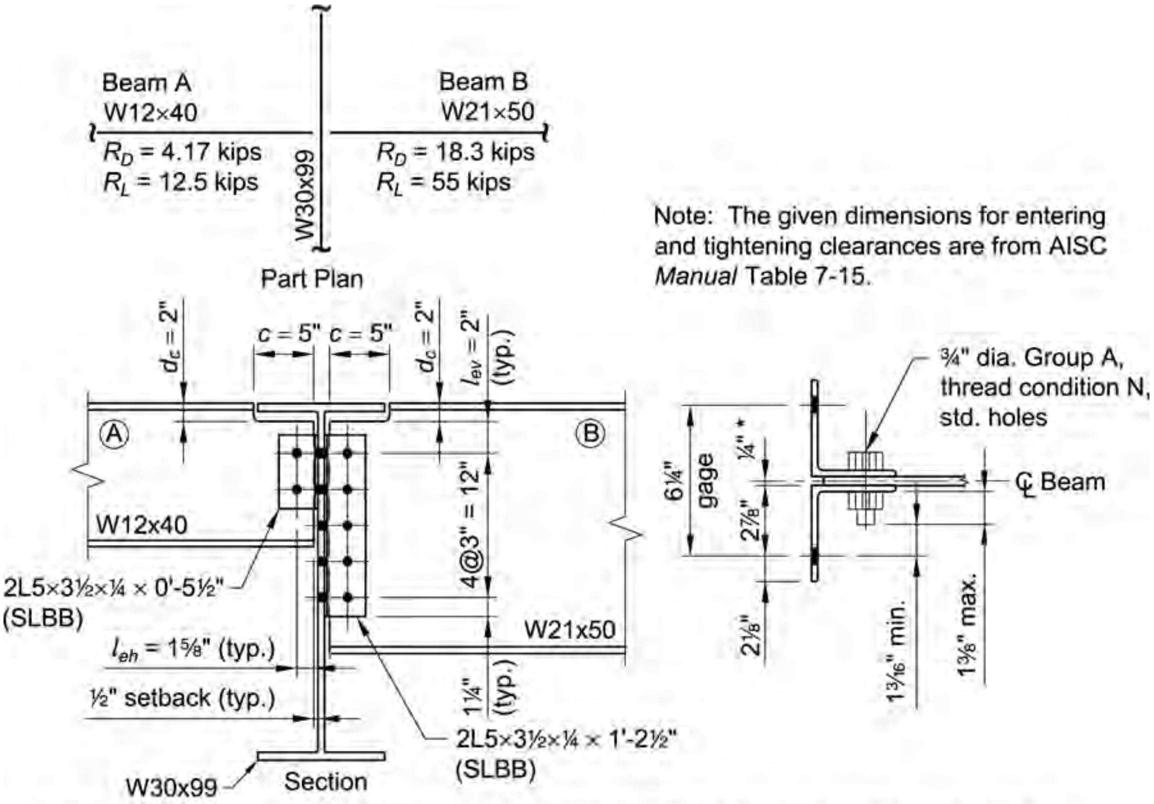
The available strength at the coped end is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b R_n = 0.90(42.1\text{ kips})$ $= 37.9\text{ kips}$	$\Omega_b = 1.67$ $\dfrac{R_n}{\Omega_b} = \dfrac{42.1\text{ kips}}{1.67}$ $= 25.2\text{ kips}$

EXAMPLE II.A-8 ALL-BOLTED DOUBLE-ANGLE CONNECTIONS (BEAMS-TO-GIRDER WEB)

Given:

Verify the all-bolted double-angle connections for back-to-back ASTM A992 W12×40 and W21×50 beams to an ASTM A992 W30×99 girder-web to support the end reactions shown in Figure II.A-8-1. Use ASTM A36 angles.



* This dimension is one-half decimal web thickness rounded to the next higher $\frac{1}{16}$ in., as in Example II.A-1.

Fig. II.A-8-1. Connection geometry for Example II.A-8.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beams and girder
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Angles
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-1 the geometric properties are as follows:

Beam
W12×40
$t_w = 0.295$ in.
$d = 11.9$ in.

Beam
W21×50
$t_w = 0.380$ in.
$d = 20.8$ in.

Girder
W30×99
$t_w = 0.520$ in.
$d = 29.7$ in.

From AISC *Specification* Table J3.3, for ¾-in.-diameter bolts with standard holes:

$d_h = {}^{13}\!/_{16}$ in.

Beam A Connection:

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(4.17 \text{ kips}) + 1.6(12.5 \text{ kips})$ $= 25.0 \text{ kips}$	$R_a = 4.17 \text{ kips} + 12.5 \text{ kips}$ $= 16.7 \text{ kips}$

Strength of Bolted Connection—Angles

AISC *Manual* Table 10-1 includes checks for the limit states of bolt shear, bolt bearing on the angles, tearout on the angles, shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles. For two rows of bolts and ¼-in. angle thickness:

LRFD	ASD
$\phi R_n = 48.9 \text{ kips} > 25.0 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 32.6 \text{ kips} > 16.7 \text{ kips}$ **o.k.**

Strength of the Bolted Connection—Beam Web

From the Commentary to AISC *Specification* Section J3.6, the strength of the bolt group is taken as the sum of the invividual strengths of the individual fasteners, taken as the lesser of the fastener shear strength per AISC *Specification* Section J3.6, the bearing strength at the bolt hole per AISC *Specification* Section J3.10, or the tearout strength at the bolt hole per AISC *Specification* Section J3.10.

From AISC *Manual* Table 7-1, the available shear strength per bolt for ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) in double shear is:

LRFD	ASD
$\phi r_n = 35.8 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 23.9 \text{ kips/bolt}$

The available bearing and tearout strength of the beam web at the top bolt is determined using AISC *Manual* Table 7-5, with $l_e = 2$ in., as follows:

LRFD	ASD
$\phi r_n = (87.8\text{ kip/in.})(0.295\text{ in.})$ $= 25.9\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (58.5\text{ kip/in.})(0.295\text{ in.})$ $= 17.3\text{ kips/bolt}$

The available bearing and tearout strength of the beam web at the bottom bolt (not adjacent to the edge) is determined using AISC *Manual* Table 7-4 with $s = 3$ in.

LRFD	ASD
$\phi r_n = (87.8\text{ kip/in.})(0.295\text{ in.})$ $= 25.9\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (58.5\text{ kip/in.})(0.295\text{ in.})$ $= 17.3\text{ kips/bolt}$

The bearing or tearout strength controls over bolt shear for both bolts in the beam web.

The strength of the bolt group in the beam web is determined by summing the strength of the individual fasteners as follows:

LRFD	ASD
$\phi R_n = (1\text{ bolt})(25.9\text{ kips/bolt})$ $+ (1\text{ bolt})(25.9\text{ kips/bolt})$ $= 51.8\text{ kips} > 25.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = (1\text{ bolt})(17.3\text{ kips/bolt})$ $+ (1\text{ bolt})(17.3\text{ kips/bolt})$ $= 34.6\text{ kips} > 16.7\text{ kips}$ **o.k.**

Coped Beam Strength

From AISC *Manual* Part 9, the available coped beam web strength is the lesser of the limit states of flexural local web buckling, shear yielding, shear rupture, and block shear rupture.

Flexural local web buckling of beam web

The limit state of flexural yielding and local web buckling of the coped beam web are checked using AISC *Manual* Part 9 as follows:

$$\begin{aligned} e &= c + setback \\ &= 5\text{ in.} + \tfrac{1}{2}\text{ in.} \\ &= 5.50\text{ in.} \end{aligned}$$

$$\begin{aligned} h_o &= d - d_c \quad \text{(from AISC } \textit{Manual} \text{ Figure 9-2)} \\ &= 11.9\text{ in.} - 2\text{ in.} \\ &= 9.90\text{ in.} \end{aligned}$$

$$\begin{aligned} \frac{c}{d} &= \frac{5\text{ in.}}{11.9\text{ in.}} \\ &= 0.420 \end{aligned}$$

$$\begin{aligned} \frac{c}{h_o} &= \frac{5\text{ in.}}{9.90\text{ in.}} \\ &= 0.505 \end{aligned}$$

Because $\dfrac{c}{d} \le 1.0,$ the buckling adjustment factor, *f*, is calculated as follows:

$$f = 2\left(\frac{c}{d}\right) \qquad \text{(\textit{Manual} Eq. 9-14a)}$$
$$= 2(0.420)$$
$$= 0.840$$

Because $\dfrac{c}{h_o} \le 1.0,$ the plate buckling coefficient, *k*, is calculated as follows:

$$k = 2.2\left(\frac{h_o}{c}\right)^{1.65} \qquad \text{(\textit{Manual} Eq. 9-13a)}$$
$$= 2.2\left(\frac{9.90\text{ in.}}{5\text{ in.}}\right)^{1.65}$$
$$= 6.79$$

$$\lambda = \frac{h_o}{t_w} \qquad \text{(\textit{Manual} Eq. 9-11)}$$
$$= \frac{9.90\text{ in.}}{0.295\text{ in.}}$$
$$= 33.6$$

$$k_1 = fk \ge 1.61 \qquad \text{(\textit{Manual} Eq. 9-10)}$$
$$= (0.840)(6.79) \ge 1.61$$
$$= 5.70 > 1.61$$

$$\lambda_p = 0.475\sqrt{\frac{k_1 E}{F_y}} \qquad \text{(\textit{Manual} Eq. 9-12)}$$
$$= 0.475\sqrt{\frac{(5.70)(29{,}000\text{ ksi})}{50\text{ ksi}}}$$
$$= 27.3$$

$$2\lambda_p = 2(27.3)$$
$$= 54.6$$

Because $\lambda_p < \lambda \le 2\lambda_p$, calculate the nominal moment strength using AISC *Manual* Equation 9-7.

The plastic section modulus of the coped section, Z_{net}, is determined from Table IV-11 (included in Part IV of this document).

$$Z_{net} = 14.0\text{ in.}^3$$

$$\begin{aligned} M_p &= F_y Z_{net} \\ &= (50\text{ ksi})(14.0\text{ in.}^3) \\ &= 700\text{ kip-in.} \end{aligned}$$

From AISC *Manual* Table 9-2:

$$S_{net} = 8.03\text{ in.}^3$$

$$\begin{aligned} M_y &= F_y S_{net} \\ &= (50\text{ ksi})(8.03\text{ in.}^3) \\ &= 402\text{ kip-in.} \end{aligned}$$

$$M_n = M_p - (M_p - M_y)\left(\frac{\lambda}{\lambda_p} - 1\right) \qquad \text{(\textit{Manual} Eq. 9-7)}$$

$$\begin{aligned} &= 700\text{ kip-in.} - (700\text{ kip-in.} - 402\text{ kip-in.})\left[\left(\frac{33.6}{27.3}\right) - 1\right] \\ &= 631\text{ kip-in.} \end{aligned}$$

$$\begin{aligned} R_n &= \frac{M_n}{e} \\ &= \frac{631\text{ kip-in.}}{5.50\text{ in.}} \\ &= 115\text{ kips} \end{aligned}$$

The available strength of the coped section is:

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(115\text{ kips})$ $= 104\text{ kips} > 25.0\text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{115\text{ kips}}{1.67}$ $= 68.9\text{ kips} > 16.7\text{ kips}$ **o.k.**

Shear strength of beam web

From AISC *Specification* Section J4.2, the available shear yielding strength of the beam web is determined as follows:

$$\begin{aligned} A_{gv} &= h_o t_w \\ &= (9.90\text{ in.})(0.295\text{ in.}) \\ &= 2.92\text{ in.}^2 \end{aligned}$$

$$R_n = 0.60 F_y A_{gv} \qquad \text{(\textit{Spec}. Eq. J4-3)}$$

$$\begin{aligned} &= 0.60(50\text{ ksi})(2.92\text{ in.}^2) \\ &= 87.6\text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(87.6\text{ kips})$ $= 87.6\text{ kips} > 25.0\text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{87.6\text{ kips}}{1.50}$ $= 58.4\text{ kips} > 16.7\text{ kips}$ **o.k.**

From AISC *Specification* Section J4.2, the available shear rupture strength of the beam web is determined as follows:

$$
\begin{aligned}
A_{nv} &= \left[h_o - n\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t_w \\
&= \left[9.90\text{ in.} - 2\left(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right](0.295\text{ in.}) \\
&= 2.40\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
R_n &= 0.60F_u A_{nv} \\
&= 0.60(65\text{ ksi})\left(2.40\text{ in.}^2\right) \\
&= 93.6\text{ kips}
\end{aligned}
\qquad (\textit{Spec}.\text{ Eq. J4-4})
$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(93.6\text{ kips})$ $= 70.2\text{ kips} > 25.0\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{93.6\text{ kips}}{2.00}$ $= 46.8\text{ kips} > 16.7\text{ kips}$ **o.k.**

Block shear rupture of beam web

The nominal strength for the limit state of block shear rupture is given by AISC *Specification* Section J4.3.

$$R_n = 0.60F_u A_{nv} + U_{bs}F_u A_{nt} \leq 0.60F_y A_{gv} + U_{bs}F_u A_{nt} \qquad (\textit{Spec}.\text{ Eq. J4-5})$$

The available block shear rupture strength of the beam web is determined as follows, using AISC *Manual* Tables 9-3a, 9-3b and 9-3c and AISC *Specification* Equation J4-5, with $n = 2$, $l_{eh} = 1\tfrac{3}{8}$ in. (includes ¼-in. tolerance to account for possible beam underrun), $l_{ev} = 2$ in. and $U_{bs} = 1.0$.

LRFD	ASD
Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{\phi F_u A_{nt}}{t} = 45.7\text{ kip/in.}$ Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{\phi 0.60F_y A_{gv}}{t} = 113\text{ kip/in.}$	Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{F_u A_{nt}}{\Omega t} = 30.5\text{ kip/in.}$ Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{0.60F_y A_{gv}}{\Omega t} = 75.0\text{ kip/in.}$

LRFD	ASD
Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{\phi 0.60 F_u A_{nv}}{t} = 108 \text{ kip/in.}$ The design block shear rupture strength is: $\phi R_n = \phi 0.60 F_u A_{nv} + \phi U_{bs} F_u A_{nt}$ $\leq \phi 0.60 F_y A_{gv} + \phi U_{bs} F_u A_{nt}$ $= (108 \text{ kip/in.} + 45.7 \text{ kip/in.})(0.295 \text{ in.})$ $\leq (113 \text{ kip/in.} + 45.7 \text{ kip/in.})(0.295 \text{ in.})$ $= 45.3 \text{ kips} < 46.8 \text{ kips}$ Therefore: $\phi R_n = 45.3 \text{ kips} > 25.0 \text{ kips}$ **o.k.**	Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{0.60 F_u A_{nv}}{\Omega t} = 71.9 \text{ kip/in.}$ The allowable block shear rupture strength is: $\dfrac{R_n}{\Omega} = \dfrac{0.60 F_u A_{nv}}{\Omega} + \dfrac{U_{bs} F_u A_{nt}}{\Omega}$ $\leq \dfrac{0.60 F_y A_{gv}}{\Omega} + \dfrac{U_{bs} F_u A_{nt}}{\Omega}$ $= (71.9 \text{ kip/in.} + 30.5 \text{ kip/in.})(0.295 \text{ in.})$ $\leq (75.0 \text{ kip/in.} + 30.5 \text{ kip/in.})(0.295 \text{ in.})$ $= 30.2 \text{ kips} < 31.1 \text{ kips}$ Therefore: $\dfrac{R_n}{\Omega} = 30.2 \text{ kips} > 16.7 \text{ kips}$ **o.k.**

Beam B Connection:

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(18.3 \text{ kips}) + 1.6(55 \text{ kips})$ $= 110 \text{ kips}$	$R_a = 18.3 \text{ kips} + 55 \text{ kips}$ $= 73.3 \text{ kips}$

Strength of the Bolted Connection—Angles

AISC *Manual* Table 10-1 includes checks for the limit states of bolt shear, bolt bearing on the angles, tearout on the angles, shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles. For five rows of bolts and ¼-in. angle thickness:

LRFD	ASD
$\phi R_n = 126 \text{ kips} > 110 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 83.8 \text{ kips} > 73.3 \text{ kips}$ **o.k.**

Strength of the Bolted Connection—Beam Web

From AISC *Manual* Table 7-1, the available shear strength per bolt for ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) in double shear is:

LRFD	ASD
$\phi r_n = 35.8 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 23.9 \text{ kips/bolt}$

The available bearing and tearout strength of the beam web at the top edge bolt is determined using AISC *Manual* Table 7-5 with $l_e = 2$ in.

LRFD	ASD
$\phi r_n = (87.8\text{ kip/in.})(0.380\text{ in.})$ $= 33.4\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (58.5\text{ kip/in.})(0.380\text{ in.})$ $= 22.2\text{ kips/bolt}$

The available bearing and tearout strength of the beam web at the interior bolts (not adjacent to the edge) is determined using AISC *Manual* Table 7-4 with $s = 3$ in.

LRFD	ASD
$\phi r_n = (87.8\text{ kip/in.})(0.380\text{ in.})$ $= 33.4\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (58.5\text{ kip/in.})(0.380\text{ in.})$ $= 22.2\text{ kips/bolt}$

The strength of the bolt group in the beam web is determined as follows:

LRFD	ASD
$\phi R = (1\text{ bolt})(33.4\text{ kips/bolt})$ $+ (4\text{ bolts})(33.4\text{ kips/bolt})$ $= 167\text{ kips} > 110\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = (1\text{ bolt})(22.2\text{ kips/bolt})$ $+ (4\text{ bolt})(22.2\text{ kips/bolt})$ $= 111\text{ kips} > 73.3\text{ kips}$ **o.k.**

Coped Beam Strength

From AISC *Manual* Part 9, the available coped beam web strength is the lesser of the limit states of flexural local web buckling, shear yielding, shear rupture, and block shear rupture.

Flexural local web buckling of beam web

The limit state of flexural yielding and local web buckling of the coped beam web are checked using AISC *Manual* Part 9 as follows:

$$\begin{aligned} e &= c + setback \\ &= 5\text{ in.} + \tfrac{1}{2}\text{ in.} \\ &= 5.50\text{ in.} \end{aligned}$$

$$\begin{aligned} h_o &= d - d_c \quad \text{(from AISC } \textit{Manual} \text{ Figure 9-2)} \\ &= 20.8\text{ in.} - 2\text{ in.} \\ &= 18.8\text{ in.} \end{aligned}$$

$$\begin{aligned} \frac{c}{d} &= \frac{5\text{ in.}}{20.8\text{ in.}} \\ &= 0.240 \end{aligned}$$

$$\begin{aligned} \frac{c}{h_o} &= \frac{5\text{ in.}}{18.8\text{ in.}} \\ &= 0.266 \end{aligned}$$

Because $\dfrac{c}{d} \le 1.0,$ the buckling adjustment factor, f, is calculated as follows:

$$\begin{aligned} f &= 2\left(\frac{c}{d}\right) \\ &= 2(0.240) \\ &= 0.480 \end{aligned}$$ (*Manual* Eq. 9-14a)

Because $\frac{c}{h_o} \le 1.0$, the plate buckling coefficient, *k*, is calculated as follows:

$$\begin{aligned} k &= 2.2\left(\frac{h_o}{c}\right)^{1.65} \\ &= 2.2\left(\frac{18.8\text{ in.}}{5\text{ in.}}\right)^{1.65} \\ &= 19.6 \end{aligned}$$ (*Manual* Eq. 9-13a)

$$\begin{aligned} \lambda &= \frac{h_o}{t_w} \\ &= \frac{18.8\text{ in.}}{0.380\text{ in.}} \\ &= 49.5 \end{aligned}$$ (*Manual* Eq. 9-11)

$$\begin{aligned} k_1 &= fk \ge 1.61 \\ &= (0.480)(19.6) \ge 1.61 \\ &= 9.41 > 1.61 \end{aligned}$$ (*Manual* Eq. 9-10)

$$\begin{aligned} \lambda_p &= 0.475\sqrt{\frac{k_1 E}{F_y}} \\ &= 0.475\sqrt{\frac{(9.41)(29{,}000\text{ ksi})}{50\text{ ksi}}} \\ &= 35.1 \end{aligned}$$ (*Manual* Eq. 9-12)

$$\begin{aligned} 2\lambda_p &= 2(35.1) \\ &= 70.2 \end{aligned}$$

Because $\lambda_p < \lambda \le 2\lambda_p$, calculate the nominal moment strength using AISC *Manual* Equation 9-7.

The plastic section modulus of the coped section, Z_{net}, is determined from Table IV-11 (included in Part IV of this document).

$$Z_{net} = 56.5\text{ in.}^3$$

$$\begin{aligned} M_p &= F_y Z_{net} \\ &= (50\text{ ksi})(56.5\text{ in.}^3) \\ &= 2{,}830\text{ kip-in.} \end{aligned}$$

From AISC *Manual* Table 9-2:

$S_{net} = 32.5 \text{ in.}^3$

$$\begin{aligned} M_y &= F_y S_{net} \\ &= (50 \text{ ksi})(32.5 \text{ in.}^3) \\ &= 1{,}630 \text{ kip-in.} \end{aligned}$$

$$M_n = M_p - (M_p - M_y)\left(\frac{\lambda}{\lambda_p} - 1\right) \qquad (\textit{Manual} \text{ Eq. 9-7})$$

$$\begin{aligned} &= 2{,}830 \text{ kip-in.} - (2{,}830 \text{ kip-in.} - 1{,}630 \text{ kip-in.})\left[\left(\frac{49.5}{35.1}\right) - 1\right] \\ &= 2{,}340 \text{ kip-in.} \end{aligned}$$

$$\begin{aligned} R_n &= \frac{M_n}{e} \\ &= \frac{2{,}340 \text{ kip-in.}}{5.50 \text{ in.}} \\ &= 425 \text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(425 \text{ kips})$ $= 383 \text{ kips} > 110 \text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{425 \text{ kips}}{1.67}$ $= 254 \text{ kips} > 73.3 \text{ kips}$ **o.k.**

Shear strength of beam web

From AISC *Specification* Section J4.2, the available shear yielding strength of the beam web is determined as follows:

$$\begin{aligned} A_{gv} &= h_o t_w \\ &= (18.8 \text{ in.})(0.380 \text{ in.}) \\ &= 7.14 \text{ in.}^2 \end{aligned}$$

$$R_n = 0.60 F_y A_{gv} \qquad (\textit{Spec}. \text{ Eq. J4-3})$$

$$\begin{aligned} &= 0.60(50 \text{ ksi})(7.14 \text{ in.}^2) \\ &= 214 \text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(214 \text{ kips})$ $= 214 \text{ kips} > 110 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{214 \text{ kips}}{1.50}$ $= 143 \text{ kips} > 73.3 \text{ kips}$ **o.k.**

From AISC *Specification* Section J4.2, the available shear rupture strength of the beam web is determined as follows:

$$
\begin{aligned}
A_{nv} &= \left[h_o - n\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t_w \\
&= \left[18.8\text{ in.} - 5\left(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right](0.380\text{ in.}) \\
&= 5.48\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
R_n &= 0.60F_uA_{nv} \\
&= 0.60(65\text{ ksi})\left(5.48\text{ in.}^2\right) \\
&= 214\text{ kips}
\end{aligned}
$$

(*Spec*. Eq. J4-4)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(214\text{ kips})$ $= 161\text{ kips} > 110\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{214\text{ kips}}{2.00}$ $= 107\text{ kips} > 73.3\text{ kips}$ **o.k.**

Block shear rupture of beam web

The nominal strength for the limit state of block shear rupture is given by AISC *Specification* Section J4.3.

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.60F_yA_{gv} + U_{bs}F_uA_{nt}$$

(*Spec*. Eq. J4-5)

The available block shear rupture strength of the beam web is determined as follows, using AISC *Manual* Tables 9-3a, 9-3b and 9-3c and AISC *Specification* Equation J4-5, with n = 5, l_{eh} = 1⅜ in. (includes ¼ in. tolerance to account for possible beam underrun), l_{ev} = 2 in. and U_{bs} = 1.0.

LRFD	ASD
Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{\phi F_uA_{nt}}{t} = 45.7\text{ kip/in.}$	Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{F_uA_{nt}}{\Omega t} = 30.5\text{ kip/in.}$
Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{\phi 0.60F_yA_{gv}}{t} = 315\text{ kip/in.}$	Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{0.60F_yA_{gv}}{\Omega t} = 210\text{ kip/in.}$
Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{\phi 0.60F_uA_{nv}}{t} = 294\text{ kip/in.}$	Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{0.60F_uA_{nv}}{\Omega t} = 196\text{ kip/in.}$

LRFD	ASD
$\phi R_n = \phi 0.60 F_u A_{nv} + \phi U_{bs} F_u A_{nt}$ $\leq \phi 0.60 F_y A_{gv} + \phi U_{bs} F_u A_{nt}$ $= (294 \text{ kip/in.} + 45.7 \text{ kip/in.})(0.380 \text{ in.})$ $\leq (315 \text{ kip/in.} + 45.7 \text{ kip/in.})(0.380 \text{ in.})$ $= 129 \text{ kips} < 137 \text{ kips}$ Therefore: $\phi R_n = 129 \text{ kips} > 110 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{0.60 F_u A_{nv}}{\Omega} + \dfrac{U_{bs} F_u A_{nt}}{\Omega}$ $\leq \dfrac{0.60 F_y A_{gv}}{\Omega} + \dfrac{U_{bs} F_u A_{nt}}{\Omega}$ $= (196 \text{ kip/in.} + 30.5 \text{ kip/in.})(0.380 \text{ in.})$ $\leq (210 \text{ kip/in.} + 30.5 \text{ kip/in.})(0.380 \text{ in.})$ $= 86.1 \text{ kips} < 91.4 \text{ kips}$ Therefore: $\dfrac{R_n}{\Omega} = 86.1 \text{ kips} > 73.3 \text{ kips}$ **o.k.**

Supporting Girder Connection

Supporting Girder Web

The required effective strength per bolt is the minimum from the limit states of bolt shear, bolt bearing and tearout. The bolts that are loaded by both connections will have the largest demand.. Thus, for the design of these four critical bolts, the required strength is determined as follows:

LRFD	ASD
From the W12×40 beam, each bolt must support one-fourth of 25.0 kips or 6.25 kips/bolt. From the W21×50 beam, each bolt must support one-tenth of 110 kips or 11.0 kips/bolt.	From the W12×40 beam, each bolt must support one-fourth of 16.7 kips or 4.18 kips/bolt. From the W21×50 beam, each bolt must support one-tenth of 73.3 kips or 7.33 kips/bolt.

The required strength for each of the shared bolts is:

LRFD	ASD
$R_u = 6.25 \text{ kips/bolt} + 11.0 \text{ kips/bolt}$ $= 17.3 \text{ kips/bolt}$	$R_a = 4.18 \text{ kips/bolt} + 7.33 \text{ kips/bolt}$ $= 11.5 \text{ kips/bolt}$

From AISC *Manual* Table 7-1, the available shear strength per bolt for ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) in double shear is:

LRFD	ASD
$\phi r_n = 35.8 \text{ kips/bolt} > 17.3 \text{ kips/bolt}$ **o.k.**	$\dfrac{r_n}{\Omega} = 23.9 \text{ kips/bolt} > 11.5 \text{ kips/bolt}$ **o.k.**

The available bearing and tearout strength of the girder web is determined using AISC *Manual* Table 7-4 with $s = 3$ in.

LRFD	ASD
$\phi r_n = (87.8 \text{ kip/in.})(0.520 \text{ in.})$ $= 45.7 \text{ kips/bolt} > 17.3 \text{ kips/bolt}$ **o.k.**	$\dfrac{r_n}{\Omega} = (58.5 \text{ kip/in.})(0.520 \text{ in.})$ $= 30.4 \text{ kips/bolt} > 11.5 \text{ kips/bolt}$ **o.k.**

Conclusion

The connection is found to be adequate as given for the applied loads.

EXAMPLE II.A-9 OFFSET ALL-BOLTED DOUBLE-ANGLE CONNECTIONS (BEAMS-TO-GIRDER WEB)

Given:

Verify the all-bolted double-angle connections for back-to-back ASTM A992 W16×45 beams to an ASTM A992 W30×99 girder-web to support the end reactions shown in Figure II.A-9-1. The beam centerlines are offset 6 in. and the beam connections share a vertical row of bolts. Use ASTM A36 angles. The strength of the W16×45 beams and angles are verified in Example II.A-4 and are not repeated here.

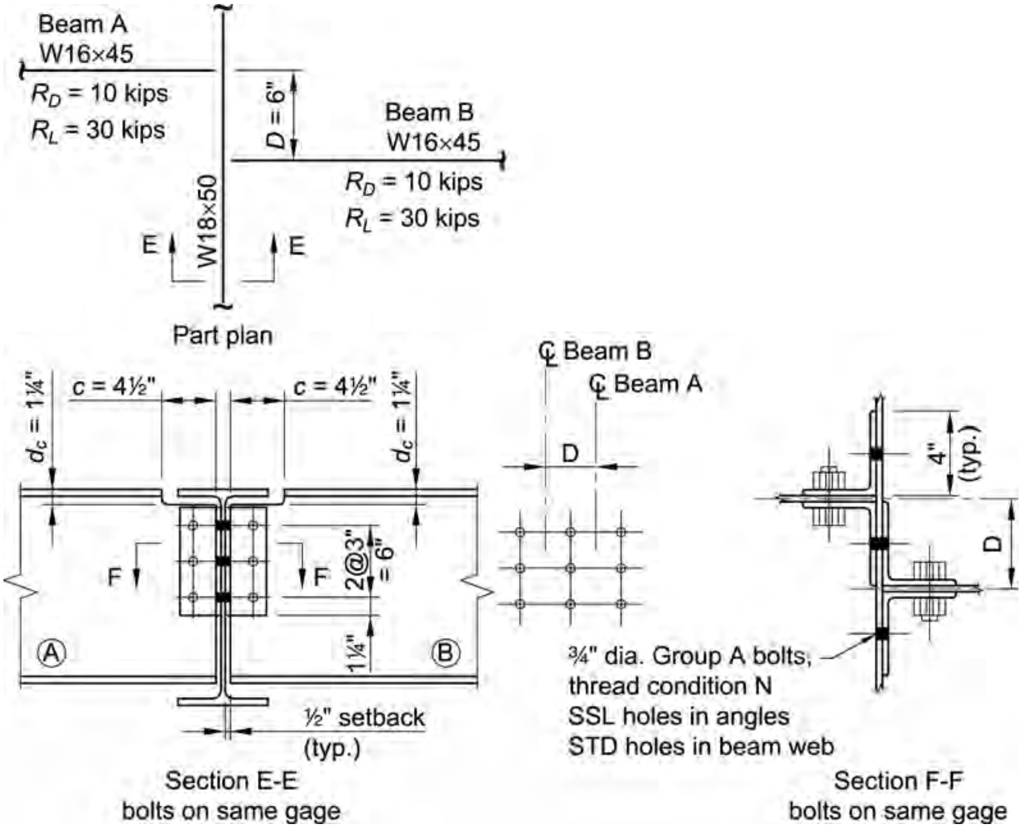


Fig. II.A-9-1. Connection geometry for Example II.A-9.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beams and girder
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Angles
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Girder
W18×50
t_w = 0.355 in.
d = 18.0 in.

Beam
W16×45
t_w = 0.345 in.
d = 16.1 in.

Modify the 2L5×3½×¼ SLBB connection designed in Example II.A-4 to work in the configuration shown in Figure II.A-9-1. The offset dimension (6 in.) is approximately equal to the gage on the support from the previous example (6¼ in.) and, therefore, is not recalculated.

Thus, the available strength of the middle vertical row of bolts (through both connections) that carry a portion of the reaction for both connections must be verified for this new configuration.

From ASCE/SEI 7, Chapter 2, the required strength of the Beam A and Beam B connections to the girder web is:

LRFD	ASD
$R_u = 1.2(10 \text{ kips}) + 1.6(30 \text{ kips})$ $= 60.0 \text{ kips}$	$R_a = 10 \text{ kips} + 30 \text{ kips}$ $= 40.0 \text{ kips}$

In the girder web connection, each bolt will have the same effective strength; therefore, check the individual bolt effective strength. At the middle vertical row of bolts, the required strength for one bolt is the sum of the required shear strengths per bolt for each connection.

LRFD	ASD
$r_u = (2 \text{ sides})\left(\dfrac{60.0 \text{ kips}}{6 \text{ bolts}}\right)$ $= 20.0 \text{ kips/bolt (for middle vertical row)}$	$r_a = (2 \text{ sides})\left(\dfrac{40.0 \text{ kips}}{6 \text{ bolts}}\right)$ $= 13.3 \text{ kips/bolt (for middle vertical row)}$

Bolt Shear

From AISC *Manual* Table 7-1, the available shear strength per bolt for ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) in double shear is:

LRFD	ASD
$\phi r_n = 35.8 \text{ kips/bolt} > 20.0 \text{ kips/bolt}$ **o.k.**	$\dfrac{r_n}{\Omega} = 23.9 \text{ kips/bolt} > 13.3 \text{ kips/bolt}$ **o.k.**

Bearing on the Girder Web

The available bearing strength per bolt is determined from AISC *Manual* Table 7-4 with s = 3 in.

LRFD	ASD
$\phi r_n = (87.8 \text{ kip/in.})(0.355 \text{ in.})$ $= 31.2 \text{ kips/bolt} > 20.0 \text{ kips/bolt}$ **o.k.**	$\dfrac{r_n}{\Omega} = (58.5 \text{ kip/in.})(0.355 \text{ in.})$ $= 20.8 \text{ kips/bolt} > 13.3 \text{ kips/bolt}$ **o.k.**

Note: If the bolts are not spaced equally from the supported beam web, the force in each column of bolts should be determined by using a simple beam analogy between the bolts, and applying the laws of statics.

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Conclusion

The connections are found to be adequate as given for the applied loads.

EXAMPLE II.A-10 SKEWED DOUBLE BENT-PLATE CONNECTION (BEAM-TO-GIRDER WEB)

Given:

Design the skewed double bent-plate connection between an ASTM A992 W16×77 beam and ASTM A992 W27×94 girder-web to support the following beam end reactions:

R_D = 13.3 kips
R_L = 40 kips

Use 70-ksi electrodes and ASTM A36 plates. The final design is shown in Figure II.A-10-1.

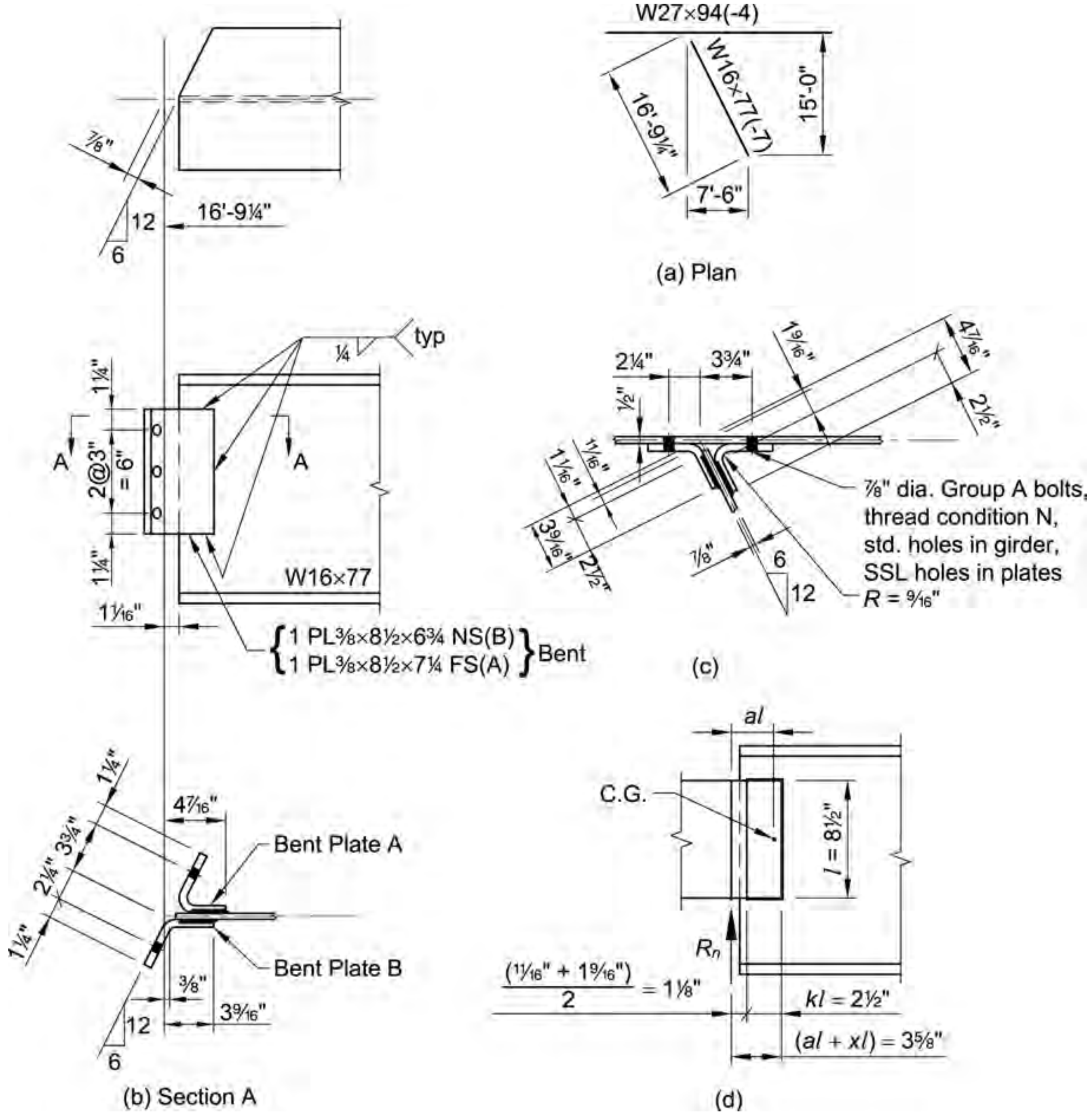

Fig. II.A-10-1. Skewed double bent-plate connection (beam-to-girder web).

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Beam and girder
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Plate
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W16×77
$t_w = 0.455$ in.
$d = 16.5$ in.

Girder
W27×94
$t_w = 0.490$ in.

From AISC *Specification* Table J3.3, for ⅞-in.-diameter bolts with standard holes:

$d_h = {}^{15}\!/_{16}$ in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(13.3\text{ kips}) + 1.6(40\text{ kips})$ $= 80.0\text{ kips}$	$R_a = 13.3\text{ kips} + 40\text{ kips}$ $= 53.3\text{ kips}$

From Figure II.A-10-1(c), assign load to each vertical row of bolts by assuming a simple beam analogy between bolts and applying the principles of statics.

LRFD	ASD
Required strength for bent plate A: $R_u = \dfrac{(80.0\text{ kips})(2\tfrac{1}{4}\text{ in.})}{6.00\text{ in.}}$ $= 30.0\text{ kips}$	Required strength for bent plate A: $R_a = \dfrac{(53.3\text{ kips})(2\tfrac{1}{4}\text{ in.})}{6.00\text{ in.}}$ $= 20.0\text{ kips}$
Required strength for bent plate B: $R_u = 80.0\text{ kips} - 30.0\text{ kips}$ $= 50.0\text{ kips}$	Required strength for bent plate B: $R_a = 53.3\text{ kips} - 20.0\text{ kips}$ $= 33.3\text{ kips}$

Assume that the welds across the top and bottom of the plates will be 2½ in. long, and that the load acts at the intersection of the beam centerline and the support face.

While the welds do not coincide on opposite faces of the beam web and the weld groups are offset, the locations of the weld groups will be averaged and considered identical. See Figure II.A-10-1(d).

Weld Design

Assume a plate length of l = 8½ in.

$$\begin{aligned} k &= \frac{kl}{l} \\ &= \frac{2\tfrac{1}{2}\text{ in.}}{8\tfrac{1}{2}\text{ in.}} \\ &= 0.294 \end{aligned}$$

Interpolating from AISC *Manual* Table 8-8, with angle = 0°, and $k = 0.294$,

$$x = 0.0544$$

$$\begin{aligned} xl &= (0.0544)(8\tfrac{1}{2}\text{ in.}) \\ &= 0.462\text{ in.} \end{aligned}$$

$$\begin{aligned} a &= \frac{(al + xl) - xl}{l} \\ &= \frac{3\tfrac{5}{8}\text{ in} - 0.462\text{ in.}}{8\tfrac{1}{2}\text{ in.}} \\ &= 0.372 \end{aligned}$$

Interpolating from AISC *Manual* Table 8-8, with θ = 0°, $a = 0.372$, and $k = 0.294$,

$$C = 2.52$$

The required weld size is determined as follows:

LRFD	ASD
$\phi = 0.75$ $D_{req} = \dfrac{R_u}{\phi C C_1 l}$ $= \dfrac{50.0\text{ kips}}{0.75(2.52)(1.0)(8\tfrac{1}{2}\text{ in.})}$ $= 3.11$ sixteenths	$\Omega = 2.00$ $D_{req} = \dfrac{\Omega R_a}{C C_1 l}$ $= \dfrac{2.00(33.3\text{ kips})}{2.52(1.0)(8\tfrac{1}{2}\text{ in.})}$ $= 3.11$ sixteenths

Use ¼-in. fillet welds and at least ⁵⁄₁₆-in.-thick bent plates to allow for the welds.

Beam Web Strength at Fillet Weld

The minimum beam web thickness required to match the shear rupture strength of the weld to that of the base metal is:

$$t_{min} = \frac{6.19D_{min}}{F_u}$$ (from *Manual* Eq. 9-3)

$$= \frac{6.19(3.11)}{65\text{ ksi}}$$

$$= 0.296\text{ in.} < 0.455\text{ in.}\quad \textbf{o.k.}$$

Bolt Strength

The effective strength of the individual fasteners is the lesser of the bolt shear strength per AISC *Specification* Section J3.6, and the bolt bearing and tearout strength per AISC *Specification* Section J3.10. By observation, the bent plate will govern over the girder web as it is thinner and lower strength material. Trying a 5⁄16-in. plate the available strength at the critical vertical row of bolts (bent plate B) is determined as follows.

From AISC *Manual* Table 7-1, the available shear strength per bolt for ⅞-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) in single shear is:

LRFD	ASD
$\phi r_n = 24.3\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 16.2\text{ kips/bolt}$

The available bearing and tearout strength of the bent-plate at the top edge bolt is determined using AISC *Manual* Table 7-5 with l_{ev} = 1¼ in.

LRFD	ASD
$\phi r_n = (40.8\text{ kip/in.})(5/16\text{ in.})$ $= 12.8\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (27.2\text{ kip/in.})(5/16\text{ in.})$ $= 8.50\text{ kips/bolt}$

The available bearing and tearout strength of the bent-plate at the other bolts (not adjacent to the edge) is determined using AISC *Manual* Table 7-4 with s = 3 in.

LRFD	ASD
$\phi r_n = (91.4\text{ kip/in.})(5/16\text{ in.})$ $= 28.6\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (60.9\text{ kip/in.})(5/16\text{ in.})$ $= 19.0\text{ kips/bolt}$

The bolt shear strength governs over bearing and tearout for the other bolts (not adjacent to the edge); therefore, the effective strength of the bolt group is determined as follows:

LRFD	ASD
$\phi R_n = (1\text{ bolt})(12.8\text{ kips/bolt})$ $+ (2\text{ bolts})(24.3\text{ kips/bolt})$ $= 61.4\text{ kips} > 50.0\text{ kips}\quad \textbf{o.k.}$	$\dfrac{R_n}{\Omega} = (1\text{ bolt})(8.50\text{ kips/bolt})$ $+ (2\text{ bolts})(16.2\text{ kips/bolt})$ $= 40.9\text{ kips} > 33.3\text{ kips}\quad \textbf{o.k.}$

Shear Strength of Plate

From AISC *Specification* Section J4.2, the available shear yielding strength of bent plate B (see Figure II.A-10-1) is determined as follows:

$$\begin{aligned} A_{gv} &= lt \\ &= (8\tfrac{1}{2}\text{ in.})(\tfrac{5}{16}\text{ in.}) \\ &= 2.66\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60F_yA_{gv} \\ &= 0.60(36\text{ ksi})(2.66\text{ in.}^2) \\ &= 57.5\text{ kips} \end{aligned}$$ (*Spec*. Eq. J4-3)

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(57.5\text{ kips})$ $= 57.5\text{ kips} > 50.0\text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{57.5\text{ kips}}{1.50}$ $= 38.3\text{ kips} > 33.3\text{ kips}$ **o.k.**

From AISC *Specification* Section J4.2, the available shear rupture strength of bent plate B is determined as follows:

$$\begin{aligned} A_{nv} &= \left[l - n\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t \\ &= \left[8\tfrac{1}{2}\text{ in.} - 3\left(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right](\tfrac{5}{16}\text{ in.}) \\ &= 1.72\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60F_uA_{nv} \\ &= 0.60(58\text{ ksi})(1.72\text{ in.}^2) \\ &= 59.9\text{ kips} \end{aligned}$$ (*Spec*. Eq. J4-4)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(59.9\text{ kips})$ $= 44.9\text{ kips} < 50.0\text{ kips}$ **n.g.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{59.9\text{ kips}}{2.00}$ $= 30.0\text{ kips} < 33.3\text{ kips}$ **n.g.**

Therefore, the plate thickness is increased to ⅜ in. The available shear rupture strength is:

$$\begin{aligned} A_{nv} &= \left[d - n\left(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right]t \\ &= \left[8\tfrac{1}{2}\text{ in.} - 3\left(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right](\tfrac{3}{8}\text{ in.}) \\ &= 2.06\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60F_uA_{nv} \\ &= 0.60(58\text{ ksi})(2.06\text{ in.}^2) \\ &= 71.7\text{ kips} \end{aligned}$$ (*Spec*. Eq. J4-4)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(71.7 \text{ kips})$ $= 53.8 \text{ kips} > 50.0 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{71.7 \text{ kips}}{2.00}$ $= 35.9 \text{ kips} > 33.3 \text{ kips}$ **o.k.**

Block Shear Rupture of Plate

The nominal strength for the limit state of block shear rupture is given by AISC *Specification* Section J4.3.

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad (\textit{Spec.}\text{ Eq. J4-5})$$

The available block shear rupture strength of the plate is determined as follows, using AISC *Manual* Tables 9-3a, 9-3b and 9-3c and AISC *Specification* Equation J4-5, with $n = 3$, $l_{ev} = l_{eh} = 1\frac{1}{4}$ in., and $U_{bs} = 1.0$.

LRFD	ASD
Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{\phi F_uA_{nt}}{t} = 32.6 \text{ kip/in.}$	Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{F_uA_{nt}}{\Omega t} = 21.8 \text{ kip/in.}$
Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{\phi 0.6F_yA_{gv}}{t} = 117 \text{ kip/in.}$	Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{0.6F_yA_{gv}}{\Omega t} = 78.3 \text{ kip/in.}$
Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{\phi 0.6F_uA_{nv}}{t} = 124 \text{ kip/in.}$	Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{0.6F_uA_{nv}}{\Omega t} = 82.6 \text{ kip/in.}$
$\phi R_n = \phi 0.60F_uA_{nv} + \phi U_{bs}F_uA_{nt}$ $\leq \phi 0.60F_yA_{gv} + \phi U_{bs}F_uA_{nt}$ $= (124 \text{ kip/in.} + 32.6 \text{ kip/in.})(\frac{3}{8} \text{ in.})$ $\leq (117 \text{ kip/in.} + 32.6 \text{ kip/in.})(\frac{3}{8} \text{ in.})$ $= 58.7 \text{ kips} > 56.1 \text{ kips}$	$\dfrac{R_n}{\Omega} = \dfrac{0.60F_uA_{nv}}{\Omega} + \dfrac{U_{bs}F_uA_{nt}}{\Omega}$ $\leq \dfrac{0.60F_yA_{gv}}{\Omega} + \dfrac{U_{bs}F_uA_{nt}}{\Omega}$ $= (82.6 \text{ kip/in.} + 21.8 \text{ kip/in.})(\frac{3}{8} \text{ in.})$ $\leq (78.3 \text{ kip/in.} + 21.8 \text{ kip/in.})(\frac{3}{8} \text{ in.})$ $= 39.2 \text{ kips} > 37.5 \text{ kips}$
Therefore: $\phi R_n = 56.1 \text{ kips} > 50.0 \text{ kips}$ **o.k.**	Therefore: $\dfrac{R_n}{\Omega} = 37.5 \text{ kips} > 33.3 \text{ kips}$ **o.k.**

Thus, the configuration shown in Figure II.A-10-1 can be supported using ⅜-in. bent plates, and ¼-in. fillet welds.

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EXAMPLE II.A-11A SHEAR END-PLATE CONNECTION (BEAM-TO-GIRDER WEB)

Given:

Verify a shear end-plate connection to connect an ASTM A992 W18×50 beam to an ASTM A992 W21×62 girder web, as shown in Figure II.A-11A-1, to support the following beam end reactions:

$R_D = 10$ kips
$R_L = 30$ kips

Use 70-ksi electrodes and ASTM A36 plate.

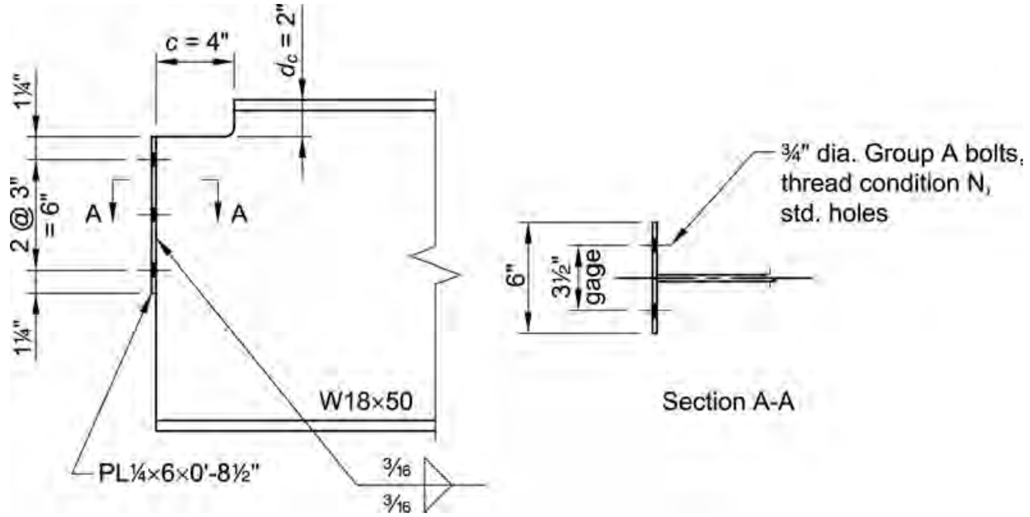

Fig. II.A-11A-1. Connection geometry for Example II.A-11A.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Beam and girder
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Plate
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W18×50
$t_w = 0.355$ in.

Girder
W21×62
$t_w = 0.400$ in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(10\text{ kips}) + 1.6(30\text{ kips})$ $= 60.0\text{ kips}$	$R_a = 10\text{ kips} + 30\text{ kips}$ $= 40.0\text{ kips}$

Bolt and End-Plate Available Strength

Tabulated values in AISC *Manual* Table 10-4 consider the limit states of bolt shear, bolt bearing on the end plate, tearout on the end plate, shear yielding of the end plate, shear rupture of the end plate, and block shear rupture of the end plate.

From AISC *Manual* Table 10-4, for three rows of ¾-in.-diameter bolts and ¼-in. plate thickness:

LRFD	ASD
$\phi R_n = 76.4\text{ kips} > 60.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 50.9\text{ kips} > 40.0\text{ kips}$ **o.k.**

Weld and Beam Web Available Strength

Try $^3/_{16}$-in. weld. From AISC *Manual* Table 10-4, the minimum beam web thickness is:

$$t_{w\,min} = 0.286\text{ in.} < 0.355\text{ in.} \quad \textbf{o.k.}$$

From AISC *Manual* Table 10-4, the weld and beam web available strength is:

LRFD	ASD
$\phi R_n = 67.9\text{ kips} > 60.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 45.2\text{ kips} > 40.0\text{ kips}$ **o.k.**

Bolt Bearing on Girder Web

From AISC *Manual* Table 10-4:

LRFD	ASD
$\phi R_n = (527\text{ kip/in.})(0.400\text{ in.})$ $= 211\text{ kips} > 60.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = (351\text{ kip/in.})(0.400\text{ in.})$ $= 140\text{ kips} > 40.0\text{ kips}$ **o.k.**

Coped Beam Strength

As was shown in Example II.A-4, the coped section does not control the design. **o.k.**

Beam Web Shear Yielding

As was shown in Example II.A-4, beam web shear does not control the design. **o.k.**

EXAMPLE II.A-11B END-PLATE CONNECTION SUBJECT TO AXIAL AND SHEAR LOADING

Given:

Verify the available strength of an end-plate connection for an ASTM A992 W18x50 beam, as shown in Figure II.A-11B-1, to support the following beam end reactions:

LRFD	ASD
Shear, V_u = 75 kips Axial tension, N_u = 60 kips	Shear, V_a = 50 kips Axial tension, N_a = 40 kips

Use 70-ksi electrodes and ASTM A36 plate.

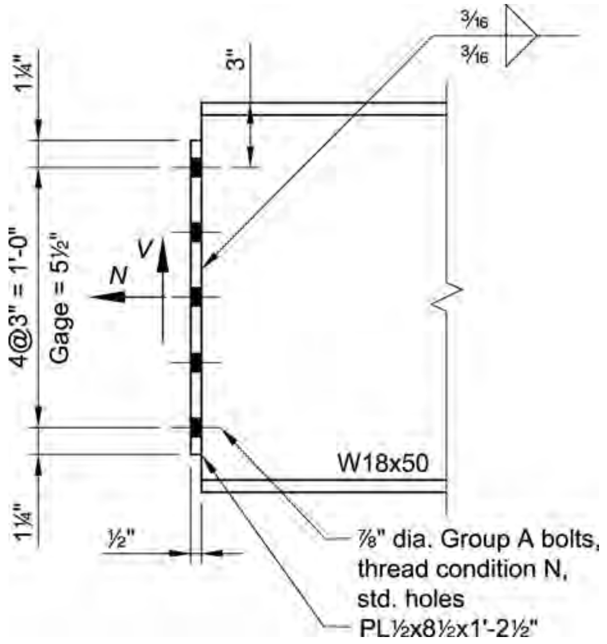

Fig. II.A-11B-1. Connection geometry for Example II.A-11B.

Solution:

From AISC *Manual* Table 2-4 and 2-5, the material properties are as follows:

Beam
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Plate
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W18×50
d = 18.0 in.

t_w = 0.355 in.
A_g = 14.7 in.^{2}

From AISC *Specification* Table J3.3, for ⅞-in.-diameter bolts with standard holes:

d_h = 15⁄16 in.

The resultant load is:

LRFD	ASD
$R_u = \sqrt{V_u{}^2 + N_u{}^2}$ $= \sqrt{(75\text{ kips})^2 + (60\text{ kips})^2}$ $= 96.0\text{ kips}$	$R_a = \sqrt{V_a{}^2 + N_a{}^2}$ $= \sqrt{(50\text{ kips})^2 + (40\text{ kips})^2}$ $= 64.0\text{ kips}$

The connection will first be checked for the shear load. The following bolt shear, bearing and tearout calculations are for a pair of bolts.

Bolt Shear

From AISC *Manual* Table 7-1, the available shear strength for ⅞-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) in double shear, or pair of bolts in this example, is:

LRFD	ASD
$\phi r_n = 48.7\text{ kips/pair of bolts}$	$\dfrac{r_n}{\Omega} = 32.5\text{ kips/pair of bolts}$

Bolt Bearing on the Plate

The nominal bearing strength of the plate is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration:

$$\begin{aligned} r_n &= (2\text{ bolts/row})2.4dtF_u \qquad \text{(from } \textit{Spec.} \text{ Eq. J3-6a)} \\ &= (2\text{ bolts/row})(2.4)(⅞\text{ in.})(½\text{ in.})(58\text{ ksi}) \\ &= 122\text{ kips (for a pair of bolts)} \end{aligned}$$

From AISC *Specification* Section J3.10, the available bearing strength of the plate for a pair of bolts is:

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(122\text{ kips})$ $= 91.5\text{ kips/pair of bolts}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{122\text{ kips}}{2.00}$ $= 61.0\text{ kips/pair of bolts}$

Bolt Tearout on the Plate

The available tearout strength of the plate is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration. For the top edge bolts:

$$
\begin{aligned}
l_c &= l_e - 0.5d_h \\
&= 1\tfrac{1}{4}\text{ in.} - 0.5\left(\tfrac{15}{16}\text{ in.}\right) \\
&= 0.781\text{ in.}
\end{aligned}
$$

$$
\begin{aligned}
r_n &= (2\text{ bolts/row})1.2l_ctF_u \\
&= (2\text{ bolts/row})(1.2)(0.781\text{ in.})(\tfrac{1}{2}\text{ in.})(58\text{ ksi}) \\
&= 54.4\text{ kips (for a pair of bolts)}
\end{aligned}
$$

(from *Spec*. Eq. J3-6c)

The available bolt tearout strength for the pair of top edge bolts is:

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(54.4\text{ kips})$ $= 40.8\text{ kips/pair of bolts}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{54.4\text{ kips}}{2.00}$ $= 27.2\text{ kips/pair of bolts}$

Tearout controls over bolt shear and bearing strength for the top edge bolts in the plate.

For interior bolts:

$$
\begin{aligned}
l_c &= s - d_h \\
&= 3.00\text{ in.} - \tfrac{15}{16}\text{ in.} \\
&= 2.06\text{ in.}
\end{aligned}
$$

$$
\begin{aligned}
r_n &= (2\text{ bolts/row})1.2l_ctF_u \\
&= (2\text{ bolts/row})(1.2)(2.06\text{ in.})(\tfrac{1}{2}\text{ in.})(58\text{ ksi}) \\
&= 143\text{ kips/pair of bolts}
\end{aligned}
$$

(from *Spec*. Eq. J3-6c)

The available bolt tearout strength for a pair of interior bolts is:

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(143\text{ kips})$ $= 107\text{ kips/pair of bolts}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{143\text{ kips}}{2.00}$ $= 71.5\text{ kips/pair of bolts}$

Bolt shear controls over tearout and bearing strength for the interior bolts in the plate.

Shear Strength of Bolted Connection

LRFD	ASD
$\phi R_n = (1\text{ row})(40.8\text{ kips/pair of bolts})$ $+(4\text{ rows})(48.7\text{ kips/pair of bolts})$ $= 236\text{ kips} > 75\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = (1\text{ row})(27.2\text{ kips/pair of bolts})$ $+(4\text{ rows})(32.5\text{ kips/pair of bolts})$ $= 157\text{ kips} > 50\text{ kips}$ **o.k.**

Bolt Shear and Tension Interaction

The available strength of the bolts due to the effect of combined tension and shear is determined from AISC *Specification* Section J3.7. The required shear stress is:

$$f_{rv} = \frac{V_r}{nA_b}$$

where

$A_b = 0.601 \text{ in.}^2$ (from AISC *Manual* Table 7-1)
$n = 10 \text{ bolts}$

LRFD	ASD
$f_{rv} = \dfrac{75 \text{ kips}}{10\left(0.601 \text{ in.}^2\right)}$ $= 12.5 \text{ ksi}$	$f_{rv} = \dfrac{50 \text{ kips}}{10\left(0.601 \text{ in.}^2\right)}$ $= 8.32 \text{ ksi}$

The nominal tensile stress modified to include the effects of shear stress is determined from AISC *Specification* Section J3.7 as follows. From AISC *Specification* Table J3.2:

$F_{nt} = 90 \text{ ksi}$
$F_{nv} = 54 \text{ ksi}$

LRFD	ASD
$\phi = 0.75$ $F'_{nt} = 1.3F_{nt} - \dfrac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt}$ (*Spec*. Eq. J3-3a) $= 1.3(90 \text{ ksi}) - \dfrac{90 \text{ ksi}}{0.75(54 \text{ ksi})}(12.5 \text{ ksi}) \le 90 \text{ ksi}$ $= 89.2 \text{ ksi} < 90 \text{ ksi}$ **o.k.**	$\Omega = 2.00$ $F'_{nt} = 1.3F_{nt} - \dfrac{\Omega F_{nt}}{F_{nv}} f_{rv} \le F_{nt}$ (*Spec*. Eq. J3-3b) $= 1.3(90 \text{ ksi}) - \dfrac{2.00(90 \text{ ksi})}{54 \text{ ksi}}(8.32 \text{ ksi}) \le 90 \text{ ksi}$ $= 89.3 \text{ ksi} < 90 \text{ ksi}$ **o.k.**

Using the value of $F'_{nt} = 89.2 \text{ ksi}$ determined for LRFD, the nominal tensile strength of one bolt is:

$$\begin{aligned} r_n &= F'_{nt}A_b \qquad \text{(\textit{Spec}. Eq. J3-2)} \\ &= (89.2 \text{ ksi})\left(0.601 \text{ in.}^2\right) \\ &= 53.6 \text{ kips} \end{aligned}$$

The available tensile strength due to combined tension and shear is:

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(53.6 \text{ kips})$ $= 40.2 \text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{53.6 \text{ kips}}{2.00}$ $= 26.8 \text{ kips/bolt}$

LRFD	ASD
$\phi R_n = n\phi r_n$ $= (10\text{ bolts})(40.2\text{ kips/bolt})$ $= 402\text{ kips} > 60\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = n\dfrac{r_n}{\Omega}$ $= (10\text{ bolts})(26.8\text{ kips/bolt})$ $= 268\text{ kips} > 40\text{ kips}$ **o.k.**

Prying Action

From AISC *Manual* Part 9, the available tensile strength of the bolts in the end-plate taking prying action into account is determined as follows:

$$\begin{aligned} a &= \frac{width\ of\ \ plate - gage}{2} \\ &= \frac{8\tfrac{1}{2}\text{ in.} - 5\tfrac{1}{2}\text{ in.}}{2} \\ &= 1.50\text{ in.} \end{aligned}$$

Note: If a at the supporting element is smaller than a = 1.50 in., use the smaller a in the preceding calculations.

$$\begin{aligned} b &= \frac{\text{gage} - t_w}{2} \\ &= \frac{5\tfrac{1}{2}\text{ in.} - 0.355\text{ in.}}{2} \\ &= 2.57\text{ in.} \end{aligned}$$

$$\begin{aligned} a' &= \left(a + \frac{d_b}{2}\right) \le \left(1.25b + \frac{d_b}{2}\right) \\ &= 1.50\text{ in.} + \frac{\tfrac{7}{8}\text{ in.}}{2} \le 1.25(2.57\text{ in.}) + \frac{\tfrac{7}{8}\text{ in.}}{2} \\ &= 1.94\text{ in.} < 3.65\text{ in.} \\ &= 1.94\text{ in.} \end{aligned}$$ (*Manual* Eq. 9-23)

$$\begin{aligned} b' &= \left(b - \frac{d_b}{2}\right) \\ &= 2.57\text{ in.} - \frac{\tfrac{7}{8}\text{ in.}}{2} \\ &= 2.13\text{ in.} \end{aligned}$$ (*Manual* Eq. 9-18)

$$\begin{aligned} \rho &= \frac{b'}{a'} \\ &= \frac{2.13\text{ in.}}{1.94\text{ in.}} \\ &= 1.10 \end{aligned}$$ (*Manual* Eq. 9-22)

Note that end distances of 1¼ in. are used on the end-plate, so p is the average pitch of the bolts:

$$\begin{aligned} p &= \frac{l}{n} \\ &= \frac{14\tfrac{1}{2}\text{ in.}}{5} \\ &= 2.90\text{ in.} \end{aligned}$$

Check

$p \le s$

$2.90\text{ in.} < 3\text{ in.}$ **o.k.**

$$\begin{aligned} d' &= d_h \\ &= \tfrac{15}{16}\text{ in.} \end{aligned}$$

$$\begin{aligned} \delta &= 1 - \frac{d'}{p} \\ &= 1 - \frac{\tfrac{15}{16}\text{ in.}}{2.90\text{ in.}} \\ &= 0.677 \end{aligned}$$

(*Manual* Eq. 9-20)

From AISC *Manual* Equations 9-26a or 9-26b, the required end-plate thickness to develop the available strength of the bolt without prying action is:

LRFD	ASD
$\phi = 0.90$ $B_c = 40.2\text{ kips/bolt (calculated previously)}$ $t_c = \sqrt{\dfrac{4B_c b'}{\phi p F_u}}$ $= \sqrt{\dfrac{4(40.2\text{ kips/bolt})(2.13\text{ in.})}{0.90(2.90\text{ in.})(58\text{ ksi})}}$ $= 1.50\text{ in.}$	$\Omega = 1.67$ $B_c = 26.8\text{ kips/bolt (calculated previously)}$ $t_c = \sqrt{\dfrac{\Omega 4 B_c b'}{p F_u}}$ $= \sqrt{\dfrac{1.67(4)(26.8\text{ kips/bolt})(2.13\text{ in.})}{(2.90\text{ in.})(58\text{ ksi})}}$ $= 1.51\text{ in.}$

Because the end-plate thickness of ½ in. is less than t_c, using the value of $t_c = 1.51$ in. determined for ASD, calculate the effect of prying action on the bolts.

$$\begin{aligned} \alpha' &= \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] \\ &= \frac{1}{0.677(1+1.10)}\left[\left(\frac{1.51\text{ in.}}{\tfrac{1}{2}\text{ in.}}\right)^2 - 1\right] \\ &= 5.71 \end{aligned}$$

(*Manual* Eq. 9-28)

Because $\alpha' > 1$, the end-plate has insufficient strength to develop the bolt strength, therefore:

$$
\begin{aligned}
Q &= \left(\frac{t}{t_c}\right)^2 (1+\delta) \\
&= \left(\frac{\text{½ in.}}{1.51\text{ in.}}\right)^2 (1+0.677) \\
&= 0.184
\end{aligned}
$$

The available tensile strength of the bolts taking prying action into account is determined from AISC *Manual* Equation 9-27 as follows:

LRFD	ASD
$T_c = B_c Q$ $= (40.2\text{ kips/bolt})(0.186)$ $= 7.48\text{ kips/bolt}$ $\phi R_n = T_c n$ $= (7.48\text{ kips/bolt})(10\text{ bolts})$ $= 74.8\text{ kips} > 60\text{ kips}$ **o.k.**	$T_c = B_c Q$ $= (26.8\text{ kips/bolt})(0.184)$ $= 4.93\text{ kips/bolt}$ $\dfrac{R_n}{\Omega} = T_c n$ $= (4.93\text{ kips/bolt})(10\text{ bolts})$ $= 49.3\text{ kips} > 40\text{ kips}$ **o.k.**

Weld Design

Assume a 3⁄16-in. fillet weld on each side of the beam web, with the weld stopping short of the end of the plate at a distance equal to the weld size.

$$
\begin{aligned}
l_w &= 14\tfrac{1}{2}\text{ in.} - 2(\tfrac{3}{16}\text{ in.}) \\
&= 14.1\text{ in.}
\end{aligned}
$$

LRFD	ASD
$\theta = \tan^{-1}\left(\dfrac{N_u}{V_u}\right)$ $= \tan^{-1}\left(\dfrac{60\text{ kips}}{75\text{ kips}}\right)$ $= 38.7°$	$\theta = \tan^{-1}\left(\dfrac{N_a}{V_a}\right)$ $= \tan^{-1}\left(\dfrac{40\text{ kips}}{50\text{ kips}}\right)$ $= 38.7°$

From AISC *Manual* Table 8-4 for Angle = 30° (which will lead to a conservative result):

Special Case: $k = a = 0$

$C = 4.37$

The required weld size is determined from AISC *Manual* Equation 8-21 as follows:

LRFD	ASD
$\phi = 0.75$ $D_{min} = \dfrac{R_u}{\phi C C_1 l_w}$ $= \dfrac{96.0 \text{ kips}}{0.75(4.37)(1.0)(14.1 \text{ in.})}$ $= 2.08 \text{ sixteenths}$	$\Omega = 2.00$ $D_{min} = \dfrac{\Omega R_a}{C C_1 l_w}$ $= \dfrac{2.00(64.0 \text{ kips})}{(4.37)(1.0)(14.1 \text{ in.})}$ $= 2.08 \text{ sixteenths}$

Use a 3⁄16-in. fillet weld (minimum size from AISC *Specification* Table J2.4).

Beam Web Strength at Fillet Weld

The minimum beam web thickness required to match the shear rupture strength of the connecting element to that of the base metal is:

$$\begin{aligned} t_{min} &= \frac{6.19 D_{min}}{F_u} \\ &= \frac{6.19(2.08)}{65 \text{ ksi}} \\ &= 0.198 \text{ in.} < 0.355 \text{ in.} \quad \textbf{o.k.} \end{aligned}$$

(from *Manual* Eq. 9-3)

Shear Strength of the Plate

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the plate is determined as follows:

$$\begin{aligned} A_{gv} &= 2lt \\ &= (2)(14\tfrac{1}{2} \text{ in.})(\tfrac{1}{2} \text{ in.}) \\ &= 14.5 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60 F_y A_{gv} \\ &= 0.60(36 \text{ ksi})(14.5 \text{ in.}^2) \\ &= 313 \text{ kips} \end{aligned}$$

(*Spec*. Eq. J4-3)

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(313 \text{ kips})$ $= 313 \text{ kips} > 96.0 \text{ kips} \quad \textbf{o.k.}$	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{313 \text{ kips}}{1.50}$ $= 209 \text{ kips} > 64.0 \text{ kips} \quad \textbf{o.k.}$

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the plate is determined as follows:

$$\begin{aligned} A_{nv} &= 2\left[l - n\left(d_h + \tfrac{1}{16} \text{ in.}\right)\right]t \\ &= 2\left[14\tfrac{1}{2} \text{ in.} - 5\left(\tfrac{15}{16} \text{ in.} + \tfrac{1}{16} \text{ in.}\right)\right]\left(\tfrac{1}{2} \text{ in.}\right) \\ &= 9.50 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned}R_n &= 0.60F_uA_{nv} \\ &= 0.60(58\text{ ksi})(9.50\text{ in.}^2) \\ &= 331\text{ kips}\end{aligned}$$ (*Spec*. Eq. J4-4)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(331\text{ kips})$ $= 248\text{ kips} > 96.0\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{331\text{ kips}}{2.00}$ $= 166\text{ kips} > 64.0\text{ kips}$ **o.k.**

Block Shear Rupture Strength of the Plate

The nominal strength for the limit state of block shear rupture of the plate assuming an L-shaped tearout relative to shear load, is determined as follows. The tearout pattern is shown in Figure II.A-11B-2.

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.60F_yA_{gv} + U_{bs}F_uA_{nt}$$ (*Spec*. Eq. J4-5)

where

$$\begin{aligned}l_{eh} &= \frac{b - gage}{2} \\ &= \frac{8\tfrac{1}{2}\text{ in.} - 5\tfrac{1}{2}\text{ in.}}{2} \\ &= 1.50\text{ in.}\end{aligned}$$

$$\begin{aligned}A_{gv} &= (2)\left[l_{ev} + (n-1)s\right](t) \\ &= (2)\left[1\tfrac{1}{4}\text{ in.} + (5-1)(3.00\text{ in.})\right](\tfrac{1}{2}\text{ in.}) \\ &= 13.3\text{ in.}^2\end{aligned}$$

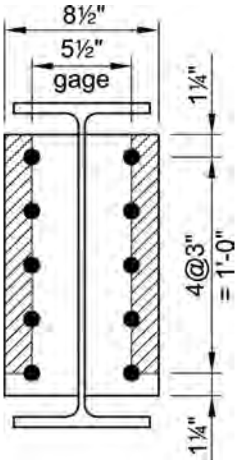

Fig. II.A-11B-2. Block shear rupture of end-plate.

$$\begin{aligned}A_{nv} &= A_{gv} - (2)(n - 0.5)(d_h + \tfrac{1}{16}\text{ in.})(t)\\ &= 13.3\text{ in.}^2 - (2)(5 - 0.5)(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})(\tfrac{1}{2}\text{ in.})\\ &= 8.80\text{ in.}^2\end{aligned}$$

$$\begin{aligned}A_{nt} &= (2)\left[l_{eh} - 0.5(d_h + \tfrac{1}{16}\text{ in.})\right](t)\\ &= (2)\left[1.50\text{ in.} - 0.5(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})\right](\tfrac{1}{2}\text{ in.})\\ &= 1.00\text{ in.}^2\end{aligned}$$

$$U_{bs} = 1.0$$

and

$$\begin{aligned}R_n &= 0.60(58\text{ ksi})(8.80\text{ in.}^2) + 1.0(58\text{ ksi})(1.00\text{ in.}^2) \le 0.60(36\text{ ksi})(13.3\text{ in.}^2) + 1.0(58\text{ ksi})(1.00\text{ in.}^2)\\ &= 364\text{ kips} > 345\text{ kips}\end{aligned}$$

Therefore:

$$R_n = 345\text{ kips}$$

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75(345\text{ kips})$ $= 259\text{ kips} > 75\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{345\text{ kips}}{2.00}$ $= 173\text{ kips} > 50\text{ kips}$ **o.k.**

Shear Strength of Beam

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the beam is determined as follows:

$$\begin{aligned}A_{gv} &= dt_w\\ &= (18.0\text{ in.})(0.355\text{ in.})\\ &= 6.39\text{ in.}^2\end{aligned}$$

$$\begin{aligned}R_n &= 0.60F_yA_{gv}\\ &= 0.60(50\text{ ksi})(6.39\text{ in.}^2)\\ &= 192\text{ kips}\end{aligned}$$ (*Spec*. Eq. J4-3)

LRFD	ASD
$\phi = 1.00$	$\Omega = 1.50$
$\phi R_n = 1.00(192\text{ kips})$ $= 192\text{ kips} > 75\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{192\text{ kips}}{1.50}$ $= 128\text{ kips} > 50\text{ kips}$ **o.k.**

The limit state of shear rupture of the beam web does not apply in this example because the beam is uncoped.

Tensile Strength of Beam

From AISC *Specification* Section J4.1, the available tensile yield strength of the beam is determined as follows:

$$\begin{aligned} R_n &= F_y A_g \qquad \text{(Spec. Eq. J4-1)} \\ &= (50\text{ ksi})(14.7\text{ in.}^2) \\ &= 735\text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(735\text{ kips})$ $= 662\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{735\text{ kips}}{1.67}$ $= 440\text{ kips} > 40\text{ kips}$ **o.k.**

From AISC *Specification* Section J4.1, determine the available tensile rupture strength of the beam. The effective net area is $A_e = A_n U$ from AISC *Specification* Section D3, where U is determined from AISC *Specification* Table D3.1, Case 3.

$$U = 1.0$$

$$\begin{aligned} A_n &= \text{area of the directly connected elements} \\ &= l_w t_w \\ &= (14.1\text{ in.})(0.355\text{ in.}) \\ &= 5.01\text{ in.}^2 \end{aligned}$$

The available tensile rupture strength is:

$$\begin{aligned} R_n &= F_u A_e \qquad \text{(Spec. Eq. J4-2)} \\ &= F_u A_n U \\ &= (65\text{ ksi})(5.01\text{ in.}^2)(1.0) \\ &= 326\text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(326\text{ kips})$ $= 245\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{326\text{ kips}}{2.00}$ $= 163\text{ kips} > 40\text{ kips}$ **o.k.**

Conclusion

The connection is found to be adequate as given for the applied loads.

EXAMPLE II.A-11C SHEAR END-PLATE CONNECTION—STRUCTURAL INTEGRITY CHECK

Given:

Verify the shear end-plate connection from Example II.A-11B for the structural integrity provisions of AISC *Specification* Section B3.9. The ASTM A992 W18×50 beam is bracing a column and the connection geometry is shown in Figure II.A-11C-1. Note that these checks are necessary when design for structural integrity is required by the applicable building code.

Use 70-ksi electrodes and ASTM A36 plate.

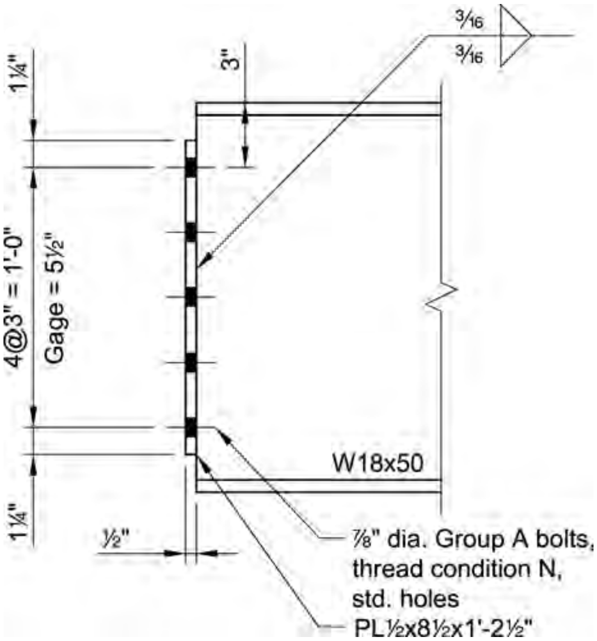

Fig. II.A-11C-1. Connection geometry for Example II.A-11C.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Plate
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W18x50
$t_w = 0.355$ in.

From Example II.A-11B, the required shear strength is:

LRFD	ASD
$V_u = 75 \text{ kips}$	$V_a = 50 \text{ kips}$

From AISC *Specification* Section B3.9, the required axial tensile strength is:

LRFD	ASD
$T_u = \frac{2}{3}V_u \geq 10 \text{ kips}$ $= \frac{2}{3}(75 \text{ kips}) > 10 \text{ kips}$ $= 50 \text{ kips} > 10 \text{ kips}$ $= 50 \text{ kips}$	$T_a = V_a \geq 10 \text{ kips}$ $= 50 \text{ kips} > 10 \text{ kips}$ $= 50 \text{ kips}$

From AISC *Specification* Section B3.9, these requirements are evaluated independently from other strength requirements.

Bolt Tension

From AISC *Specification* Section J3.6, the nominal bolt tensile strength is:

F_{nt} = 90 ksi, from AISC *Specification* Table J3.2

$$T_n = nF_{nt}A_b$$ (from *Spec*. Eq. J3-1)

$$= (10 \text{ bolts})(90 \text{ ksi})\left(0.601 \text{ in.}^2\right)$$
$$= 541 \text{ kips}$$

Angle Bending and Prying Action

From AISC *Manual* Part 9, the nominal strength of the end-plate accounting for prying action is determined as follows:

$$a = \frac{width\ of\ \ plate - gage}{2}$$
$$= \frac{8\text{½ in.} - 5\text{½ in.}}{2}$$
$$= 1.50 \text{ in.}$$

$$b = \frac{gage - t_w}{2}$$
$$= \frac{5\text{½ in.} - 0.355 \text{ in.}}{2}$$
$$= 2.57 \text{ in.}$$

$$a' = \left(a + \frac{d_b}{2}\right) \leq \left(1.25b + \frac{d_b}{2}\right) \qquad (\textit{Manual}\text{ Eq. 9-23})$$

$$= 1.50\text{ in.} + \frac{\tfrac{7}{8}\text{ in.}}{2} \leq 1.25(2.57\text{ in.}) + \frac{\tfrac{7}{8}\text{ in.}}{2}$$

$$= 1.94\text{ in.} < 3.65\text{ in.}$$

$$= 1.94\text{ in.}$$

$$b' = b - \frac{d_b}{2} \qquad (\textit{Manual}\text{ Eq. 9-18})$$

$$= 2.57\text{ in.} - \frac{\tfrac{7}{8}\text{ in.}}{2}$$

$$= 2.13\text{ in.}$$

$$\rho = \frac{b'}{a'} \qquad (\textit{Manual}\text{ Eq. 9-22})$$

$$= \frac{2.13\text{ in.}}{1.94\text{ in.}}$$

$$= 1.10$$

Note that end distances of 1¼ in. are used on the end-plate, so p is the average pitch of the bolts:

$$p = \frac{l}{n}$$

$$= \frac{14\tfrac{1}{2}\text{ in.}}{5}$$

$$= 2.90\text{ in.}$$

Check

$p \leq s = 3.00\text{ in.}$ **o.k.**

$$d' = d_h$$

$$= \tfrac{15}{16}\text{ in.}$$

$$\delta = 1 - \frac{d'}{p} \qquad (\textit{Manual}\text{ Eq. 9-20})$$

$$= 1 - \frac{\tfrac{15}{16}\text{ in.}}{2.90\text{ in.}}$$

$$= 0.677$$

$$B_n = F_{nt} A_b$$

$$= (90\text{ ksi})\left(0.601\text{ in.}^2\right)$$

$$= 54.1\text{ kips/bolt}$$

$$\begin{aligned} t_c &= \sqrt{\frac{4B_n b'}{pF_u}} \\ &= \sqrt{\frac{4(54.1\text{ kips/bolt})(2.13\text{ in.})}{(2.90\text{ in.})(58\text{ ksi})}} \\ &= 1.66\text{ in.} \end{aligned}$$ (from *Manual* Eq. 9-26)

$$\begin{aligned} \alpha' &= \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] \\ &= \frac{1}{0.677(1+1.10)}\left[\left(\frac{1.66\text{ in.}}{\text{½ in.}}\right)^2 - 1\right] \\ &= 7.05 \end{aligned}$$ (*Manual* Eq. 9-28)

Because $\alpha' > 1$, the end-plate has insufficient strength to develop the bolt strength, therefore:

$$\begin{aligned} Q &= \left(\frac{t}{t_c}\right)^2 (1+\delta) \\ &= \left(\frac{\text{½ in.}}{1.66\text{ in.}}\right)^2 (1+0.677) \\ &= 0.152 \end{aligned}$$

$$\begin{aligned} T_n &= B_n Q \\ &= (10\text{ bolts})(54.1\text{ kips/bolt})(0.152) \\ &= 82.2\text{ kips} \end{aligned}$$ (from *Manual* Eq. 9-27)

Weld Strength

From AISC *Specification* Section J2.4, the nominal tensile strength of the weld is determined as follows:

$$\begin{aligned} F_{nw} &= 0.60F_{EXX}\left(1.0 + 0.50\sin^{1.5}\theta\right) \\ &= 0.60(70\text{ ksi})\left(1.0 + 0.50\sin^{1.5} 90°\right) \\ &= 63.0\text{ ksi} \end{aligned}$$ (*Spec*. Eq. J2-5)

The weld length accounts for termination equal to the weld size.

$$\begin{aligned} l_w &= l - 2w \\ &= 14\tfrac{1}{2}\text{ in.} - 2\left(\tfrac{3}{16}\text{ in.}\right) \\ &= 14.1\text{ in.} \end{aligned}$$

The throat dimension is used to calculate the effective area of the fillet weld.

$$
\begin{aligned}
A_{we} &= \frac{w}{\sqrt{2}} l_w \left(2 \text{ welds}\right) \\
&= \frac{\text{3/16 in.}}{\sqrt{2}} \left(14.1 \text{ in.}\right)\left(2 \text{ welds}\right) \\
&= 3.74 \text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
T_n &= F_{nw} A_{we} \\
&= \left(63.0 \text{ ksi}\right)\left(3.74 \text{ in.}^2\right) \\
&= 236 \text{ kips}
\end{aligned}
$$

(from *Spec*. Eq. J2-4)

Tensile Strength of Beam Web at the Weld

From AISC *Specification* Section J4.1, the nominal tensile strength of the beam web at the weld is:

$$
\begin{aligned}
A_e &= l_w t_w \\
&= \left(14.1 \text{ in.}\right)\left(0.355 \text{ in.}\right) \\
&= 5.01 \text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
T_n &= F_u A_e \\
&= \left(65 \text{ ksi}\right)\left(5.01 \text{ in.}^2\right) \\
&= 326 \text{ kips}
\end{aligned}
$$

(*Spec*. Eq. J4-2)

Nominal Tensile Strength

The controlling nominal tensile strength, T_n, is the least of those previously calculated:

$$
\begin{aligned}
T_n &= \min\left\{541 \text{ kips},\ 82.2 \text{ kips},\ 236 \text{ kips},\ 326 \text{ kips}\right\} \\
&= 82.2 \text{ kips}
\end{aligned}
$$

LRFD	ASD
$T_n = 82.2 \text{ kips} > 50 \text{ kips}$ **o.k.**	$T_n = 82.2 \text{ kips} > 50 \text{ kips}$ **o.k.**

Column Bracing

From AISC *Specification* Section B3.9(c), the minimum axial tension strength for the connection of a member bracing a column is equal to 1% of two-thirds of the required column axial strength for LRFD and equal to 1% of the required column axial for ASD. These requirements are evaluated independently from other strength requirements.

The maximum column axial force this connection is able to brace is determined as follows:

LRFD	ASD
$T_n \geq 0.01\left(\frac{2}{3}P_u\right)$	$T_n \geq 0.01P_a$

LRFD	ASD
Solving for the column axial force: $P_u \le 100\left(\frac{3}{2}T_n\right)$ $= 100\left(\frac{3}{2}\right)(82.2\text{ kips})$ $= 12{,}300\text{ kips}$	Solving for the column axial force: $P_a \le 100T_n$ $= 100(82.2\text{ kips})$ $= 8{,}220\text{ kips}$

As long as the required column axial strength is less than or equal to P_u = 12,300 kips or P_a = 8,220 kips, this connection is an adequate column brace.

EXAMPLE II.A-12A ALL-BOLTED UNSTIFFENED SEATED CONNECTION (BEAM-TO-COLUMN WEB)

Given:

Verify the all-bolted unstiffened seated connection between an ASTM A992 W16×50 beam and an ASTM A992 W14×90 column web, as shown in Figure II.A-12A-1, to support the following end reactions:

R_D = 9 kips
R_L = 27.5 kips

Use ASTM A36 angles.

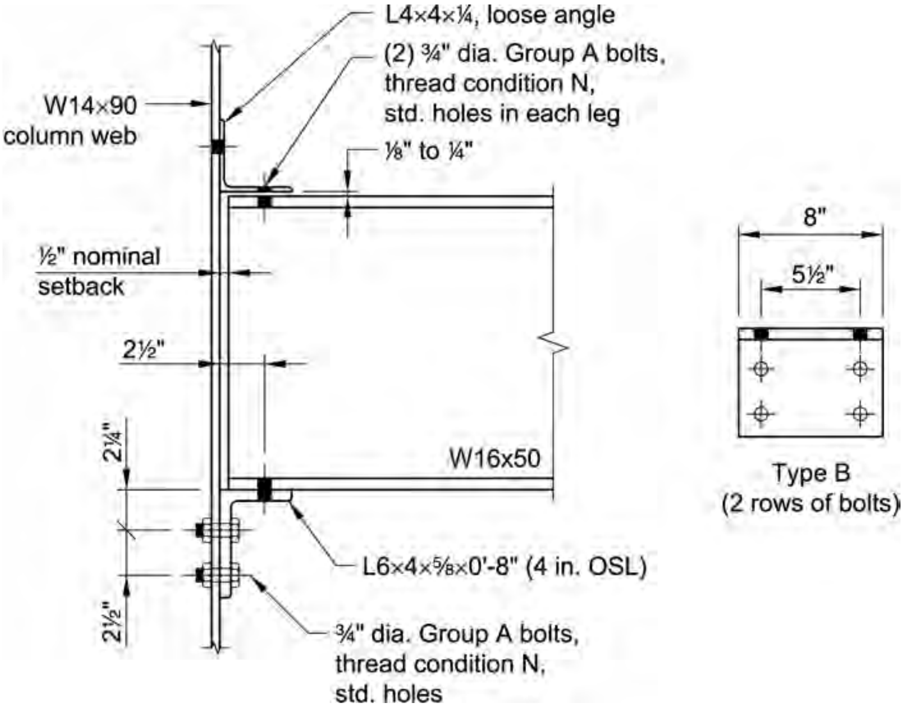

Fig. II.A-12A-1. Connection geometry for Example II.A-12A-1.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam and column
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Angles
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W16×50
t_w = 0.380 in.
d = 16.3 in.
b_f = 7.07 in.
t_f = 0.630 in.
k_{des} = 1.03 in.

Column
W14×90
t_w = 0.440 in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(9\text{ kips}) + 1.6(27.5\text{ kips})$ $= 54.8\text{ kips}$	$R_a = 9\text{ kips} + 27.5\text{ kips}$ $= 36.5\text{ kips}$

Minimum Bearing Length

From AISC *Manual* Part 10, the minimum required bearing length, $l_{b\ min}$, is the length of bearing required for the limit states of web local yielding and web local crippling on the beam, but not less than k_{des}.

Using AISC *Manual* Equations 9-46a or 9-46b and AISC *Manual* Table 9-4, the minimum required bearing length for web local yielding is:

LRFD	ASD
$l_{b\ min} = \dfrac{R_u - \phi R_1}{\phi R_2} \geq k_{des}$ $= \dfrac{54.8\text{ kips} - 48.9\text{ kips}}{19.0\text{ kip/in.}} \geq 1.03\text{ in.}$ $= 0.311\text{ in.} < 1.03\text{ in.}$ Therefore, $l_{b\ min} = k_{des} = 1.03\text{ in.}$	$l_{b\ min} = \dfrac{R_a - R_1/\Omega}{R_2/\Omega} \geq k_{des}$ $= \dfrac{36.5\text{ kips} - 32.6\text{ kips}}{12.7\text{ kip/in.}} \geq 1.03\text{ in.}$ $= 0.307\text{ in.} < 1.03\text{ in.}$ Therefore, $l_{b\ min} = k_{des} = 1.03\text{ in.}$

For web local crippling, the maximum bearing length-to-depth ratio is determined as follows (including ¼-in. tolerance to account for possible beam underrun):

$$\left(\frac{l_b}{d}\right)_{max} = \frac{3.25\text{ in.}}{16.3\text{ in.}}$$
$$= 0.199 < 0.2$$

Using AISC *Manual* Equations 9-48a or 9-48b and AISC *Manual* Table 9-4, when $\dfrac{l_b}{d} \leq 0.2$:

LRFD	ASD
$l_{b\ min} = \dfrac{R_u - \phi R_3}{\phi R_4}$ $= \dfrac{54.8 \text{ kips} - 67.2 \text{ kips}}{5.79 \text{ kip/in.}}$ This results in a negative quantity; therefore, $l_{b\ min} = k_{des} = 1.03 \text{ in.}$	$l_{b\ min} = \dfrac{R_a - R_3 / \Omega}{R_4 / \Omega}$ $= \dfrac{36.5 \text{ kips} - 44.8 \text{ kips}}{3.86 \text{ kip/in.}}$ This results in a negative quantity; therefore, $l_{b\ min} = k_{des} = 1.03 \text{ in.}$

Connection Selection

AISC *Manual* Table 10-5 includes checks for the limit states of shear yielding and flexural yielding of the outstanding angle leg.

For an 8-in. angle length with a ⅝-in. thickness, a 3½-in. minimum outstanding leg, and conservatively using $l_{b,\,req} = 1\frac{1}{16}$ in., from AISC *Manual* Table 10-5:

LRFD	ASD
$\phi R_n = 90.0 \text{ kips} > 54.8 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 59.9 \text{ kips} > 36.5 \text{ kips}$ **o.k.**

The L6×4×⅝ (4-in. OSL), 8-in. long with 5½-in. bolt gage, Connection Type B (four bolts), is acceptable.

From the bottom portion of AISC *Manual* Table 10-5 for L6, with ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N), the available shear strength is:

LRFD	ASD
$\phi R_n = 71.6 \text{ kips} > 54.8 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 47.7 \text{ kips} > 36.5 \text{ kips}$ **o.k.**

Bolt Bearing and Tearout on the Angle

Due to the presence and location of the bolts in the outstanding leg of the angle, tearout does not control. The nominal bearing strength of the angles is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration:

$$\begin{aligned} R_n &= (4 \text{ bolts})2.4dtF_u \\ &= (4 \text{ bolts})(2.4)(\tfrac{3}{4} \text{ in.})(\tfrac{5}{8} \text{ in.})(58 \text{ ksi}) \\ &= 261 \text{ kips} \end{aligned}$$

(from *Spec.* Eq. J3-6a)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(261 \text{ kips})$ $= 196 \text{ kips} > 54.8 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{261 \text{ kips}}{2.00}$ $= 131 \text{ kips} > 36.5 \text{ kips}$ **o.k.**

Note that the effective strength of the bolt group is controlled by bolt shear.

Bolt Bearing on the Column

The nominal bearing strength of the column web determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration, is:

$$R_n = (4\text{ bolts})2.4dt_wF_u \qquad \text{(from } Spec. \text{ Eq. J3-6a)}$$

$$= (4\text{ bolts})(2.4)(\tfrac{3}{4}\text{ in.})(0.440\text{ in.})(65\text{ ksi})$$

$$= 206\text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(206\text{ kips})$ $= 155\text{ kips} > 54.8\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{206\text{ kips}}{2.00}$ $= 103\text{ kips} > 36.5\text{ kips}$ **o.k.**

Top Angle and Bolts

As discussed in AISC *Manual* Part 10, use an L4×4×¼ with two ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) through each leg.

Conclusion

The connection design shown in Figure II.A-12A-1 is acceptable.

EXAMPLE II.A-12B ALL-BOLTED UNSTIFFENED SEATED CONNECTION—STRUCTURAL INTEGRITY CHECK

Given:

Verify the all-bolted unstiffened seated connection from Example II.A-12A, as shown in Figure II.A-12B-1, for the structural integrity provisions of AISC *Specification* Section B3.9. The connection is verified as a beam and girder end connection and as an end connection of a member bracing a column. Note that these checks are necessary when design for structural integrity is required by the applicable building code.

The beam is an ASTM A992 W16×50 and the angles are ASTM A36 material.

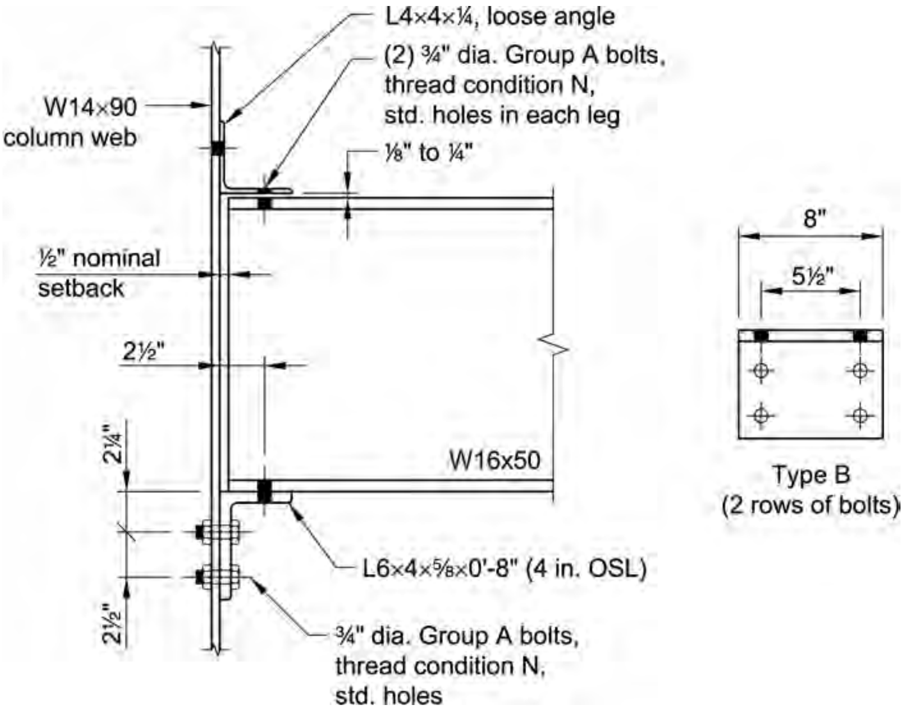

Fig. II.A-12B-1. Connection geometry for Example II.A-12B.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Angle
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W16x50
$b_f = 7.07$ in.
$t_f = 0.630$ in.

From AISC *Specification* Table J3.3, the hole diameter for ¾-in.-diameter bolts with standard holes is:

$d_h = {}^{13}\!/_{16}$ in.

From Example II.A-12A, the required shear strength is:

LRFD	ASD
$V_u = 54.8 \text{ kips}$	$V_a = 36.5 \text{ kips}$

From AISC *Specification* Section B3.9(b), the required axial tensile strength is:

LRFD	ASD
$T_u = \frac{2}{3}V_u \geq 10 \text{ kips}$ $= \frac{2}{3}(54.8 \text{ kips}) > 10 \text{ kips}$ $= 36.5 \text{ kips}$	$T_a = V_a \geq 10 \text{ kips}$ $= 36.5 \text{ kips} > 10 \text{ kips}$ $= 36.5 \text{ kips}$

From AISC *Specification* Section B3.9, these strength requirements are evaluated independently from other strength requirements.

Bolt Shear

Bolt shear is checked for the outstanding leg of the seat angle. From AISC *Specification* Section J3.6, the nominal bolt shear strength is:

$F_{nv} = 54$ ksi, from AISC *Specification* Table J3.2

$$\begin{aligned} T_n &= nF_{nv}A_b \qquad \text{(from } \textit{Spec.} \text{ Eq. J3-1)} \\ &= (2 \text{ bolts})(54 \text{ ksi})(0.442 \text{ in.}^2) \\ &= 47.7 \text{ kips} \end{aligned}$$

Bolt Tension

Bolt tension is checked for the top row of bolts on the support leg of the seat angle. From AISC *Specification* Section J3.6, the nominal bolt tensile strength is:

$F_{nt} = 90$ ksi, from AISC *Specification* Table J3.2

$$\begin{aligned} T_n &= nF_{nt}A_b \qquad \text{(from } \textit{Spec.} \text{ Eq. J3-1)} \\ &= (2 \text{ bolts})(90 \text{ ksi})(0.442 \text{ in.}^2) \\ &= 79.6 \text{ kips} \end{aligned}$$

Bolt Bearing and Tearout

Bolt bearing and tearout is checked for the outstanding leg of the seat angle. From AISC *Specification* Section B3.9, for the purpose of satisfying structural integrity requirements, inelastic deformations of the connection are permitted; therefore, AISC *Specification* Equations J3-6b and J3-6d are used to determine the nominal bearing and tearout strength. By inspection, bolt bearing and tearout will control for the angle.

For bolt bearing on the angle:

$$\begin{aligned} T_n &= n3.0dtF_u \\ &= (2\text{ bolts})(3.0)(\tfrac{3}{4}\text{ in.})(\tfrac{5}{8}\text{ in.})(58\text{ ksi}) \\ &= 163\text{ kips} \end{aligned}$$

(from *Spec.* Eq. J3-6b)

For bolt tearout on the angle:

$$\begin{aligned} l_c &= leg - 2\tfrac{1}{2}\text{ in.} - 0.5d_h \\ &= 4.00\text{ in.} - 2\tfrac{1}{2}\text{ in.} - 0.5(\tfrac{13}{16}\text{ in.}) \\ &= 1.09\text{ in.} \end{aligned}$$

$$\begin{aligned} T_n &= n1.5l_ctF_u \\ &= (2\text{ bolts})(1.5)(1.09\text{ in.})(\tfrac{5}{8}\text{ in.})(58\text{ ksi}) \\ &= 119\text{ kips} \end{aligned}$$

(from *Spec.* Eq. J3-6d)

Angle Bending and Prying Action

From AISC *Manual* Part 9, the nominal strength of the angle accounting for prying action is determined as follows:

$$\begin{aligned} b &= 2\tfrac{1}{4}\text{ in.} - \frac{\tfrac{5}{8}\text{ in.}}{2} \\ &= 1.94\text{ in.} \end{aligned}$$

$$\begin{aligned} a &= \min\{2.50\text{ in.}, 1.25b\} \\ &= \min\{2.50\text{ in.}, 1.25(1.94\text{ in.})\} \\ &= 2.43\text{ in.} \end{aligned}$$

$$\begin{aligned} b' &= \left(b - \frac{d_b}{2}\right) \\ &= 1.94\text{ in.} - \frac{\tfrac{3}{4}\text{ in.}}{2} \\ &= 1.57\text{ in.} \end{aligned}$$

(*Manual* Eq. 9-18)

$$\begin{aligned} a' &= a + \frac{d_b}{2} \\ &= 2.43\text{ in.} + \frac{\tfrac{3}{4}\text{ in.}}{2} \\ &= 2.81\text{ in.} \end{aligned}$$

(from *Manual* Eq. 9-23)

$$\rho = \frac{b'}{a'} \qquad \text{(}\textit{Manual}\text{ Eq. 9-22)}$$
$$= \frac{1.57\text{ in.}}{2.81\text{ in.}}$$
$$= 0.559$$

Note that end distances of 1¼ in. are used on the angles, so p is the average pitch of the bolts:

$$p = \frac{l}{n}$$
$$= \frac{8.00\text{ in.}}{2}$$
$$= 4.00\text{ in.}$$

Check

$p \le s = 5\tfrac{1}{2}\text{ in.}$ **o.k.**

$$d' = d_h$$
$$= \tfrac{13}{16}\text{ in.}$$

$$\delta = 1 - \frac{d'}{p} \qquad \text{(}\textit{Manual}\text{ Eq. 9-20)}$$
$$= 1 - \frac{\tfrac{13}{16}\text{ in.}}{4.00\text{ in.}}$$
$$= 0.797$$

$$B_n = F_{nt}A_b$$
$$= (90\text{ ksi})(0.442\text{ in.}^2)$$
$$= 39.8\text{ kips/bolt}$$

$$t_c = \sqrt{\frac{4B_nb'}{pF_u}} \qquad \text{(from }\textit{Manual}\text{ Eq. 9-26)}$$
$$= \sqrt{\frac{4(39.8\text{ kips/bolt})(1.57\text{ in.})}{(4.00\text{ in.})(58\text{ ksi})}}$$
$$= 1.04\text{ in.}$$

$$\alpha' = \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] \qquad \text{(}\textit{Manual}\text{ Eq. 9-28)}$$
$$= \frac{1}{0.797(1+0.559)}\left[\left(\frac{1.04\text{ in.}}{\tfrac{5}{8}\text{ in.}}\right)^2 - 1\right]$$
$$= 1.42$$

Because $\alpha' > 1$, the angle has insufficient strength to develop the bolt strength, therefore:

$$
\begin{aligned}
Q &= \left(\frac{t}{t_c}\right)^2 (1+\delta) \\
&= \left(\frac{\text{5/8 in.}}{\text{1.04 in.}}\right)^2 (1+0.797) \\
&= 0.649
\end{aligned}
$$

$$
\begin{aligned}
T_n &= B_n Q \qquad \text{(from } \textit{Manual} \text{ Eq. 9-27)} \\
&= (2 \text{ bolts})(39.8 \text{ kips/bolt})(0.649) \\
&= 51.7 \text{ kips}
\end{aligned}
$$

Block Shear Rupture

By comparison of the seat angle length and flange width, block shear rupture of the beam flange will control. The block shear rupture failure path is shown in Figure II.A-12B-2. From AISC *Specification* Section J4.3, the available block shear rupture strength of the beam flange is determined as follows (account for a possible ¼-in. beam underrun):

$$T_n = 0.60F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.60F_y A_{gv} + U_{bs} F_u A_{nt} \qquad \text{(from } \textit{Spec.} \text{ Eq. J4-5)}$$

where

$$
\begin{aligned}
A_{gv} &= (2) l_e t_f \\
&= (2)(1\tfrac{3}{4} \text{ in.})(0.630 \text{ in.}) \\
&= 2.21 \text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
A_{nv} &= (2)\left[l_e - 0.5\left(d_h + \tfrac{1}{16} \text{ in.}\right)\right] t_f \\
&= (2)\left[1\tfrac{3}{4} \text{ in.} - 0.5\left(\tfrac{13}{16} \text{ in.} + \tfrac{1}{16} \text{ in.}\right)\right](0.630 \text{ in.}) \\
&= 1.65 \text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
A_{nt} &= (2)\left[\frac{b_f - gage}{2} - 0.5\left(d_h + \tfrac{1}{16} \text{ in.}\right)\right] t_f \\
&= (2)\left[\frac{7.07 \text{ in.} - 5\tfrac{1}{2} \text{ in.}}{2} - 0.5\left(\tfrac{13}{16} \text{ in.} + \tfrac{1}{16} \text{ in.}\right)\right](0.630 \text{ in.}) \\
&= 0.438 \text{ in.}^2
\end{aligned}
$$

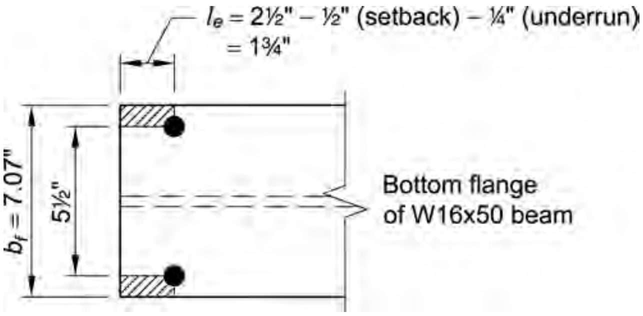

Fig. II.A-12B-2. Beam flange block shear rupture.

$$U_{bs} = 1.0$$

and

$$\begin{aligned} T_n &= 0.60(65\text{ ksi})(1.65\text{ in.}^2) + 1.0(65\text{ ksi})(0.438\text{ in.}^2) \le 0.60(50\text{ ksi})(2.21\text{ in.}^2) + 1.0(65\text{ ksi})(0.438\text{ in.}^2) \\ &= 92.8\text{ kips} < 94.8\text{ kips} \\ &= 92.8\text{ kips} \end{aligned}$$

Nominal Tensile Strength

The controlling tensile strength, T_n, is the least of those previously calculated:

$$\begin{aligned} T_n &= \min\{47.7\text{ kips},\ 79.6\text{ kips},\ 163\text{ kips},\ 119\text{ kips},\ 51.7\text{ kips},\ 92.8\text{ kips}\} \\ &= 47.7\text{ kips} \end{aligned}$$

LRFD	ASD
$T_n = 47.7\text{ kips} > 36.5\text{ kips}$ **o.k.**	$T_n = 47.7\text{ kips} > 36.5\text{ kips}$ **o.k.**

Column Bracing

From AISC *Specification* Section B3.9(c), the minimum axial tension strength for the connection of a member bracing a column is equal to 1% of two-thirds of the required column axial strength for LRFD and equal to 1% of the required column axial for ASD. These requirements are evaluated independently from other strength requirements.

The maximum column axial force this connection is able to brace is determined as follows,

LRFD	ASD
$T_n \ge 0.01\left(\frac{2}{3}P_u\right)$ Solving for the column axial force: $P_u \le 100\left(\frac{3}{2}T_n\right)$ $= 100\left(\frac{3}{2}\right)(47.7\text{ kips})$ $= 7{,}160\text{ kips}$	$T_n \ge 0.01P_a$ Solving for the column axial force: $P_a \le 100T_n$ $= 100(47.7\text{ kips})$ $= 4{,}770\text{ kips}$

As long as the required column axial strength is less than P_u = 7,160 kips or P_a = 4,770 kips, this connection is an adequate column brace.

EXAMPLE II.A-13 BOLTED/WELDED UNSTIFFENED SEATED CONNECTION (BEAM-TO-COLUMN FLANGE)

Given:

Verify the unstiffened seated connection between an ASTM A992 W21×62 beam and an ASTM A992 W14×61 column flange, as shown in Figure II.A-13-1, to support the following beam end reactions:

R_D = 9 kips
R_L = 27.5 kips

Use ASTM A36 angles and 70-ksi weld electrodes.

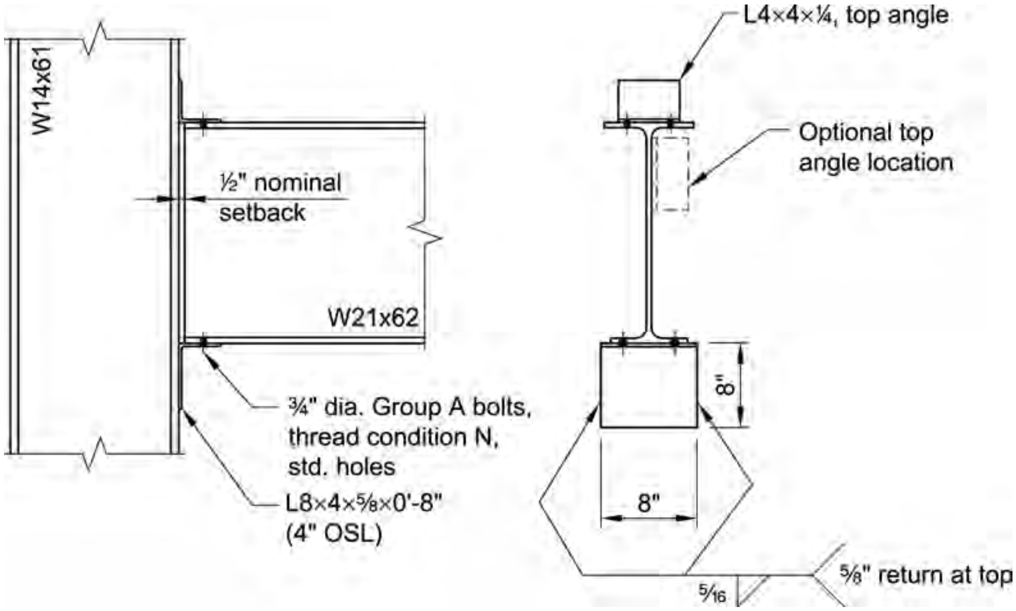

Fig. II.A-13-1. Connection geometry for Example II.A-13.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam and column
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Angles
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W21×62
t_w = 0.400 in.
d = 21.0 in.
b_f = 8.24 in.
t_f = 0.615 in.
k_{des}= 1.12 in.

Column
W14×61
t_f = 0.645 in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(9 \text{ kips}) + 1.6(27.5 \text{ kips})$ $= 54.8 \text{ kips}$	$R_a = 9 \text{ kips} + 27.5 \text{ kips}$ $= 36.5 \text{ kips}$

Minimum Bearing Length

From AISC *Manual* Part 10, the minimum required bearing length, $l_{b\,min}$, is the length of bearing required for the limit states of web local yielding and web local crippling on the beam, but not less than k_{des}.

Using AISC *Manual* Equations 9-46a or 9-46b and AISC *Manual* Table 9-4, the minimum required bearing length for web local yielding is:

LRFD	ASD
$l_{b\,min} = \dfrac{R_u - \phi R_1}{\phi R_2} \geq k_{des}$ $= \dfrac{54.8 \text{ kips} - 56.0 \text{ kips}}{20.0 \text{ kip/in.}} \geq 1.12 \text{ in.}$ which results in a negative quantity. Therefore, $l_{b\,min} = k_{des} = 1.12 \text{ in.}$	$l_{b\,min} = \dfrac{R_a - R_1/\Omega}{R_2/\Omega} \geq k_{des}$ $= \dfrac{36.5 \text{ kips} - 37.3 \text{ kips}}{13.3 \text{ kip/in.}} \geq 1.12 \text{ in.}$ which results in a negative quantity. Therefore, $l_{b\,min} = k_{des} = 1.12 \text{ in.}$

For web local crippling, the maximum bearing length-to-depth ratio is determined as follows (including a ¼-in. tolerance to account for possible beam underrun):

$$\left(\frac{l_b}{d}\right)_{max} = \frac{3.25 \text{ in.}}{21.0 \text{ in.}}$$
$$= 0.155 < 0.2$$

From AISC *Manual* Equations 9-48a or 9-48b and AISC *Manual* Table 9-4, when $\dfrac{l_b}{d} \leq 0.2$:

LRFD	ASD
$l_{b\,min} = \dfrac{R_u - \phi R_3}{\phi R_4}$ $= \dfrac{54.8 \text{ kips} - 71.7 \text{ kips}}{5.37 \text{ kip/in.}}$	$l_{b\,min} = \dfrac{R_a - R_3/\Omega}{R_4/\Omega}$ $= \dfrac{36.5 \text{ kips} - 47.8 \text{ kips}}{3.58 \text{ kip/in.}}$

LRFD	ASD
This results in a negative quantity; therefore, $l_{b\,min} = k_{des} = 1.12 \text{ in.}$	This results in a negative quantity; therefore, $l_{b\,min} = k_{des} = 1.12 \text{ in.}$

Connection Selection

AISC *Manual* Table 10-6 includes checks for the limit states of shear yielding and flexural yielding of the outstanding angle leg.

For an 8-in. angle length with a ⅝-in. thickness, a 3½-in. minimum outstanding leg, and conservatively using $l_{b,\,req}$ = 1⅛ in., from AISC *Manual* Table 10-6:

LRFD	ASD
$\phi R_n = 81.0 \text{ kips} > 54.8 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 53.9 \text{ kips} > 36.5 \text{ kips}$ **o.k.**

From AISC *Manual* Table 10-6, for a L8×4×⅝ (4-in. OSL), 8-in. long, with 5⁄16-in. fillet welds, the weld available strength is:

LRFD	ASD
$\phi R_n = 66.7 \text{ kips} > 54.8 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 44.5 \text{ kips} > 36.5 \text{ kips}$ **o.k.**

Use two ¾-in.-diameter bolts with threads not excluded from the shear plane (thread condition N) to connect the beam to the seat angle.

The strength of the bolts, welds and angles must be verified if horizontal forces are added to the connection.

Top Angle, Bolts and Welds

Use an L4×4×¼ with two ¾-in.-diameter bolts with threads not excluded from the shear plane (thread condition N) through the supported beam leg of the angle. Use a 3⁄16-in. fillet weld along the toe of the angle to the column flange. See the discussion in AISC *Manual* Part 10.

Conclusion

The connection design shown in Figure II.A-13-1 is acceptable.

EXAMPLE II.A-14 BOLTED/WELDED STIFFENED SEATED CONNECTION (BEAM-TO-COLUMN FLANGE)

Given:

Verify the bolted/welded stiffened seated connection between an ASTM A992 W21×68 beam and an ASTM A992 W14×90 column flange, as shown in Figure II.A-14-1, to support the following end reactions:

$R_D = 21$ kips
$R_L = 62.5$ kips

Use 70-ksi weld electrodes and ASTM A36 angles and plate.

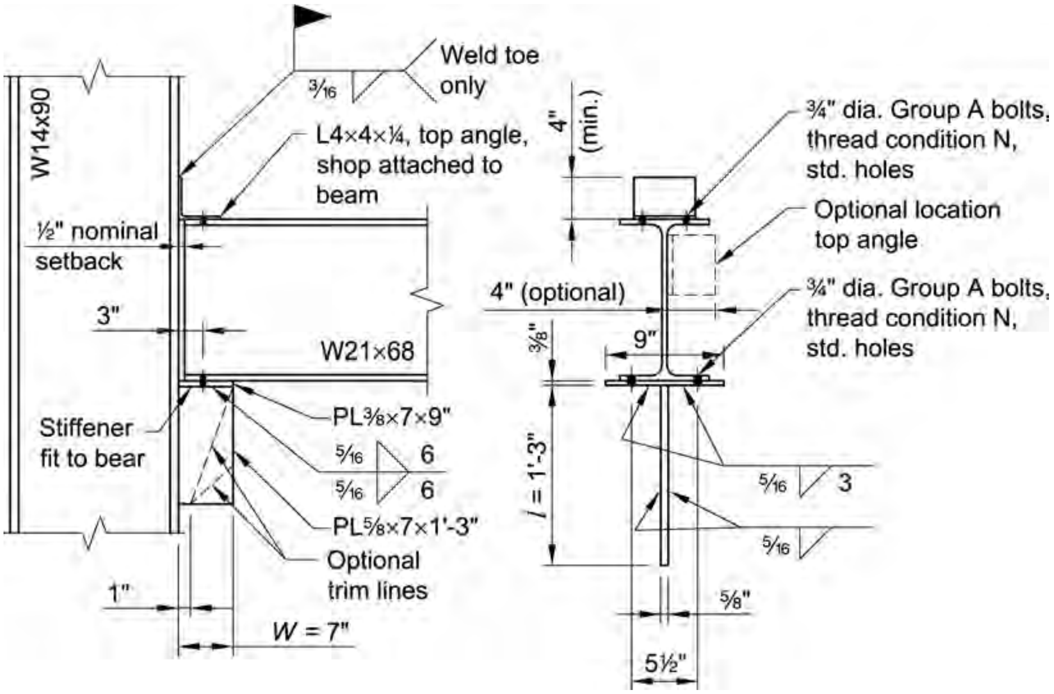

Fig. II.A-14-1. Connection geometry for Example II.A-14.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Beam and column
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Angle and plates
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W21×68
t_w = 0.430 in.
d = 21.1 in.
b_f = 8.27 in.
t_f = 0.685 in.
k_{des} = 1.19 in.

Column
W14×90
t_f = 0.710 in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(21\text{ kips}) + 1.6(62.5\text{ kips})$ $= 125\text{ kips}$	$R_a = 21\text{ kips} + 62.5\text{ kips}$ $= 83.5\text{ kips}$

Required Stiffener Width

The minimum stiffener width, W_{min}, is determined based on limit states of web local yielding and web local crippling for the beam.

The minimum stiffener width for web local crippling of the beam web, for the force applied less than one-half of the depth of the beam from the end of the beam and assuming $l_b/d > 0.2$, is determined from AISC *Manual* Equations 9-49a or 9-49b and AISC *Manual* Table 9-4, as follows (including a ¼-in. tolerance to account for possible beam underrun):

LRFD	ASD
$W_{min} = \dfrac{R_u - \phi R_5}{\phi R_6} + \text{setback} + \text{underrun}$ $= \dfrac{125\text{ kips} - 75.9\text{ kips}}{7.95\text{ kip/in.}} + \tfrac{1}{2}\text{ in.} + \tfrac{1}{4}\text{ in.}$ $= 6.93\text{ in.}$	$W_{min} = \dfrac{R_a - R_5/\Omega}{R_6/\Omega} + \text{setback} + \text{underrun}$ $= \dfrac{83.5\text{ kips} - 50.6\text{ kips}}{5.30\text{ kip/in.}} + \tfrac{1}{2}\text{ in.} + \tfrac{1}{4}\text{ in.}$ $= 6.96\text{ in.}$

The minimum stiffener width for web local yielding of the beam, for the force applied less than the depth of the beam from the end of the beam, is determined from AISC *Manual* Equations 9-46a or 9-46b and AISC *Manual* Table 9-4, as follows (including a ¼-in. tolerance to account for possible beam underrun):

LRFD	ASD
$W_{min} = \dfrac{R_u - \phi R_1}{\phi R_2} + \text{setback} + \text{underrun}$ $= \dfrac{125\text{ kips} - 64.0\text{ kips}}{21.5\text{ kip/in.}} + \tfrac{1}{2}\text{ in.} + \tfrac{1}{4}\text{ in.}$ $= 3.59\text{ in.}$	$W_{min} = \dfrac{R_a - R_1/\Omega}{R_2/\Omega} + \text{setback} + \text{underrun}$ $= \dfrac{83.5\text{ kips} - 42.6\text{ kips}}{14.3\text{ kip/in.}} + \tfrac{1}{2}\text{ in.} + \tfrac{1}{4}\text{ in.}$ $= 3.61\text{ in.}$

Use W = 7 in.

Check assumption:

$$\begin{aligned}\frac{l_b}{d} &= \frac{6.25\text{ in.}}{21.1\text{ in.}} \\ &= 0.296 > 0.2 \quad \textbf{o.k.}\end{aligned}$$

Stiffener Length and Stiffene- to-Column Flange Weld Size

Use a stiffener with l = 15 in. and 5⁄16-in. fillet welds.

From AISC *Manual* Table 10-8, with W = 7 in.:

LRFD	ASD
$\phi R_n = 139\text{ kips} > 125\text{ kips} \quad \textbf{o.k.}$	$\dfrac{R_n}{\Omega} = 93.0\text{ kips} > 83.5\text{ kips} \quad \textbf{o.k.}$

Seat Plate Welds

Use 5⁄16-in. fillet welds on each side of the stiffener. From AISC *Manual* Figure 10-10(b), minimum length of seat plate-to-column flange weld is 0.2(L) = 3 in. As discussed in AISC *Manual* Part 10, the weld between the seat plate and stiffener plate is required to have a strength equal to or greater than the weld between the seat plate and the column flange, use 5⁄16-in. fillet welds on each side of the stiffener to the seat plate; length of weld = 6 in. per side.

Seat Plate Dimensions

A dimension of 9 in. is adequate to accommodate the ¾-in.-diameter bolts on a 5½-in. gage connecting the beam flange to the seat plate.

Use a PL⅜ in.×7 in.×9 in. for the seat.

Stiffener Plate Thickness

As discussed in AISC *Manual* Part 10, the minimum stiffener plate thickness to develop the seat plate weld for F_y = 36 ksi plate material is:

$$\begin{aligned}t_{min} &= 2w \\ &= 2\left(5/16\text{ in.}\right) \\ &= 5/8\text{ in.}\end{aligned}$$

As discussed in AISC *Manual* Part 10, the minimum plate thickness for a stiffener with F_y = 36 ksi and a beam with F_y = 50 ksi is:

$$\begin{aligned}t_{min} &= \left(\frac{50\text{ ksi}}{36\text{ ksi}}\right)t_w \\ &= \left(\frac{50\text{ ksi}}{36\text{ ksi}}\right)\left(0.430\text{ in.}\right) \\ &= 0.597\text{ in.} < 5/8\text{ in.}\end{aligned}$$

Use a PL⅝ in.×7 in.×1 ft 3 in.

Top Angle, Bolts and Welds

Use an L4×4×¼ with two ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) through the supported beam leg of the angle. Use a $^3/_{16}$-in. fillet weld along the toe of the angle to the column flange. See discussion in AISC *Manual* Part 10.

Conclusion

The connection design shown in Figure II.A-14-1 is acceptable.

EXAMPLE II.A-15 BOLTED/WELDED STIFFENED SEATED CONNECTION (BEAM-TO-COLUMN WEB)

Given:

Verify the stiffened seated connection between an ASTM A992 W21×68 beam and an ASTM A992 W14×90 column web, as shown in Figure II.A-15-1, to support the following beam end reactions:

$R_D = 21$ kips
$R_L = 62.5$ kips

Use 70-ksi weld electrodes and ASTM A36 angles and plate.

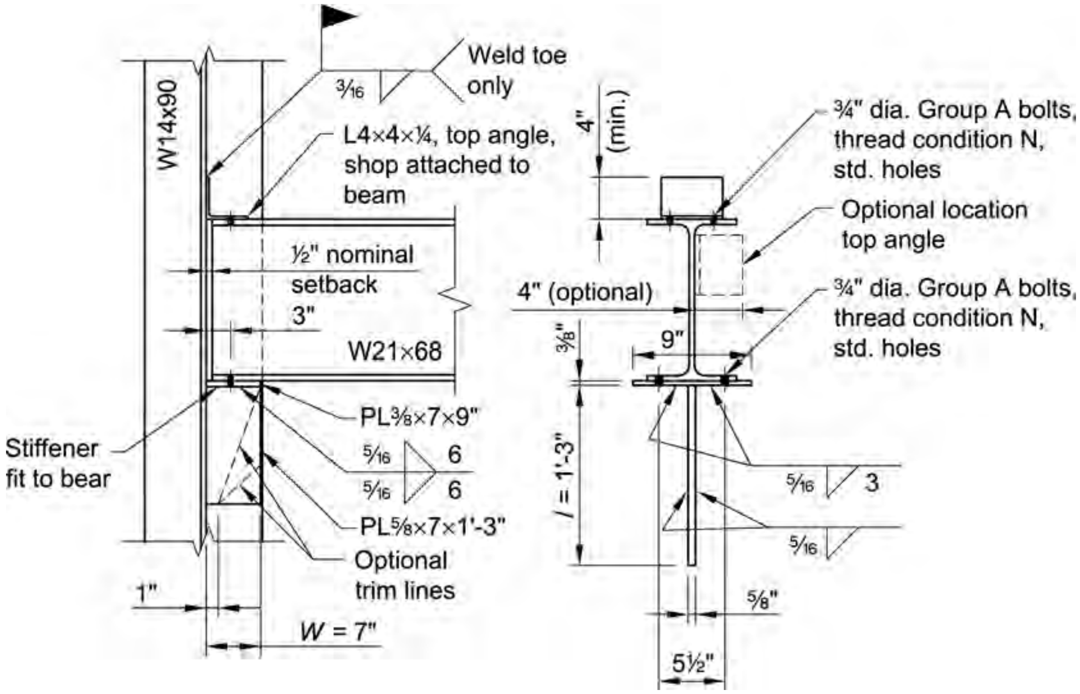

Fig. II.A-15-1. Connection geometry for Example II.A-15.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Beam and column
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Angle and Plates
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W21×68
t_w = 0.430 in.
d = 21.1 in.
b_f = 8.27 in.
t_f = 0.685 in.
k_{des} = 1.19 in.

Column
W14×90
t_w= 0.440 in.
T = 10 in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(21\text{ kips}) + 1.6(62.5\text{ kips})$ $= 125\text{ kips}$	$R_a = 21\text{ kips} + 62.5\text{ kips}$ $= 83.5\text{ kips}$

Required Stiffener Width

The minimum stiffener width, W_{min}, is determined based on limit states of web local yielding and web local crippling for the beam.

The minimum stiffener width for web local crippling of the beam web, for the force applied less than one-half of the depth of the beam from the end of the beam and assuming $l_b/d > 0.2$, is determined from AISC *Manual* Equations 9-49a or 9-49b and AISC *Manual* Table 9-4, as follows (including a ¼-in. tolerance to account for possible beam underrun):

LRFD	ASD
$W_{min} = \dfrac{R_u - \phi R_5}{\phi R_6} + \text{setback} + \text{underrun}$ $= \dfrac{125\text{ kips} - 75.9\text{ kips}}{7.95\text{ kip/in.}} + \tfrac{1}{2}\text{ in.} + \tfrac{1}{4}\text{ in.}$ $= 6.93\text{ in.}$	$W_{min} = \dfrac{R_a - R_5/\Omega}{R_6/\Omega} + \text{setback} + \text{underrun}$ $= \dfrac{83.5\text{ kips} - 50.6\text{ kips}}{5.30\text{ kip/in.}} + \tfrac{1}{2}\text{ in.} + \tfrac{1}{4}\text{ in.}$ $= 6.96\text{ in.}$

The minimum stiffener width for web local yielding of the beam, for the force applied less than the depth of the beam from the end of the beam, is determined from AISC *Manual* Equations 9-46a or 9-46b and AISC *Manual* Table 9-4, as follows (including a ¼-in. tolerance to account for possible beam underrun):

LRFD	ASD
$W_{min} = \dfrac{R_u - \phi R_1}{\phi R_2} + \text{setback} + \text{underrun}$ $= \dfrac{125\text{ kips} - 64.0\text{ kips}}{21.5\text{ kip/in.}} + \tfrac{1}{2}\text{ in.} + \tfrac{1}{4}\text{ in.}$ $= 3.59\text{ in.}$	$W_{min} = \dfrac{R_a - R_1/\Omega}{R_2/\Omega} + \text{setback} + \text{underrun}$ $= \dfrac{83.5\text{ kips} - 42.6\text{ kips}}{14.3\text{ kip/in.}} + \tfrac{1}{2}\text{ in.} + \tfrac{1}{4}\text{ in.}$ $= 3.61\text{ in.}$

Use W = 7 in.

Check assumption:

$$\begin{aligned}\frac{l_b}{d} &= \frac{6.25\text{ in.}}{21.1\text{ in.}}\\ &= 0.296 > 0.2 \quad \textbf{o.k.}\end{aligned}$$

Stiffener Length and Stiffener to Column Flange Weld Size

Use a stiffener with l = 15 in. and 5⁄16-in. fillet welds.

From AISC *Manual* Table 10-8, with W = 7 in., the weld available strength is:

LRFD	ASD
$\phi R_n = 139\text{ kips} > 125\text{ kips} \quad \textbf{o.k.}$	$\dfrac{R_n}{\Omega} = 93.0\text{ kips} > 83.5\text{ kips} \quad \textbf{o.k.}$

Seat Plate Welds

Use 5⁄16-in. fillet welds on each side of the stiffener. From AISC *Manual* Figure 10-10(b), minimum length of seat plate-to-column flange weld is 0.2(L) = 3 in. As discussed in AISC *Manual* Part 10, the weld between the seat plate and stiffener plate is required to have a strength equal to or greater than the weld between the seat plate and the column flange, use 5⁄16-in. fillet welds on each side of the stiffener to the seat plate; length of weld = 6 in. per side.

Seat Plate Dimensions

A dimension of 9 in. is adequate to accommodate the ¾-in.-diameter bolts on a 5½-in. gage connecting the beam flange to the seat plate.

Use a PL⅜ in.×7 in.×9 in. for the seat.

Stiffener Plate Thickness

As discussed in AISC *Manual* Part 10, the minimum stiffener plate thickness to develop the seat plate weld for F_y = 36 ksi plate material is:

$$\begin{aligned}t_{min} &= 2w\\ &= 2\left(\tfrac{5}{16}\text{ in.}\right)\\ &= \tfrac{5}{8}\text{ in.}\end{aligned}$$

As discussed in AISC *Manual* Part 10, the minimum plate thickness for a stiffener with F_y = 36 ksi and a beam with F_y = 50 ksi is:

$$\begin{aligned}t_{min} &= \left(\frac{50\text{ ksi}}{36\text{ ksi}}\right)t_w\\ &= \left(\frac{50\text{ ksi}}{36\text{ ksi}}\right)(0.430\text{ in.})\\ &= 0.597\text{ in.} < \tfrac{5}{8}\text{ in.}\end{aligned}$$

Use a PL⅝ in.×7 in.×1 ft 3 in.

Top Angle, Bolts and Welds

Use an L4×4×¼ with two ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) through the supported beam leg of the angle. Use a 3⁄16-in. fillet weld along the toe of the angle to the column web. See discussion in AISC *Manual* Part 10.

Column Web

If the seat is welded to a column web, the base metal strength of the column must be checked.

If only one side of the column web has a stiffened seated connection, then:

$$t_{min} = \frac{3.09D}{F_u} \qquad \text{(\textit{Manual} Eq. 9-2)}$$

$$= \frac{3.09(5\text{ sixteenths})}{65\text{ ksi}}$$

$$= 0.238\text{ in.} < 0.440\text{ in.} \quad \textbf{o.k.}$$

If both sides of the column web have a stiffened seated connection, then:

$$t_{min} = \frac{6.19D}{F_u} \qquad \text{(\textit{Manual} Eq. 9-3)}$$

$$= \frac{6.19(5\text{ sixteenths})}{65\text{ ksi}}$$

$$= 0.476\text{ in.} > 0.440\text{ in.} \quad \textbf{n.g.}$$

The column is sufficient for a one-sided stiffened seated connection. For a two-sided connection the weld available strength must be reduced as discussed in AISC *Manual* Part 10.

Note: Additional detailing considerations for stiffened seated connections are given in Part 10 of the AISC *Manual*.

Conclusion

The connection design shown in Figure II.A-15-1 is acceptable.

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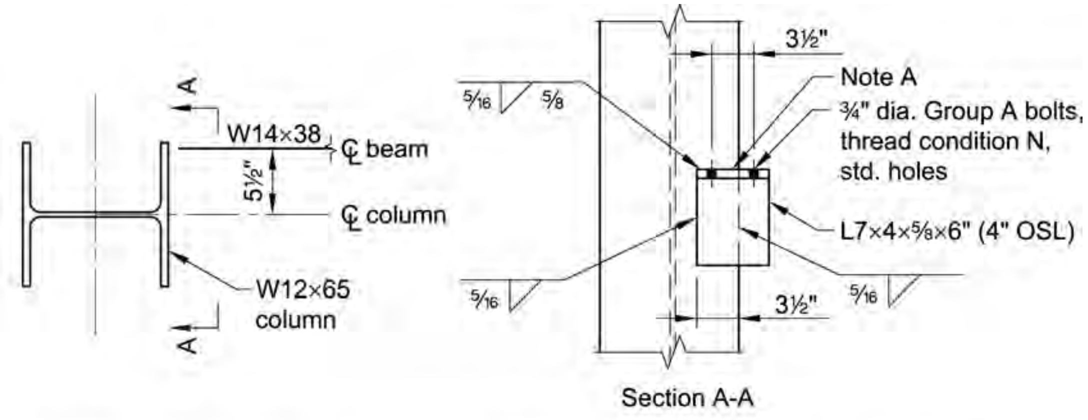
EXAMPLE II.A-16 OFFSET UNSTIFFENED SEATED CONNECTION (BEAM-TO-COLUMN FLANGE)

Given:

Verify the seat angle and weld size required for the unstiffened seated connection between an ASTM A992 W14×38 beam and an ASTM A992 W12×65 column flange connection with an offset of 5½ in., as shown in Figure II.A-16-1, to support the following beam end reactions:

$R_D = 5$ kips
$R_L = 15$ kips

Use an ASTM A36 angle and 70-ksi weld electrodes.



Note A: End return is omitted because the AWS Code does not permit weld returns to be carried around the corner formed by the column flange toe and seat angle heel.
Note B: Beam and top angle not shown for clarity.
Note C: The nominal setback of the beam from the face of the flange is ½ in.

Fig. II.A-16-1. Connection geometry for Example II.A-16.

Solution:

From AISC *Manual* Tables 2-4, the material properties are as follows:

Beam and column
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Angle
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W14×38
$d = 14.1$ in.
$k_{des} = 0.915$ in.

Column
W12×65
$t_f = 0.605$ in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(5 \text{ kips}) + 1.6(15 \text{ kips})$ $= 30.0 \text{ kips}$	$R_a = 5 \text{ kips} + 15 \text{ kips}$ $= 20.0 \text{ kips}$

Minimum Bearing Length

From AISC *Manual* Part 10, the minimum required bearing length, $l_{b\ min}$, is the length of bearing required for the limit states of web local yielding and web local crippling on the beam, but not less than k_{des}.

From AISC *Manual* Equations 9-46a or 9-46b and AISC *Manual* Table 9-4, the minimum required bearing length for web local yielding is:

LRFD	ASD
$l_{b\ min} = \dfrac{R_u - \phi R_1}{\phi R_2} \geq k_{des}$ $= \dfrac{30.0 \text{ kips} - 35.5 \text{ kips}}{15.5 \text{ kip/in.}} \geq 0.915 \text{ in.}$ This results in a negative quantity; therefore, $l_{b\ min} = k_{des} = 0.915 \text{ in.}$	$l_{b\ min} = \dfrac{R_a - R_1/\Omega}{R_2/\Omega} \geq k_{des}$ $= \dfrac{20.0 \text{ kips} - 23.6 \text{ kips}}{10.3 \text{ kip/in.}} \geq 0.915 \text{ in.}$ This results in a negative quantity; therefore, $l_{b\ min} = k_{des} = 0.915 \text{ in.}$

From AISC *Manual* Equations 9-48a or 9-48b and AISC *Manual* Table 9-4, the minimum required bearing length for web local crippling, assuming $l_b/d \leq 0.2$, is:

LRFD	ASD
$l_{b\ min} = \dfrac{R_u - \phi R_3}{\phi R_4} \geq k_{des}$ $= \dfrac{30.0 \text{ kips} - 44.7 \text{ kips}}{4.45 \text{ kip/in.}} \geq 0.915 \text{ in.}$ This results in a negative quantity; therefore, $l_{b\ min} = k_{des} = 0.915 \text{ in.}$	$l_{b\ min} = \dfrac{R_a - R_3/\Omega}{R_4/\Omega} \geq k_{des}$ $= \dfrac{20.0 \text{ kips} - 29.8 \text{ kips}}{2.96 \text{ kip/in.}} \geq 0.915 \text{ in.}$ This results in a negative quantity; therefore, $l_{b\ min} = k_{des} = 0.915 \text{ in.}$

Check assumption:

$$\frac{l_b}{d} = \frac{0.915 \text{ in.}}{14.1 \text{ in.}}$$
$$= 0.0649 < 0.2 \quad \textbf{o.k.}$$

Seat Angle and Welds

The required strength for the righthand weld can be determined by summing moments about the lefthand weld.

LRFD	ASD
$R_{uR} = \frac{(30.0\text{ kips})(3.00\text{ in.})}{3.50\text{ in.}}$ $= 25.7\text{ kips}$	$R_{aR} = \frac{(20.0\text{ kips})(3.00\text{ in.})}{3.50\text{ in.}}$ $= 17.1\text{ kips}$

Conservatively design the seat for twice the force in the more highly loaded weld. Therefore, design the seat for the following:

LRFD	ASD
$R_u = 2(25.7\text{ kips})$ $= 51.4\text{ kips}$	$R_a = 2(17.1\text{ kips})$ $= 34.2\text{ kips}$

Use a 6-in. angle length with a ⅝-in. thickness and a 3½-in. minimum outstanding leg and conservatively using $l_{b,\,req}$ = 15⁄16 in., from AISC *Manual* Table 10-6:

LRFD	ASD
$\phi R_n = 81.0\text{ kips} > 51.4\text{ kips}$ **o.k.**	$\frac{R_n}{\Omega} = 54.0\text{ kips} > 34.2\text{ kips}$ **o.k.**

Use an L7×4×⅝ (4-in. OSL), 6-in. long with 5⁄16-in. fillet welds. From AISC *Manual* Table 10-6, the weld available strength is:

LRFD	ASD
$\phi R_n = 53.4\text{ kips} > 51.4\text{ kips}$ **o.k.**	$\frac{R_n}{\Omega} = 35.6\text{ kips} > 34.2\text{ kips}$ **o.k.**

Use an L7×4×⅝×0 ft 6 in. for the seat angle. Use two ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) to connect the beam to the seat angle. Weld the angle to the column with 5⁄16-in. fillet welds.

Top Angle, Bolts and Welds

Use an L4×4×¼ with two ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) through the outstanding leg of the angle.

Use a 3⁄16-in. fillet weld along the toe of the angle to the column flange [maximum size permitted by AISC *Specification* Section J2.2b(b)(2)].

Conclusion

The connection is found to be adequate as given for the applied loads.

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EXAMPLE II.A-17A SINGLE-PLATE CONNECTION (CONVENTIONAL BEAM-TO-COLUMN FLANGE)

Given:

Verify a single-plate connection between an ASTM A992 W16×50 beam and an ASTM A992 W14×90 column flange, as shown in Figure II.A-17A-1, to support the following beam end reactions:

$R_D = 8$ kips
$R_L = 25$ kips

Use 70-ksi electrodes and an ASTM A36 plate.

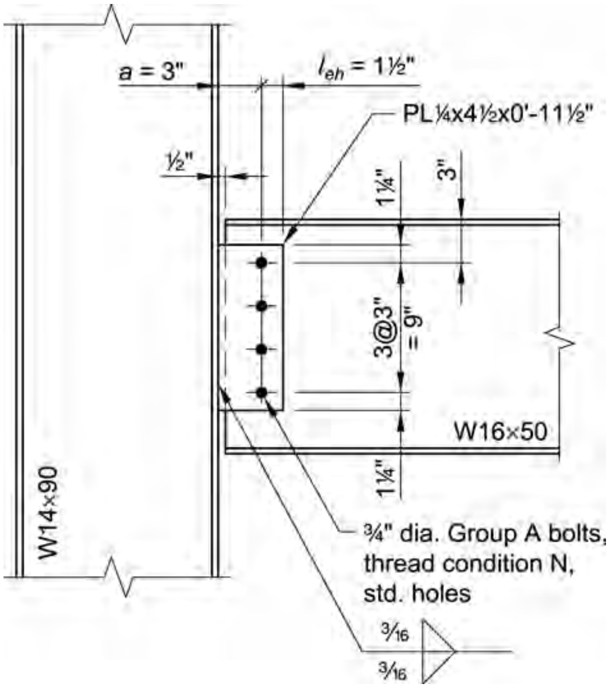

Fig. II.A-17A-1. Connection geometry for Example II.A-17A.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Beam and column
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Plate
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W16×50
$t_w = 0.380$ in.
$d = 16.3$ in.

Column
W14×90
$t_f = 0.710$ in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(8\text{ kips}) + 1.6(25\text{ kips})$ $= 49.6\text{ kips}$	$R_a = 8\text{ kips} + 25\text{ kips}$ $= 33.0\text{ kips}$

Connection Selection

AISC *Manual* Table 10-10a includes checks for the limit states of bolt shear, bolt bearing on the plate, tearout on the plate, shear yielding of the plate, shear rupture of the plate, block shear rupture of the plate, and weld shear.

Use four rows of ¾-in.-diameter bolts in standard holes, ¼-in. plate thickness, and 3⁄16-in. fillet weld size. From AISC *Manual* Table 10-10a, the bolt, weld and single-plate available strength is:

LRFD	ASD
$\phi R_n = 52.2\text{ kips} > 49.6\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 34.8\text{ kips} > 33.0\text{ kips}$ **o.k.**

Bolt Bearing and Tearout for Beam Web

Similar to the discussion in AISC *Manual* Part 10 for conventional, single-plate shear connections, the bearing and tearout are checked in accordance with AISC *Specification* Section J3.10, assuming the reaction is applied concentrically.

The available bearing and tearout strength of the beam web is determined using AISC *Manual* Table 7-4 with $s = 3$ in.

LRFD	ASD
$\phi R_n = (4\text{ bolts})(87.8\text{ kip/in.})(0.380\text{ in.})$ $= 134\text{ kips} > 49.6\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = (4\text{ bolts})(58.5\text{ kip/in.})(0.380\text{ in.})$ $= 88.9\text{ kips} > 33.0\text{ kips}$ **o.k.**

Note: To provide for stability during erection, it is recommended that the minimum plate length be one-half the T-dimension of the beam to be supported. AISC *Manual* Table 10-1 may be used as a reference to determine the recommended maximum and minimum connection lengths for a supported beam. Block shear rupture, shear yielding, and shear rupture will not control for an uncoped section.

Conclusion

The connection is found to be adequate as given for the applied loads.

EXAMPLE II.A-17B SINGLE-PLATE CONNECTION SUBJECT TO AXIAL AND SHEAR LOADING (BEAM-TO-COLUMN FLANGE)

Given:

Verify the available strength of a single-plate connection for an ASTM A992 W18×50 beam connected to an ASTM A992 W14×90 column flange, as shown in Figure II.A-17B-1, to support the following beam end reactions:

LRFD	ASD
Shear, V_u = 75 kips Axial tension, N_u = 60 kips	Shear, V_a = 50 kips Axial tension, N_a = 40 kips

Use 70-ksi electrodes and an ASTM A572 Grade 50 plate.

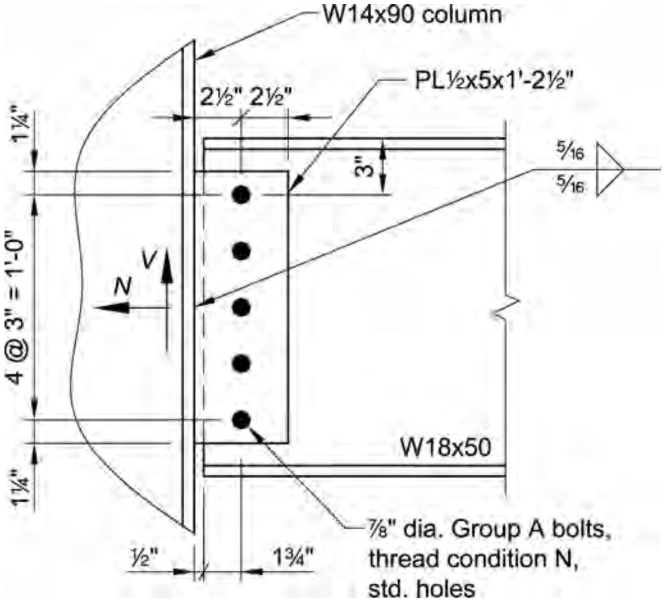

Fig. II.A-17B-1. Connection geometry for Example II.A-17B.

Solution:

From AISC *Manual* Table 2-4 and Table 2-5, the material properties are as follows:

Beam and column
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Plate
ASTM A572 Grade 50
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W18×50
A_g = 14.7 in.^{2}
d = 18.0 in.

t_w = 0.355 in.
t_f = 0.570 in.

Column
W14×90
t_f = 0.710 in.

From AISC *Specification* Table J3.3, for ⅞-in.-diameter bolts with standard holes:

d_h = 15⁄16 in.

The resultant load is:

LRFD	ASD
$R_u = \sqrt{V_u{}^2 + N_u{}^2}$ $= \sqrt{(75\text{ kips})^2 + (60\text{ kips})^2}$ $= 96.0\text{ kips}$	$R_a = \sqrt{V_a{}^2 + N_a{}^2}$ $= \sqrt{(50\text{ kips})^2 + (40\text{ kips})^2}$ $= 64.0\text{ kips}$

The resultant load angle, measured from the vertical, is:

LRFD	ASD
$\theta = \tan^{-1}\left(\dfrac{60\text{ kips}}{75\text{ kips}}\right)$ $= 38.7°$	$\theta = \tan^{-1}\left(\dfrac{40\text{ kips}}{50\text{ kips}}\right)$ $= 38.7°$

Bolt Shear Strength

From AISC *Manual* Table 10-9, for single-plate shear connections with standard holes and n = 5:

$$e = \frac{a}{2} = \frac{2\tfrac{1}{2}\text{ in.}}{2} = 1.25\text{ in.}$$

The coefficient for eccentrically loaded bolts is determined by interpolating from AISC *Manual* Table 7-6 for Angle = 30°, n = 5 and e_x = 1.25 in. Note that 30° is used conservatively in order to employ AISC *Manual* Table 7-6. A direct analysis method can be performed to obtain a more precise value using the instantaneous center of rotation method.

C = 4.60

From AISC *Manual* Table 7-1, the available shear strength for a ⅞-in.-diameter Group A bolt with threads not excluded from the shear plane (thread condition N) is:

LRFD	ASD
$\phi r_n = 24.3\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 16.2\text{ kips/bolt}$

Bolt Bearing on the Beam Web

Note that bolt bearing and tearout of the beam web will control over bearing and tearout of the plate because the beam web is thinner and has less edge distance than the plate; therefore, those limit states will only be checked on the beam web.

The nominal bearing strength is determined using AISC *Specification* Equation J3-6b in lieu of Equation J3-6a, because plowing of the bolts in the beam web is desirable to provide some flexibility in the connection.

$$\begin{aligned} r_n &= 3.0dtF_u \\ &= 3.0\left(\tfrac{7}{8}\text{ in.}\right)\left(0.355\text{ in.}\right)\left(65\text{ ksi}\right) \\ &= 60.6\text{ kips/bolt} \end{aligned}$$

(*Spec*. Eq. J3-6b)

From AISC *Specification* Section J3.10, the available bearing strength of the beam per bolt is:

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75\left(60.6\text{ kips/bolt}\right)$ $= 45.5\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{60.6\text{ kips/bolt}}{2.00}$ $= 30.3\text{ kips/bolt}$

Bolt Tearout on the Beam Web

The nominal tearout strength is determined using AISC *Specification* Equation J3-6d in lieu of Equation J3-6c, because plowing of the bolts in the beam web is desirable to provide some flexibility in the connection.

Because the direction of the load on the bolt is unknown, the minimum bolt edge distance is used to determine a worst case available tearout strength. The bolt edge distance for the web in the horizontal direction controls for this design. If a computer program is available, the true l_c can be calculated based on the instantaneous center of rotation. Therefore, for worst case edge distance in the beam web, and considering possible length underrun of ¼ in. on the beam length:

$$\begin{aligned} l_c &= l_{eh} - 0.5d_h - \text{underrun} \\ &= 1\tfrac{3}{4}\text{ in.} - 0.5\left(\tfrac{15}{16}\text{ in.}\right) - \tfrac{1}{4}\text{ in.} \\ &= 1.03\text{ in.} \end{aligned}$$

$$\begin{aligned} r_n &= 1.5l_ctF_u \\ &= 1.5\left(1.03\text{ in.}\right)\left(0.355\text{ in.}\right)\left(65\text{ ksi}\right) \\ &= 35.7\text{ kips/bolt} \end{aligned}$$

(*Spec*. Eq. J3-6d)

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75\left(35.7\text{ kips}\right)$ $= 26.8\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{35.7\text{ kips}}{2.00}$ $= 17.9\text{ kips/bolt}$

Strength of Bolted Connection

Bolt shear is the controlling limit state for all bolts at the connection to the beam web. The available strength of the connection is:

LRFD	ASD
$\phi R_n = C\phi r_n$ $= 4.60(24.3\text{ kips/bolt})$ $= 112\text{ kips} > 96.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{Cr_n}{\Omega}$ $= 4.60(16.2\text{ kips/bolt})$ $= 74.5\text{ kips} > 64.0\text{ kips}$ **o.k.**

Strength of Weld

From AISC *Manual* Part 10, a weld size of ($\frac{5}{8}$)t_p is used to develop the strength of the shear plate, because, in general, the moment generated by this connection is indeterminate.

$$\begin{aligned} w &= \tfrac{5}{8}t_p \\ &= \tfrac{5}{8}(\tfrac{1}{2}\text{ in.}) \\ &= \tfrac{5}{16}\text{ in.} \end{aligned}$$

Use a two-sided $\frac{5}{16}$-in. fillet weld.

Shear Strength of Supporting Column Flange

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the column flange is determined as follows:

$$\begin{aligned} A_{nv} &= (2\text{ shear planes})lt_f \\ &= (2\text{ shear planes})(14.5\text{ in.})(0.710\text{ in.}) \\ &= 20.6\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60F_uA_{nv} \\ &= 0.60(65\text{ ksi})(20.6\text{ in.}^2) \\ &= 803\text{ kips} \end{aligned} \qquad (Spec.\text{ Eq. J4-4})$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(803\text{ kips})$ $= 602\text{ kips} > 75\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{803\text{ kips}}{2.00}$ $= 402\text{ kips} > 50\text{ kips}$ **o.k.**

The available shear yielding strength of the column flange need not be checked because $A_{nv} = A_{gv}$ and shear rupture will control.

Shear Yielding Strength of the Plate

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the plate is determined as follows:

$$\begin{aligned} A_{gv} &= lt \\ &= (14.5\text{ in.})(\tfrac{1}{2}\text{ in.}) \\ &= 7.25\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60F_yA_{gv} \\ &= 0.60(50\text{ ksi})\left(7.25\text{ in.}^2\right) \\ &= 218\text{ kips} \end{aligned}$$ (*Spec*. Eq. J4-3)

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(218\text{ kips})$ $= 218\text{ kips} > 75\text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{218\text{ kips}}{1.50}$ $= 145\text{ kips} > 50\text{ kips}$ **o.k.**

Tensile Yielding Strength of the Plate

From AISC *Specification* Section J4.1(a), the available tensile yielding strength of the plate is determined as follows:

$$\begin{aligned} A_g &= lt \\ &= (14.5\text{ in.})(\tfrac{1}{2}\text{ in.}) \\ &= 7.25\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= F_yA_g \\ &= (50\text{ ksi})\left(7.25\text{ in.}^2\right) \\ &= 363\text{ kips} \end{aligned}$$ (*Spec*. Eq. J4-1)

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(363\text{ kips})$ $= 327\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{363\text{ kips}}{1.67}$ $= 217\text{ kips} > 40\text{ kips}$ **o.k.**

Flexural Yielding of the Plate

The required flexural strength is calculated based upon the required shear strength and the eccentricity previously calculated:

LRFD	ASD
$M_u = V_ue$ $= (75\text{ kips})(1.25\text{ in.})$ $= 93.8\text{ kip-in.}$	$M_a = V_ae$ $= (50\text{ kips})(1.25\text{ in.})$ $= 62.5\text{ kip-in.}$

From AISC *Manual* Part 10, the plate buckling will not control for the conventional configuration. The flexural yielding strength is determined as follows:

$$\begin{aligned} Z_g &= \frac{t_pl^2}{4} \\ &= \frac{(\tfrac{1}{2}\text{ in.})(14.5\text{ in.})^2}{4} \\ &= 26.3\text{ in.}^3 \end{aligned}$$

$$
\begin{aligned}
M_n &= F_y Z_g \\
&= (50\text{ ksi})\left(26.3\text{ in.}^3\right) \\
&= 1{,}320\text{ kip-in.}
\end{aligned}
$$

LRFD	ASD
$\phi = 0.90$ $\phi M_n = 0.90(1{,}320\text{ kip-in.})$ $= 1{,}190\text{ kip-in.} > 93.8\text{ kip-in.}$ **o.k.**	$\Omega = 1.67$ $\dfrac{M_n}{\Omega} = \dfrac{1{,}320\text{ kip-in.}}{1.67}$ $= 790\text{ kip-in.} > 62.5\text{ kip-in.}$ **o.k.**

Interaction of Axial, Flexural and Shear Yielding in Plate

AISC *Specification* Chapter H does not address combined flexure and shear. The method employed here is derived from Chapter H in conjunction with AISC *Manual* Equation 10-5.

LRFD	ASD
$\dfrac{N_u}{\phi R_{np}} = \dfrac{60\text{ kips}}{327\text{ kips}}$ $= 0.183$ Because $\dfrac{N_u}{\phi R_{np}} < 0.2$: $\left(\dfrac{N_u}{2\phi R_{np}} + \dfrac{M_u}{\phi M_n}\right)^2 + \left(\dfrac{V_u}{\phi R_{nv}}\right)^2 \le 1$ $\left[\dfrac{60\text{ kips}}{2(327\text{ kips})} + \dfrac{93.8\text{ kip-in.}}{1{,}190\text{ kip-in.}}\right]^2 + \left(\dfrac{75\text{ kips}}{218\text{ kips}}\right)^2 \le 1$ $0.147 < 1$ **o.k.**	$\dfrac{N_a}{R_{np}/\Omega} = \dfrac{40\text{ kips}}{217\text{ kips}}$ $= 0.184$ Because $\dfrac{N_a}{R_{np}/\Omega} < 0.2$: $\left(\dfrac{\Omega N_a}{2R_{np}} + \dfrac{\Omega M_a}{M_n}\right)^2 + \left(\dfrac{\Omega V_a}{R_{nv}}\right)^2 \le 1$ $\left[\dfrac{40\text{ kips}}{2(217\text{ kips})} + \dfrac{62.5\text{ kip-in.}}{790\text{ kip-in.}}\right]^2 + \left(\dfrac{50\text{ kips}}{145\text{ kips}}\right)^2 \le 1$ $0.148 < 1$ **o.k.**

Shear Rupture Strength of the Plate

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the plate is determined as follows:

$$
\begin{aligned}
A_{nv} &= \left[l - n\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t \\
&= \left[14.5\text{ in.} - 5\left(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right]\left(\tfrac{1}{2}\text{ in.}\right) \\
&= 4.75\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
R_n &= 0.60F_u A_{nv} \\
&= 0.60(65\text{ ksi})\left(4.75\text{ in.}^2\right) \\
&= 185\text{ kips}
\end{aligned}
$$

(*Spec*. Eq. J4-4)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(185\text{ kips})$ $= 139\text{ kips} > 75\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{185\text{ kips}}{2.00}$ $= 92.5\text{ kips} > 50\text{ kips}$ **o.k.**

Tensile Rupture of the Plate

From AISC *Specification* Section J4.1(b), the available tensile rupture strength of the plate is determined as follows:

$$\begin{aligned}A_n &= \left[l - n\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right] t\\ &= \left[14.5\text{ in.} - 5\left(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right]\left(\tfrac{1}{2}\text{ in.}\right)\\ &= 4.75\text{ in.}^2\end{aligned}$$

Table D3.1, Case 1, applies in this case because the tension load is transmitted directly to the cross-sectional element by fasteners; therefore, $U = 1.0$.

$$\begin{aligned}A_e &= A_n U\\ &= \left(4.75\text{ in.}^2\right)(1.0)\\ &= 4.75\text{ in.}^2\end{aligned}$$

(*Spec*. Eq. D3-1)

$$\begin{aligned}R_n &= F_u A_e\\ &= (65\text{ ksi})\left(4.75\text{ in.}^2\right)\\ &= 309\text{ kips}\end{aligned}$$

(*Spec*. Eq. J4-2)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(309\text{ kips})$ $= 232\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{309\text{ kips}}{2.00}$ $= 155\text{ kips} > 40\text{ kips}$ **o.k.**

Flexural Rupture of the Plate

The available flexural rupture strength of the plate is determined as follows:

$$\begin{aligned}Z_{net} &= Z_g - \frac{t_p}{4}\left[\left(d_h + \tfrac{1}{16}\text{ in.}\right)(s)\left(n^2 - 1\right) + \left(d_h + \tfrac{1}{16}\text{ in.}\right)^2\right]\\ &= 26.3\text{ in.}^3 - \frac{\tfrac{1}{2}\text{ in.}}{4}\left[\left(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)(3.00\text{ in.})\left(5^2 - 1\right) + \left(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)^2\right]\\ &= 17.2\text{ in.}^3\end{aligned}$$

$$\begin{aligned}M_n &= F_u Z_{net}\\ &= (65\text{ ksi})\left(17.2\text{ in.}^3\right)\\ &= 1{,}120\text{ kip-in.}\end{aligned}$$

(*Manual* Eq. 9-4)

LRFD	ASD
$\phi = 0.75$ $\phi M_n = 0.75(1{,}120\text{ kip-in.})$ $= 840\text{ kip-in.} > 93.8\text{ kip-in.}$ **o.k.**	$\Omega = 2.00$ $\dfrac{M_n}{\Omega} = \dfrac{1{,}120\text{ kip-in.}}{2.00}$ $= 560\text{ kip-in.} > 62.5\text{ kip-in.}$ **o.k.**

Interaction of Axial, Flexure and Shear Rupture in Plate

AISC *Specification* Chapter H does not address combined flexure and shear. The method employed here is derived from Chapter H in conjuction with AISC *Manual* Equation 10-5.

LRFD	ASD
$\dfrac{N_u}{\phi R_{np}} = \dfrac{60\text{ kips}}{232\text{ kips}}$ $= 0.259$ Because $\dfrac{N_u}{\phi R_{np}} > 0.2$: $\left(\dfrac{N_u}{\phi R_{np}} + \dfrac{8}{9}\dfrac{M_u}{\phi M_n}\right)^2 + \left(\dfrac{V_u}{\phi R_{nv}}\right)^2 \le 1$ $\left[\dfrac{60\text{ kips}}{232\text{ kips}} + \dfrac{8}{9}\left(\dfrac{93.8\text{ kip-in.}}{840\text{ kip-in.}}\right)\right]^2 + \left(\dfrac{75\text{ kips}}{139\text{ kips}}\right)^2 \le 1$ $0.419 < 1$ **o.k.**	$\dfrac{N_a}{R_{np}/\Omega} = \dfrac{40\text{ kips}}{155\text{ kips}}$ $= 0.258$ Because $\dfrac{N_a}{R_{np}/\Omega} > 0.2$: $\left(\dfrac{\Omega N_a}{R_{np}} + \dfrac{8}{9}\dfrac{\Omega M_a}{M_n}\right)^2 + \left(\dfrac{\Omega V_a}{R_{nv}}\right)^2 \le 1$ $\left[\dfrac{40\text{ kips}}{155\text{ kips}} + \dfrac{8}{9}\left(\dfrac{62.5\text{ kip-in.}}{560\text{ kip-in.}}\right)\right]^2 + \left(\dfrac{50\text{ kips}}{92.5\text{ kips}}\right)^2 \le 1$ $0.420 < 1$ **o.k.**

Block Shear Rupture Strength of the Plate—Beam Shear Direction

The nominal strength for the limit state of block shear rupture of the angles, assuming an L-shaped tearout due the shear load only, is determined as follows:

$$R_n = 0.60F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.60F_y A_{gv} + U_{bs} F_u A_{nt} \qquad (\textit{Spec}.\text{ Eq. J4-5})$$

where

$$A_{gv} = (l - l_{ev})t$$
$$= [14.5\text{ in.} - 1\tfrac{1}{4}\text{ in.}](\tfrac{1}{2}\text{ in.})$$
$$= 6.63\text{ in.}^2$$

$$A_{nv} = A_{gv} - (n - 0.5)(d_h + \tfrac{1}{16}\text{ in.})t$$
$$= 6.63\text{ in.}^2 - (5 - 0.5)(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})(\tfrac{1}{2}\text{ in.})$$
$$= 4.38\text{ in.}^2$$

$$A_{nt} = [l_{eh} - 0.5(d_h + \tfrac{1}{16}\text{ in.})]t$$
$$= [2\tfrac{1}{2}\text{ in.} - 0.5(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})](\tfrac{1}{2}\text{ in.})$$
$$= 1.00\text{ in.}^2$$

$$U_{bs} = 1.0$$

and

$$R_n = 0.60\left(65\text{ ksi}\right)\left(4.38\text{ in.}^2\right) + 1.0\left(65\text{ ksi}\right)\left(1.00\text{ in.}^2\right) \le 0.60\left(50\text{ ksi}\right)\left(6.63\text{ in.}^2\right) + 1.0\left(65\text{ ksi}\right)\left(1.00\text{ in.}^2\right)$$
$$= 236\text{ kips} < 264\text{ kips}$$

Therefore:

$$R_n = 236\text{ kips}$$

From AISC *Specification* Section J4.3, the available strength for the limit state of block shear rupture on the plate is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(236\text{ kips}\right)$ $= 177\text{ kips} > 75\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{236\text{ kips}}{2.00}$ $= 118\text{ kips} > 50\text{ kips}$ **o.k.**

Block Shear Rupture Strength of the Plate—Beam Axial Direction

The plate block shear rupture failure path due to axial load only could occur as an L- or U-shape. Assuming an L-shaped failure path due to axial load only, the available block shear rupture strength of the plate is:

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad (\textit{Spec.}\text{ Eq. J4-5})$$

where

$$A_{gv} = l_{eh}t$$
$$= \left(2\tfrac{1}{2}\text{ in.}\right)\left(\tfrac{1}{2}\text{ in.}\right)$$
$$= 1.25\text{ in.}^2$$

$$A_{nv} = A_{gv} - 0.5\left(d_h + \tfrac{1}{16}\text{ in.}\right)t$$
$$= 1.25\text{ in.}^2 - 0.5\left(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\left(\tfrac{1}{2}\text{ in.}\right)$$
$$= 1.00\text{ in.}^2$$

$$A_{nt} = \left[l - l_{ev} - \left(n - 0.5\right)\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t$$
$$= \left[14.5\text{ in.} - 1\tfrac{1}{4}\text{ in.} - \left(5 - 0.5\right)\left(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right]\left(\tfrac{1}{2}\text{ in.}\right)$$
$$= 4.38\text{ in.}^2$$

$$U_{bs} = 1.0$$

and

$$R_n = 0.60\left(65\text{ ksi}\right)\left(1.00\text{ in.}^2\right) + 1.0\left(65\text{ ksi}\right)\left(4.38\text{ in.}^2\right) \le 0.60\left(50\text{ ksi}\right)\left(1.25\text{ in.}^2\right) + 1.0\left(65\text{ ksi}\right)\left(4.38\text{ in.}^2\right)$$
$$= 324\text{ kips} > 322\text{ kips}$$

Therefore:

$$R_n = 322\text{ kips}$$

From AISC *Specification* Section J4.3, the available strength for the limit state of block shear rupture on the plate is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(322\text{ kips})$ $= 242\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{322\text{ kips}}{2.00}$ $= 161\text{ kips} > 40\text{ kips}$ **o.k.**

Assuming a U-shaped failure path in the plate due to axial load, the available block shear rupture strength of the plate is:

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad \text{(\textit{Spec.} Eq. J4-5)}$$

where

$$\begin{aligned} A_{gv} &= (2\text{ shear planes})l_{eh}t_p \\ &= (2\text{ shear planes})(2\tfrac{1}{2}\text{ in.})(\tfrac{1}{2}\text{ in.}) \\ &= 2.50\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - (2\text{ shear planes})(0.5)(d_h + \tfrac{1}{16}\text{ in.})t \\ &= 2.50\text{ in.}^2 - (2\text{ shear planes})(0.5)(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})(\tfrac{1}{2}\text{ in.}) \\ &= 2.00\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= \left[l - 2l_{ev} - (n-1)(d_h + \tfrac{1}{16}\text{ in.})\right]t \\ &= \left[14.5\text{ in.} - 2(1\tfrac{1}{4}\text{ in.}) - (5-1)(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})\right](\tfrac{1}{2}\text{ in.}) \\ &= 4.00\text{ in.}^2 \end{aligned}$$

$$U_{bs} = 1.0$$

and

$$\begin{aligned} R_n &= 0.60(65\text{ ksi})(2.00\text{ in.}^2) + 1.0(65\text{ ksi})(4.00\text{ in.}^2) \le 0.60(50\text{ ksi})(2.50\text{ in.}^2) + 1.0(65\text{ ksi})(4.00\text{ in.}^2) \\ &= 338\text{ kips} > 335\text{ kips} \end{aligned}$$

Therefore:

$$R_n = 335\text{ kips}$$

From AISC *Specification* Section J4.3, the available strength for the limit state of block shear rupture on the plate is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(335\text{ kips})$ $= 251\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{335\text{ kips}}{2.00}$ $= 168\text{ kips} > 40\text{ kips}$ **o.k.**

The L-shaped failure path controls in the shear plate.

Check shear and tension interaction for plate block shear on the L-shaped failure plane:

LRFD	ASD
$\left(\dfrac{V_u}{\phi R_{nv}}\right)^2+\left(\dfrac{N_u}{\phi R_{nt}}\right)^2\le 1$ $\left(\dfrac{75\text{ kips}}{177\text{ kips}}\right)^2+\left(\dfrac{60\text{ kips}}{242\text{ kips}}\right)^2=0.241<1$ **o.k.**	$\left(\dfrac{V_a}{R_{nv}/\Omega}\right)^2+\left(\dfrac{N_a}{R_{nt}/\Omega}\right)^2\le 1$ $\left(\dfrac{50\text{ kips}}{118\text{ kips}}\right)^2+\left(\dfrac{40\text{ kips}}{161\text{ kips}}\right)^2=0.241<1$ **o.k.**

Shear Strength of the Beam Web

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the beam is determined as follows:

$$\begin{aligned}A_{gv}&=dt_w\\&=(18.0\text{ in.})(0.355\text{ in.})\\&=6.39\text{ in.}^2\end{aligned}$$

$$\begin{aligned}R_n&=0.60F_yA_{gv}\\&=0.60(50\text{ ksi})(6.39\text{ in.}^2)\\&=192\text{ kips}\end{aligned}$$ (*Spec*. Eq. J4-3)

LRFD	ASD
$\phi=1.00$ $\phi R_n=1.00(192\text{ kips})$ $=192\text{ kips}>75\text{ kips}$ **o.k.**	$\Omega=1.50$ $\dfrac{R_n}{\Omega}=\dfrac{192\text{ kips}}{1.50}$ $=128\text{ kips}>50\text{ kips}$ **o.k.**

The limit state of shear rupture of the beam web will not control in this example because the beam is uncoped.

Tensile Strength of the Beam

From AISC *Specification* Section J4.1(a), the available tensile yielding strength of the beam is determined as follows:

$$\begin{aligned}R_n&=F_yA_g\\&=(50\text{ ksi})(14.7\text{ in.}^2)\\&=735\text{ kips}\end{aligned}$$ (*Spec*. Eq. J4-1)

LRFD	ASD
$\phi=0.90$ $\phi R_n=0.90(735\text{ kips})$ $=662\text{ kips}>60\text{ kips}$ **o.k.**	$\Omega=1.67$ $\dfrac{R_n}{\Omega}=\dfrac{735\text{ kips}}{1.67}$ $=440\text{ kips}>40\text{ kips}$ **o.k.**

From AISC *Specification* Sections J4.1, the available tensile rupture strength of the beam is determined from AISC *Specification* Equation J4-2. No cases in Table D3.1 apply to this configuration; therefore, U is determined in accordance with AISC *Specification* Section D3, where the minimum value of U is the ratio of the gross area of the connected element to the member gross area.

$$
\begin{aligned}
U &= \frac{\left(d-2t_f\right)t_w}{A_g} \\
&= \frac{\left[18.0\text{ in.}-2\left(0.570\text{ in.}\right)\right]\left(0.355\text{ in.}\right)}{14.7\text{ in.}^2} \\
&= 0.407
\end{aligned}
$$

$$
\begin{aligned}
A_n &= A_g - n\left(d_h + {}^1\!/_{16}\text{ in.}\right)t_w \\
&= 14.7\text{ in.}^2 - 5\left({}^{15}\!/_{16}\text{ in.} + {}^1\!/_{16}\text{ in.}\right)\left(0.355\text{ in.}\right) \\
&= 12.9\text{ in.}
\end{aligned}
$$

$$
\begin{aligned}
A_e &= A_nU \\
&= \left(12.9\text{ in.}^2\right)\left(0.407\right) \\
&= 5.25\text{ in.}^2
\end{aligned}
$$ (*Spec.* Eq. D3-1)

$$
\begin{aligned}
R_n &= F_uA_e \\
&= \left(65\text{ ksi}\right)\left(5.25\text{ in.}^2\right) \\
&= 341\text{ kips}
\end{aligned}
$$ (*Spec.* Eq. J4-2)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(341\text{ kips}\right)$ $= 256\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{341\text{ kips}}{2.00}$ $= 171\text{ kips} > 40\text{ kips}$ **o.k.**

Block Shear Rupture of the Beam Web

Block shear rupture is only applicable in the direction of the axial load, because the beam is uncoped and the limit state is not applicable for an uncoped beam subject to vertical shear. Assuming a U-shaped tearout relative to the axial load, and assuming a horizontal edge distance of l_{eh} = 1¾ in. – ¼ in. = 1½ in. to account for a possible beam underrun of ¼ in., the block shear rupture strength is:

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.60F_yA_{gv} + U_{bs}F_uA_{nt}$$ (*Spec.* Eq. J4-5)

where

$$
\begin{aligned}
A_{gv} &= \left(2\text{ shear planes}\right)l_{eh}t_w \\
&= \left(2\text{ shear planes}\right)\left(1\tfrac{1}{2}\text{ in.}\right)\left(0.355\text{ in.}\right) \\
&= 1.07\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
A_{nv} &= A_{gv} - \left(2\text{ shear planes}\right)\left(0.5\right)\left(d_h + {}^1\!/_{16}\text{ in.}\right)t_w \\
&= 1.07\text{ in.}^2 - \left(2\text{ shear planes}\right)\left(0.5\right)\left({}^{15}\!/_{16}\text{ in.} + {}^1\!/_{16}\text{ in.}\right)\left(0.355\text{ in.}\right) \\
&= 0.715\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
A_{nt} &= \left[12.0\text{ in.} - (n-1)\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t_w \\
&= \left[12.0\text{ in.} - (5-1)\left(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right](0.355\text{ in.}) \\
&= 2.84\text{ in.}^2
\end{aligned}
$$

$$U_{bs} = 1.0$$

and

$$
\begin{aligned}
R_n &= 0.60(65\text{ ksi})\left(0.715\text{ in.}^2\right) + 1.0(65\text{ ksi})\left(2.84\text{ in.}^2\right) \le 0.60(50\text{ ksi})\left(1.07\text{ in.}^2\right) + 1.0(65\text{ ksi})\left(2.84\text{ in.}^2\right) \\
&= 212\text{ kips} < 217\text{ kips}
\end{aligned}
$$

Therefore:

$$R_n = 212\text{ kips}$$

From AISC *Specification* Section J4.3, the available strength for the limit state of block shear rupture of the beam web is:

LRFD	ASD
$\phi R_n = 0.75(212\text{ kips})$ $= 159\text{ kips} > 60\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{212\text{ kips}}{2.00}$ $= 106\text{ kips} > 40\text{ kips}$ **o.k.**

Conclusion

The connection is found to be adequate as given for the applied loads. Note that because the supported member was assumed to be continuously laterally braced, it is not necessary to check weak-axis moment.

EXAMPLE II.A-17C SINGLE-PLATE CONNECTION—STRUCTURAL INTEGRITY CHECK

Given:

Verify the single plate connection from Example II.A-17A, as shown in Figure II.A-17C-1, for the structural integrity provisions of AISC *Specification* Section B3.9. The connection is verified as a beam and girder end connection and as an end connection of a member bracing a column. Note that these checks are necessary when design for structural integrity is required by the applicable building code.

Use 70-ksi electrodes and an ASTM A36 plate.

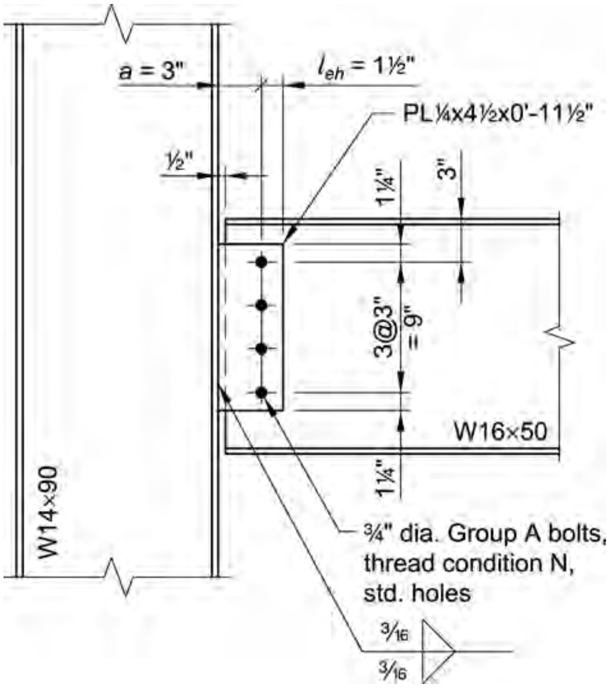

Fig. II.A-17C-1. Connection geometry for Example II.A-17C.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Plate
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W16x50
$t_w = 0.380$ in.

From AISC *Specification* Table J3.3, the hole diameter for ¾-in.-diameter bolts with standard holes is:

$d_h = \frac{13}{16}$ in.

Beam and Girder End Connection

From Example II.A-17A, the required shear strength is:

LRFD	ASD
$V_u = 49.6 \text{ kips}$	$V_a = 33.0 \text{ kips}$

From AISC *Specification* Section B3.9(b), the required axial tensile strength is:

LRFD	ASD
$T_u = \frac{2}{3}V_u \geq 10 \text{ kips}$ $= \frac{2}{3}(49.6 \text{ kips}) > 10 \text{ kips}$ $= 33.1 \text{ kips} > 10 \text{ kips}$ $= 33.1 \text{ kips}$	$T_a = V_a \geq 10 \text{ kips}$ $= 33.0 \text{ kips} > 10 \text{ kips}$ $= 33.0 \text{ kips}$

Bolt Shear

From AISC Specification Section J3.6, the nominal bolt shear strength is:

$F_{nv} = 54$ ksi, from AISC *Specification* Table J3.2

$$\begin{aligned} T_n &= nF_{nv}A_b \\ &= (4 \text{ bolts})(54 \text{ ksi})(0.442 \text{ in.}^2) \\ &= 95.5 \text{ kips} \end{aligned}$$ (from *Spec*. Eq. J3-1)

Bolt Bearing and Tearout

From AISC *Specification* Section B3.9, for the purpose of satisfying structural integrity requirements inelastic deformations of the connection are permitted; therefore, AISC *Specification* Equations J3-6b and J3-6d are used to determine the nominal bearing and tearout strength. By inspection, bolt bearing and tearout will control for the plate. For bolt bearing on the plate:

$$\begin{aligned} T_n &= (4 \text{ bolts})3.0dtF_u \\ &= (4 \text{ bolts})(3.0)(\tfrac{3}{4} \text{ in.})(\tfrac{1}{4} \text{ in.})(58 \text{ ksi}) \\ &= 131 \text{ kips} \end{aligned}$$ (from *Spec*. Eq. J3-6b)

For bolt tearout on the plate:

$$\begin{aligned} l_c &= l_{eh} - 0.5\left(d_h + \tfrac{1}{16}\text{ in.}\right) \\ &= 1\tfrac{1}{2}\text{ in.} - 0.5\left(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right) \\ &= 1.06\text{ in.} \end{aligned}$$

$$\begin{aligned} T_n &= (4\text{ bolts})1.5 l_c t F_u \\ &= (4\text{ bolts})(1.5)(1.06\text{ in.})(\tfrac{1}{4}\text{ in.})(58\text{ ksi}) \\ &= 92.2\text{ kips} \end{aligned}$$

(from *Spec*. Eq. J3-6d)

Tensile Yielding of Plate

From AISC *Specification* Section J4.1, the nominal tensile yielding strength of the shear plate is determined as follows:

$$\begin{aligned} A_g &= lt \\ &= (11.5\text{ in.})(\tfrac{1}{4}\text{ in.}) \\ &= 2.88\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} T_n &= F_y A_g \\ &= (36\text{ ksi})\left(2.88\text{ in.}^2\right) \\ &= 104\text{ kips} \end{aligned}$$

(from *Spec*. Eq. J4-1)

Tensile Rupture of Plate

From AISC *Specification* Section J4.1, the nominal tensile rupture strength of the shear plate is determined as follows:

$$\begin{aligned} A_n &= \left[l - n\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t \\ &= \left[11.5\text{ in.} - (4\text{ bolts})\left(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right](\tfrac{1}{4}\text{ in.}) \\ &= 2.00\text{ in.}^2 \end{aligned}$$

AISC *Specification* Table D3.1, Case 1 applies in this case because tension load is transmitted directly to the cross-section element by fasteners; therefore, $U = 1.0$.

$$\begin{aligned} A_e &= A_n U \\ &= \left(2.00\text{ in.}^2\right)(1.0) \\ &= 2.00\text{ in.}^2 \end{aligned}$$

(*Spec*. Eq. D3-1)

$$\begin{aligned} T_n &= F_u A_e \\ &= (58\text{ ksi})\left(2.00\text{ in.}^2\right) \\ &= 116\text{ kips} \end{aligned}$$

(from *Spec*. Eq. J4-2)

Block Shear Rupture—Plate

From AISC *Specification* Section J4.3, the nominal block shear rupture strength, due to axial load, of the shear plate is determined as follows:

$$T_n = 0.60F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.60F_y A_{gv} + U_{bs} F_u A_{nt} \qquad \text{(from } \textit{Spec.} \text{ Eq. J4-5)}$$

where

$$\begin{aligned} A_{gv} &= (2 \text{ shear planes}) l_{eh} t \\ &= (2 \text{ shear planes})(1\tfrac{1}{2} \text{ in.})(\tfrac{1}{4} \text{ in.}) \\ &= 0.750 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= (2 \text{ shear planes})\left[l_{eh} - 0.5(d_h + \tfrac{1}{16} \text{ in.})\right] t_p \\ &= (2 \text{ shear planes})\left[1\tfrac{1}{2} \text{ in.} - 0.5(\tfrac{13}{16} \text{ in.} + \tfrac{1}{16} \text{ in.})\right](\tfrac{1}{4} \text{ in.}) \\ &= 0.531 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= \left[l - 2l_{ev} - (n-1)(d_h + \tfrac{1}{16} \text{ in.})\right] t \\ &= \left[11.5 \text{ in.} - 2(1\tfrac{1}{4} \text{ in.}) - (4-1)(\tfrac{13}{16} \text{ in.} + \tfrac{1}{16} \text{ in.})\right](\tfrac{1}{4} \text{ in.}) \\ &= 1.59 \text{ in.}^2 \end{aligned}$$

$$U_{bs} = 1.0$$

and

$$\begin{aligned} T_n &= 0.60(58 \text{ ksi})(0.531 \text{ in.}^2) + 1.0(58 \text{ ksi})(1.59 \text{ in.}^2) \le 0.60(36 \text{ ksi})(0.750 \text{ in.}^2) + 1.0(58 \text{ ksi})(1.59 \text{ in.}^2) \\ &= 111 \text{ kips} > 108 \text{ kips} \\ &= 108 \text{ kips} \end{aligned}$$

Block Shear Rupture—Beam Web

From AISC *Specification* Section J4.3, the nominal block shear rupture strength, due to axial load, of the beam web is determined as follows (accounting for a possible ¼-in. beam underrun):

$$T_n = 0.60F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.60F_y A_{gv} + U_{bs} F_u A_{nt} \qquad (\textit{Spec.} \text{ Eq. J4-5})$$

where

$$\begin{aligned} A_{gv} &= (2 \text{ shear planes})(l_{eh} - underrun) t_w \\ &= (2 \text{ shear planes})(2\tfrac{1}{2} \text{ in.} - \tfrac{1}{4} \text{ in.})(0.380 \text{ in.}) \\ &= 1.71 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= (2 \text{ shear planes})\left[l_{eh} - underrun - 0.5(d_h + \tfrac{1}{16} \text{ in.})\right] t_w \\ &= (2 \text{ shear planes})\left[2\tfrac{1}{2} \text{ in.} - \tfrac{1}{4} \text{ in.} - 0.5(\tfrac{13}{16} \text{ in.} + \tfrac{1}{16} \text{ in.})\right](0.380 \text{ in.}) \\ &= 1.38 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= \left[9.00 \text{ in.} - 3(\tfrac{13}{16} \text{ in.} + \tfrac{1}{16} \text{ in.})\right](0.380 \text{ in.}) \\ &= 2.42 \text{ in.}^2 \end{aligned}$$

$$U_{bs} = 1.0$$

and

$$
\begin{aligned}
T_n &= 0.60(65\text{ ksi})(1.38\text{ in.}^2)+1.0(65\text{ ksi})(2.42\text{ in.}^2) \le 0.60(50\text{ ksi})(1.71\text{ in.}^2)+1.0(65\text{ ksi})(2.42\text{ in.}^2)\\
&= 211\text{ kips} > 209\text{ kips}\\
&= 209\text{ kips}
\end{aligned}
$$

Weld Strength

From AISC *Specification* Section J2.4, the nominal tensile strength of the weld is determined as follows:

$$
\begin{aligned}
F_{nw} &= 0.60F_{EXX}\left(1.0+0.50\sin^{1.5}\theta\right) &&(\textit{Spec}.\text{ Eq. J2-5})\\
&= 0.60(70\text{ ksi})\left(1.0+0.50\sin^{1.5}90^\circ\right)\\
&= 63.0\text{ ksi}
\end{aligned}
$$

The throat dimension is used to calculate the effective area of the fillet weld.

$$
\begin{aligned}
A_{we} &= \frac{w}{\sqrt{2}}l(2\text{ welds})\\
&= \frac{3/16\text{ in.}}{\sqrt{2}}(11.5\text{ in.})(2\text{ welds})\\
&= 3.05\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
T_n &= F_{nw}A_{we} &&(\text{from }\textit{Spec}.\text{ Eq. J2-4})\\
&= (63.0\text{ ksi})(3.05\text{ in.}^2)\\
&= 192\text{ kips}
\end{aligned}
$$

Nominal Tensile Strength

The controlling tensile strength, T_n, is the least of those previously calculated:

$$
\begin{aligned}
T_n &= \min\{95.5\text{ kips},\ 131\text{ kips},\ 92.2\text{ kips},\ 104\text{ kips},\ 116\text{ kips},\ 108\text{ kips},\ 209\text{ kips},\ 192\text{ kips}\}\\
&= 92.2\text{ kips}
\end{aligned}
$$

LRFD	ASD
$T_n = 92.2\text{ kips} > 33.1\text{ kips}$ **o.k.**	$T_n = 92.2\text{ kips} > 33.0\text{ kips}$ **o.k.**

Column Bracing

From AISC *Specification* Section B3.9(c), the minimum axial tension strength for the connection of a member bracing a column is equal to 1% of two-thirds of the required column axial strength for LRFD and equal to 1% of the required column axial for ASD. These requirements are evaluated independently from other strength requirements.

The maximum column axial force this connection is able to brace is determined as follows:

LRFD	ASD
$T_n \ge 0.01\left(\frac{2}{3}P_u\right)$	$T_n \ge 0.01P_a$

LRFD	ASD
Solving for the column axial force: $P_u \le 100\left(\frac{3}{2}T_n\right)$ $= 100\left(\frac{3}{2}\right)(92.2\text{ kips})$ $= 13{,}800\text{ kips}$	Solving for the column axial force: $P_a \le 100T_n$ $= 100(92.2\text{ kips})$ $= 9{,}220\text{ kips}$

As long as the required column axial strength is less than P_u = 13,800 kips or P_a = 9,220 kips, this connection is an adequate column brace.

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EXAMPLE II.A-18 SINGLE-PLATE CONNECTION (BEAM-TO-GIRDER WEB)

Given:

Verify a single-plate connection between an ASTM A992 W18×35 beam and an ASTM A992 W21×62 girder web, as shown in Figure II.A-18-1, to support the following beam end reactions:

$R_D = 6.5$ kips
$R_L = 20$ kips

The top flange is coped 2 in. deep by 4 in. long, $l_{ev} = 1\frac{1}{2}$ in. Use 70-ksi electrodes and an ASTM A36 plate.

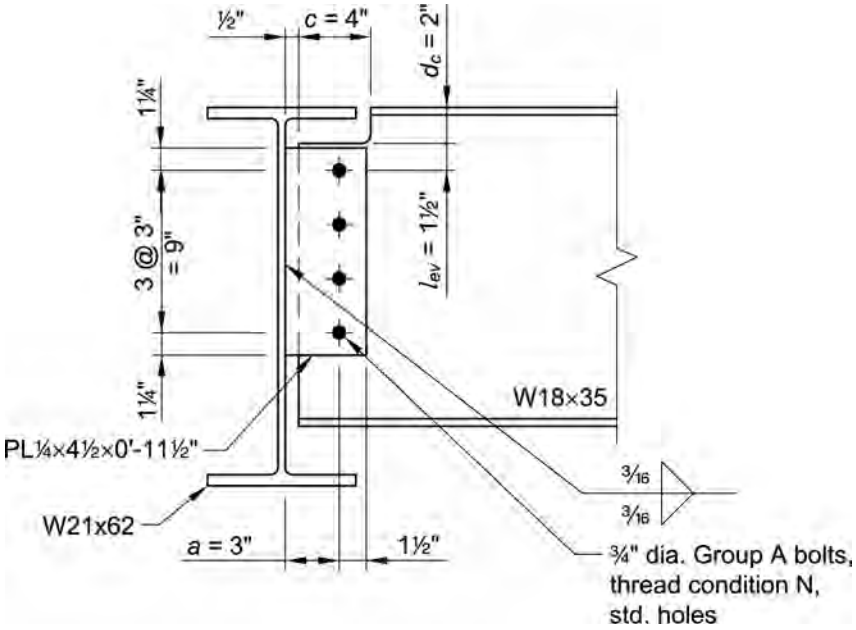

Fig. II.A-18-1. Connection geometry for Example II.A-18.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Beam and girder
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Plate
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W18×35
t_w = 0.300 in.
d = 17.7 in.
t_f = 0.425 in.

Girder
W21×62
t_w = 0.400 in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(6.5\text{ kips}) + 1.6(20\text{ kips})$ $= 39.8\text{ kips}$	$R_a = 6.5\text{ kips} + 20\text{ kips}$ $= 26.5\text{ kips}$

Connection Selection

AISC *Manual* Table 10-10a includes checks for the limit states of bolt shear, bolt bearing on the plate, tearout on the plate, shear yielding of the plate, shear rupture of the plate, block shear rupture of the plate and weld shear.

Use four rows of bolts, ¼-in. plate thickness, and 3⁄16-in. fillet weld size. From AISC *Manual* Table 10-10a:

LRFD	ASD
$\phi R_n = 52.2\text{ kips} > 39.8\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 34.8\text{ kips} > 26.5\text{ kips}$ **o.k.**

Block Shear Rupture of Beam Web

The nominal strength for the limit state of block shear rupture is given by AISC *Specification* Section J4.3.

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad (\textit{Spec}.\text{ Eq. J4-5})$$

The available block shear rupture strength of the beam web is determined as follows, using AISC *Manual* Tables 9-3a, 9-3b and 9-3c and AISC *Specification* Equation J4-5, with n = 4, l_{eh} = 2¼ in. (reduced ¼ in. to account for beam underrun), l_{ev} = 1½ in. and U_{bs} = 1.0.

LRFD	ASD
Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{\phi F_uA_{nt}}{t} = 88.4\text{ kip/in.}$ Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{\phi 0.60F_yA_{gv}}{t} = 236\text{ kip/in.}$	Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{F_uA_{nt}}{\Omega t} = 58.9\text{ kip/in.}$ Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{0.60F_yA_{gv}}{\Omega t} = 158\text{ kip/in.}$

LRFD	ASD
Shear rupture component from AISC *Manual* Table 9-3c:	Shear rupture component from AISC *Manual* Table 9-3c:
$\dfrac{\phi 0.60 F_u A_{nv}}{t} = 218 \text{ kip/in.}$	$\dfrac{0.60 F_u A_{nv}}{\Omega t} = 145 \text{ kip/in.}$
The design block shear rupture strength is:	The allowable block shear rupture strength is:
$\phi R_n = \phi 0.60 F_u A_{nv} + \phi U_{bs} F_u A_{nt}$ $\leq \phi 0.60 F_y A_{gv} + \phi U_{bs} F_u A_{nt}$ $= (218 \text{ kip/in.} + 88.4 \text{ kip/in.})(0.300 \text{ in.})$ $\leq (236 \text{ kip/in.} + 88.4 \text{ kip/in.})(0.300 \text{ in.})$ $= 91.9 \text{ kips} < 97.3 \text{ kips}$	$\dfrac{R_n}{\Omega} = \dfrac{0.60 F_u A_{nv}}{\Omega} + \dfrac{U_{bs} F_u A_{nt}}{\Omega}$ $\leq \dfrac{0.60 F_y A_{gv}}{\Omega} + \dfrac{U_{bs} F_u A_{nt}}{\Omega}$ $= (145 \text{ kip/in.} + 58.9 \text{ kip/in.})(0.300 \text{ in.})$ $\leq (158 \text{ kip/in.} + 58.9 \text{ kip/in.})(0.300 \text{ in.})$ $= 61.2 \text{ kips} < 65.1 \text{ kips}$
Therefore:	Therefore:
$\phi R_n = 91.9 \text{ kips} > 39.8 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 61.2 \text{ kips} > 26.5 \text{ kips}$ **o.k.**

Strength of the Bolted Connection—Beam Web

From the Commentary to AISC *Specification* Section J3.6, the strength of the bolt group is taken as the sum of the individual strengths of the individual fasteners, which may be taken as the lesser of the fastener shear strength per AISC *Specification* Section J3.6, the bearing strength at the bolt hole per AISC *Specification* Section J3.10, or the tearout strength at the bolt hole per AISC *Specification* Section J3.10.

From AISC *Manual* Table 7-1, the available shear strength per bolt for ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) is:

LRFD	ASD
$\phi r_n = 17.9 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 11.9 \text{ kips/bolt}$

The available bearing and tearout strength of the beam web edge bolt (top bolt shown in Figure II.A-18-1) is determined using AISC *Manual* Table 7-5, conservatively using $l_e = 1\frac{1}{4}$ in.

LRFD	ASD
$\phi r_n = (49.4 \text{ kip/in.})(0.300 \text{ in.})$ $= 14.8 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (32.9 \text{ kip/in.})(0.300 \text{ in.})$ $= 9.87 \text{ kips/bolt}$

The bearing or tearout strength controls over bolt shear for the edge bolt.

The available bearing and tearout strength of the beam web at the interior bolts is determined using AISC *Manual* Table 7-4 with s = 3 in.

LRFD	ASD
$\phi r_n = (87.8 \text{ kip/in.})(0.300 \text{ in.})$ $= 26.3 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (58.5 \text{ kip/in.})(0.300 \text{ in.})$ $= 17.6 \text{ kips/bolt}$

Bolt shear strength controls for the interior bolts.

The strength of the bolt group in the beam web is determined by summing the strength of the individual fasteners as follows:

LRFD	ASD
$\phi R_n = (1 \text{ bolt})(14.8 \text{ kips/bolt})$ $+ (3 \text{ bolts})(17.9 \text{ kips/bolt})$ $= 68.5 \text{ kips} > 39.8 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = (1 \text{ bolt})(9.87 \text{ kips/bolt})$ $+ (3 \text{ bolts})(11.9 \text{ kips/bolt})$ $= 45.6 \text{ kips} > 26.5 \text{ kips}$ **o.k.**

Strength of the Bolted Connection—Single Plate

The available bearing and tearout strength of the plate at the edge bolt (bottom bolt shown in Figure II.A-18-1) is determined using AISC *Manual* Table 7-5 with l_e = 1¼ in.

LRFD	ASD
$\phi r_n = (44.0 \text{ kip/in.})(\text{¼ in.})$ $= 11.0 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (29.4 \text{ kip/in.})(\text{¼ in.})$ $= 7.35 \text{ kips/bolt}$

The bearing or tearout strength controls over bolt shear for the edge bolt.

The available bearing and tearout strength of the plate at the interior bolts is determined using AISC *Manual* Table 7-4 with s = 3 in.

LRFD	ASD
$\phi r_n = (78.3 \text{ kip/in.})(\text{¼ in.})$ $= 19.6 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (52.2 \text{ kip/in.})(\text{¼ in.})$ $= 13.1 \text{ kips/bolt}$

Bolt shear strength controls for the interior bolts.

The strength of the bolt group in the plate is determined by summing the strength of the individual fasteners as follows:

LRFD	ASD
$\phi R_n = (1 \text{ bolt})(11.0 \text{ kips/bolt})$ $+ (3 \text{ bolts})(17.9 \text{ kips/bolt})$ $= 64.7 \text{ kips} > 39.8 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = (1 \text{ bolt})(7.35 \text{ kips/bolt})$ $+ (3 \text{ bolts})(11.9 \text{ kips/bolt})$ $= 43.1 \text{ kips} > 26.5 \text{ kips}$ **o.k.**

Shear Rupture of the Girder Web at the Weld

The minimum support thickness to match the shear rupture strength of the weld is determined as follows:

$$t_{min} = \frac{3.09D}{F_u} \qquad \text{(\textit{Manual} Eq. 9-2)}$$

$$= \frac{3.09(3 \text{ sixteenths})}{65 \text{ ksi}}$$

$$= 0.143 \text{ in.} < 0.400 \text{ in.} \quad \textbf{o.k.}$$

Note: For coped beam sections, the limit states of flexural yielding and local buckling should be checked independently per AISC *Manual* Part 9. The supported beam web should also be checked for shear yielding and shear rupture per AISC *Specification* Section J4.2. However, for the shallow cope in this example, these limit states do not govern. For an illustration of these checks, see Example II.A-4.

Conclusion

The connection is found to be adequate as given for the applied loads.

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EXAMPLE II.A-19A EXTENDED SINGLE-PLATE CONNECTION (BEAM-TO-COLUMN WEB)

Given:

Verify the connection between an ASTM A992 W16×36 beam and the web of an ASTM A992 W14×90 column, as shown in Figure II.A-19A-1, to support the following beam end reactions:

$R_D = 6$ kips
$R_L = 18$ kips

Use 70-ksi electrodes and ASTM A36 plate.

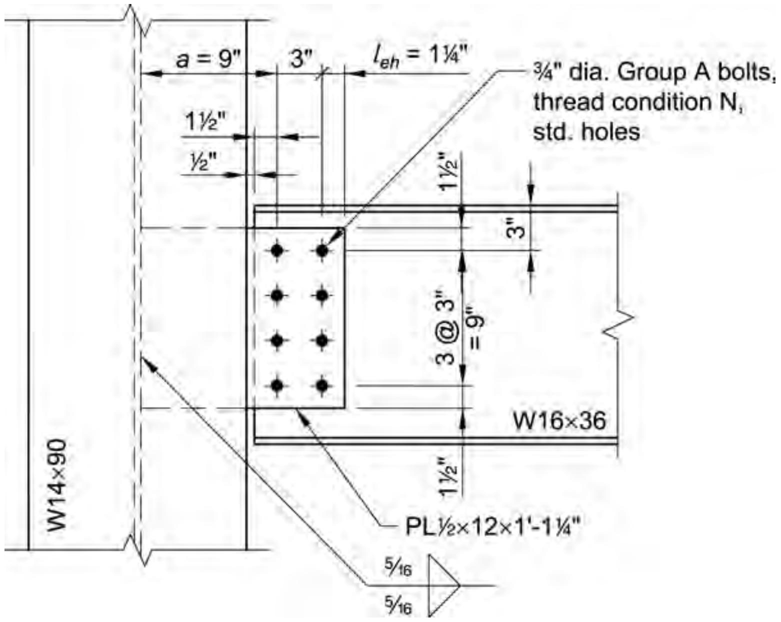

Fig. II.A-19A-1. Connection geometry for Example II.A-19A.

Note: All dimensional limitations are satisfied.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Beam and column
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Plate
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W16×36
t_w = 0.295 in.
d = 15.9 in.

Column
W14×90
t_w = 0.440 in.
b_f = 14.5 in.

From AISC *Specification* Table J3.3, the hole diameter for a ¾-in.-diameter bolt with standard holes is:

$$d_h = {}^{13}\!/_{16}\text{ in.}$$

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(6\text{ kips}) + 1.6(18\text{ kips})$ $= 36.0\text{ kips}$	$R_a = 6\text{ kips} + 18\text{ kips}$ $= 24.0\text{ kips}$

Strength of the Bolted Connection—Beam Web

From AISC *Manual* Part 10, determine the distance from the support to the first line of bolts and the distance to the center of gravity of the bolt group.

$$a = 9\text{ in.}$$

$$\begin{aligned} e &= 9\text{ in.} + \frac{3\text{ in.}}{2} \\ &= 10.5\text{ in.} \end{aligned}$$

From AISC *Manual* Table 7-1, the available shear strength per bolt for ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) is:

LRFD	ASD
$\phi r_n = 17.9\text{ kips}$	$\dfrac{r_n}{\Omega} = 11.9\text{ kips}$

Tearout for the bolts in the beam web does not control due to the presence of the beam top flange.

The available bearing strength of the beam web per bolt is determined using AISC *Manual* Table 7-4 with s = 3 in.

LRFD	ASD
$\phi r_n = (87.8\text{ kip/in.})(0.295\text{ in.})$ $= 25.9\text{ kips}$	$\dfrac{r_n}{\Omega} = (58.5\text{ kip/in.})(0.295\text{ in.})$ $= 17.3\text{ kips}$

Therefore, bolt shear controls over bearing.

The strength of the bolt group is determined by interpolating AISC *Manual* Table 7-7, with e = 10.5 in. and Angle = 0°:

$$C = 2.33$$

LRFD	ASD
$\phi R_n = C\phi r_n$ $= 2.33(17.9\text{ kips})$ $= 41.7\text{ kips} > 36.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{Cr_n}{\Omega}$ $= 2.33(11.9\text{ kips})$ $= 27.7\text{ kips} > 24.0\text{ kips}$ **o.k.**

Maximum Plate Thickness

From AISC *Manual* Part 10, determine the maximum plate thickness, t_{max}, that will result in the plate yielding before the bolts shear.

F_{nv} = 54 ksi from AISC *Specification* Table J3.2

C' = 26.0 in. from AISC *Manual* Table 7-7 for the moment-only case (Angle = 0°)

$$\begin{aligned} M_{max} &= \frac{F_{nv}}{0.90}(A_b C') \\ &= \left(\frac{54\text{ ksi}}{0.90}\right)(0.442\text{ in.}^2)(26.0\text{ in.}) \\ &= 690\text{ kip-in.} \end{aligned}$$

(*Manual* Eq. 10-4)

$$\begin{aligned} t_{max} &= \frac{6M_{max}}{F_y l^2} \\ &= \frac{6(690\text{ kip-in.})}{(36\text{ ksi})(12.0\text{ in.})^2} \\ &= 0.799\text{ in.} \end{aligned}$$

(*Manual* Eq. 10-3)

Try a plate thickness of ½ in.

Strength of the Bolted Connection—Plate

The available bearing strength of the plate per bolt is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration, as follows:

$$\begin{aligned} r_n &= 2.4dtF_u \\ &= 2.4(\tfrac{3}{4}\text{ in.})(\tfrac{1}{2}\text{ in.})(58\text{ ksi}) \\ &= 52.2\text{ kips/bolt} \end{aligned}$$

(*Spec.* Eq. J3-6a)

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(52.2\text{ kips/bolt})$ $= 39.2\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{52.2\text{ kips/bolt}}{2.00}$ $= 26.1\text{ kips/bolt}$

The available tearout strength of the bottom edge bolt in the plate is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration, as follows:

$$\begin{aligned} l_c &= l_{eh} - 0.5d_h \\ &= 1\tfrac{1}{2}\text{ in.} - 0.5\left(\tfrac{13}{16}\text{ in.}\right) \\ &= 1.09\text{ in.} \end{aligned}$$

(*Spec*. Eq. J3-6c)

$$\begin{aligned} r_n &= 1.2 l_c t F_u \\ &= 1.2(1.09\text{ in.})(\tfrac{1}{2}\text{ in.})(58\text{ ksi}) \\ &= 37.9\text{ kips/bolt} \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(37.9\text{ kips/bolt})$ $= 28.4\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{37.9\text{ kips/bolt}}{2.00}$ $= 19.0\text{ kips/bolt}$

Therefore, the bolt shear determined previously controls for the bolt group in the plate.

Shear Strength of Plate

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the plate is determined as follows:

$$\begin{aligned} A_{gv} &= lt \\ &= (12.0\text{ in.})(\tfrac{1}{2}\text{ in.}) \\ &= 6.00\text{ in.}^2 \end{aligned}$$

(*Spec*. Eq. J4-3)

$$\begin{aligned} R_n &= 0.60 F_y A_{gv} \\ &= 0.60(36\text{ ksi})\left(6.00\text{ in.}^2\right) \\ &= 130\text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(130\text{ kips})$ $= 130\text{ kips} > 36.0\text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{130\text{ kips}}{1.50}$ $= 86.7\text{ kips} > 24.0\text{ kips}$ **o.k.**

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the plate is determined using the net area determined in accordance with AISC *Specification* Section B4.3b.

$$\begin{aligned} A_{nv} &= \left[l - n\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t \\ &= \left[12.0\text{ in.} - 4\left(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right](\tfrac{1}{2}\text{ in.}) \\ &= 4.25\text{ in.}^2 \end{aligned}$$

(*Spec*. Eq. J4-4)

$$\begin{aligned} R_n &= 0.60 F_u A_{nv} \\ &= 0.60(58\text{ ksi})\left(4.25\text{ in.}^2\right) \\ &= 148\text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(148\text{ kips})$ $= 111\text{ kips} > 36.0\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{148\text{ kips}}{2.00}$ $= 74.0\text{ kips} > 24.0\text{ kips}$ **o.k.**

Block Shear Rupture of Plate

From AISC *Specification* Section J4.3, the block shear rupture strength of the plate is determined as follows.

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad \text{(}\textit{Spec}\text{. Eq. J4-5)}$$

where

$$\begin{aligned} A_{gv} &= \left(l - l_{ev}\right)t \\ &= \left(12.0\text{ in.} - 1\tfrac{1}{2}\text{ in.}\right)\left(\tfrac{1}{2}\text{ in.}\right) \\ &= 5.25\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - (n - 0.5)\left(d_h + \tfrac{1}{16}\text{ in.}\right)t \\ &= 5.25\text{ in.}^2 - (4 - 0.5)\left(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\left(\tfrac{1}{2}\text{ in.}\right) \\ &= 3.72\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= \left[3\text{ in.} + 1\tfrac{1}{4}\text{ in.} - 1.5\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t \\ &= \left[3\text{ in.} + 1\tfrac{1}{4}\text{ in.} - 1.5\left(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right]\left(\tfrac{1}{2}\text{ in.}\right) \\ &= 1.47\text{ in.}^2 \end{aligned}$$

$$U_{bs} = 0.5$$

and

$$\begin{aligned} R_n &= 0.60(58\text{ ksi})\left(3.72\text{ in.}^2\right) + 0.5(58\text{ ksi})\left(1.47\text{ in.}^2\right) < 0.60(36\text{ ksi})\left(5.25\text{ in.}^2\right) + 0.5(58\text{ ksi})\left(1.47\text{ in.}^2\right) \\ &= 172\text{ kips} > 156\text{ kips} \end{aligned}$$

Therefore:

$$R_n = 156\text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(156\text{ kips})$ $= 117\text{ kips} > 36.0\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{156\text{ kips}}{2.00}$ $= 78.0\text{ kips} > 24.0\text{ kips}$ **o.k.**

Interaction of Shear Yielding and Flexural Yielding of Plate

From AISC *Manual* Part 10, the plate is checked for the interaction of shear yielding and yielding due to flexure as follows:

LRFD	ASD
$\left(\frac{V_r}{V_c}\right)^2+\left(\frac{M_r}{M_c}\right)^2\leq 1.0$ (*Manual* Eq. 10-5)	$\left(\frac{V_r}{V_c}\right)^2+\left(\frac{M_r}{M_c}\right)^2\leq 1.0$ (*Manual* Eq. 10-5)
From the preceding calculations:	From the preceding calculations:
$V_r = V_u$ $= 36.0\text{ kips}$	$V_r = V_a$ $= 24.0\text{ kips}$
$V_c = \phi_v V_n$ $= 130\text{ kips}$	$V_c = \frac{V_n}{\Omega_v}$ $= 86.7\text{ kips}$
From AISC *Manual* Part 10:	From AISC *Manual* Part 10:
$M_c = \phi_b M_n$ $= \phi_b F_y Z_{pl}$ $= 0.90(36\text{ ksi})\left[\frac{(\text{½ in.})(12\text{ in.})^2}{4}\right]$ $= 583\text{ kip-in.}$	$M_c = \frac{M_n}{\Omega_b}$ $= \frac{F_y Z_{pl}}{\Omega_b}$ $= \left(\frac{36\text{ ksi}}{1.67}\right)\left[\frac{(\text{½ in.})(12\text{ in.})^2}{4}\right]$ $= 388\text{ kip-in.}$
$M_r = M_u$ $= V_u a$ $= (36.0\text{ kips})(9\text{ in.})$ $= 324\text{ kip-in.}$	$M_r = M_a$ $= V_a a$ $= (24.0\text{ kips})(9\text{ in.})$ $= 216\text{ kip-in.}$
$\left(\frac{36.0\text{ kips}}{130\text{ kips}}\right)^2+\left(\frac{324\text{ kip-in.}}{583\text{ kip-in.}}\right)^2 = 0.386 < 1.0$ **o.k.**	$\left(\frac{24.0\text{ kips}}{86.7\text{ kips}}\right)^2+\left(\frac{216\text{ kip-in.}}{388\text{ kip-in.}}\right)^2 = 0.387 < 1.0$ **o.k.**

Lateral-Torsional Buckling of Plate

The plate is checked for the limit state of buckling using the double-coped beam procedure as given in AISC *Manual* Part 9, where the unbraced length for lateral-torsional buckling, L_b, is taken as the distance from the first column of bolts to the supporting column web and the top cope dimension, d_{ct}, is conservatively taken as the distance from the top of the beam to the first row of bolts.

$$C_b = \left[3+\ln\left(\frac{L_b}{d}\right)\right]\left(1-\frac{d_{ct}}{d}\right)\geq 1.84 \qquad \text{(\textit{Manual} Eq. 9-15)}$$

$$= \left[3+\ln\left(\frac{9\text{ in.}}{12\text{ in.}}\right)\right]\left(1-\frac{3\text{ in.}}{12\text{ in.}}\right)\geq 1.84$$

$$= 2.03 > 1.84$$

Therefore:

$$C_b = 2.03$$

From AISC *Specification* Section F11, the flexural strength of the plate for the limit state of lateral-torsional buckling is determined as follows:

$$\begin{aligned}\frac{L_b d}{t^2} &= \frac{(9\text{ in.})(12\text{ in.})}{(\text{½ in.})^2}\\ &= 432\end{aligned}$$

$$\begin{aligned}\frac{0.08E}{F_y} &= \frac{0.08(29{,}000\text{ ksi})}{36\text{ ksi}}\\ &= 64.4\end{aligned}$$

$$\begin{aligned}\frac{1.9E}{F_y} &= \frac{1.9(29{,}000\text{ ksi})}{36\text{ ksi}}\\ &= 1{,}530\end{aligned}$$

Because $\dfrac{0.08E}{F_y} < \dfrac{L_b d}{t^2} \le \dfrac{1.9E}{F_y}$, use AISC *Specification* Section F11.2(b):

$$\begin{aligned}M_p &= F_y Z_x\\ &= (36\text{ ksi})\left[\frac{(\text{½ in.})(12\text{ in.})^2}{4}\right]\\ &= 648\text{ kip-in.}\end{aligned}$$

$$\begin{aligned}M_y &= F_y S_x\\ &= (36\text{ ksi})\left[\frac{(\text{½ in.})(12\text{ in.})^2}{6}\right]\\ &= 432\text{ kip-in.}\end{aligned}$$

$$M_n = C_b\left[1.52 - 0.274\left(\frac{L_b d}{t^2}\right)\frac{F_y}{E}\right]M_y \le M_p \qquad (\textit{Spec}.\text{ Eq. F11-2})$$

$$\begin{aligned}&= (2.03)\left[1.52 - 0.274(432)\left(\frac{36\text{ ksi}}{29{,}000\text{ ksi}}\right)\right](432\text{ kip-in.}) > 648\text{ kip-in.}\\ &= 1{,}200\text{ kip-in.} > 648\text{ kip-in.}\end{aligned}$$

Therefore:

$$M_n = 648\text{ kip-in.}$$

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(648 \text{ kip-in.})$ $= 583 \text{ kip-in.} > 324 \text{ kip-in.}$ **o.k.**	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{648 \text{ kip-in}}{1.67}$ $= 388 \text{ kip-in.} > 216 \text{ kip-in.}$ **o.k.**

Flexural Rupture of Plate

The net plastic section modulus of the plate, Z_{net}, is determined from AISC *Manual* Table 15-3:

$$Z_{net} = 12.8 \text{ in.}^3$$

From AISC *Manual* Equation 9-4:

$$\begin{aligned} M_n &= F_u Z_{net} \\ &= (58 \text{ ksi})(12.8 \text{ in.}^3) \\ &= 742 \text{ kip-in.} \end{aligned}$$

(*Manual* Eq. 9-4)

LRFD	ASD
$\phi_b = 0.75$ $\phi_b M_n = 0.75(742 \text{ kip-in.})$ $= 557 \text{ kip-in.} > 324 \text{ kip-in.}$ **o.k.**	$\Omega_b = 2.00$ $\dfrac{M_n}{\Omega} = \dfrac{742 \text{ kip-in.}}{2.00}$ $= 371 \text{ kip-in.} > 216 \text{ kip-in.}$ **o.k.**

Weld Between Plate and Column Web (AISC Manual *Part 10)*

From AISC *Manual* Part 10, a weld size of (⅝)t_p is used to develop the strength of the shear plate.

$$\begin{aligned} w &= \tfrac{5}{8} t_p \\ &= \tfrac{5}{8}(\tfrac{1}{2} \text{ in.}) \\ &= \tfrac{5}{16} \text{ in.} \end{aligned}$$

Use a two-sided 5⁄16-in. fillet weld.

Strength of Column Web at Weld

The minimum column web thickness to match the shear rupture strength of the weld is determined as follows:

$$\begin{aligned} t_{min} &= \frac{3.09D}{F_u} \\ &= \frac{3.09(5 \text{ sixteenths})}{65 \text{ ksi}} \\ &= 0.238 \text{ in.} < 0.440 \text{ in.} \quad \textbf{o.k.} \end{aligned}$$

(*Manual* Eq. 9-2)

Conclusion

The connection is found to be adequate as given for the applied loads.

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EXAMPLE II.A-19B EXTENDED SINGLE-PLATE CONNECTION SUBJECT TO AXIAL AND SHEAR LOADING

Given:

Verify the available strength of an extended single-plate connection for an ASTM A992 W18×60 beam to the web of an ASTM A992 W14×90 column, as shown in Figure II.A-19B-1, to support the following beam end reactions:

LRFD	ASD
Shear, V_u = 75 kips Axial tension, N_u = 60 kips	Shear, V_a = 50 kips Axial tension, N_a = 40 kips

Use 70-ksi electrodes and ASTM A572 Grade 50 plate.

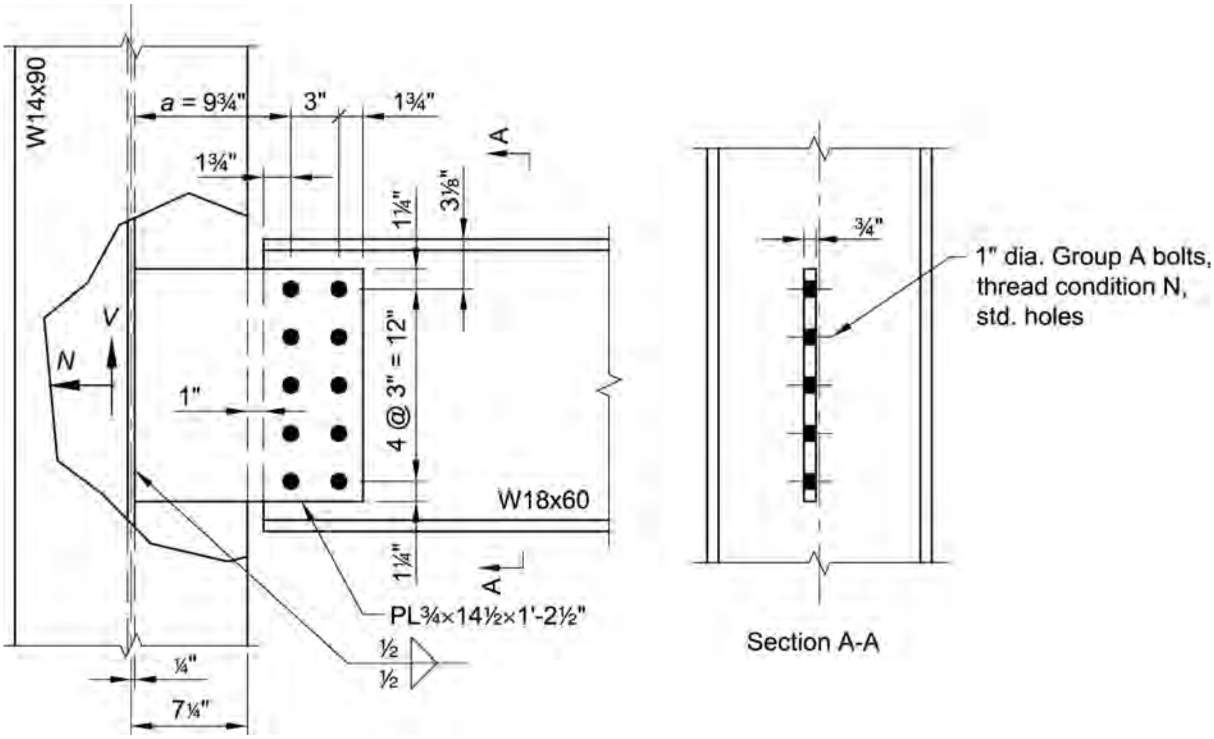

Fig. II.A-19B-1. Connection geometry for Example II.A-19B.

Solution:

From AISC *Manual* Table 2-4 and Table 2-5, the material properties are as follows:

Beam, column
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Plate
ASTM A572 Grade 50
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W18×60
$A_g = 17.6 \text{ in.}^2$
$d = 18.2 \text{ in.}$
$t_w = 0.415 \text{ in.}$
$b_f = 7.56 \text{ in.}$
$t_f = 0.695 \text{ in.}$

Column
W14×90
$d = 14.0 \text{ in.}$
$t_w = 0.440 \text{ in.}$
$k_{des} = 1.31 \text{ in.}$

From AISC *Specification* Table J3.3, for 1-in.-diameter bolts with standard holes:

$d_h = 1\frac{1}{8} \text{ in.}$

Per AISC *Specification* Section J3.2, standard holes are required for both the plate and beam web because the beam axial force acts longitudinally to the direction of a slotted hole and bolts are designed for bearing.

The resultant load is determined as follows:

LRFD	ASD
$R_u = \sqrt{{V_u}^2 + {N_u}^2}$ $= \sqrt{(75 \text{ kips})^2 + (60 \text{ kips})^2}$ $= 96.0 \text{ kips}$	$R_a = \sqrt{{V_a}^2 + {N_a}^2}$ $= \sqrt{(50 \text{ kips})^2 + (40 \text{ kips})^2}$ $= 64.0 \text{ kips}$

The resultant load angle is determined as follows:

LRFD	ASD
$\theta = \tan^{-1}\left(\dfrac{60 \text{ kips}}{75 \text{ kips}}\right)$ $= 38.7°$	$\theta = \tan^{-1}\left(\dfrac{40 \text{ kips}}{50 \text{ kips}}\right)$ $= 38.7°$

Strength of Bolted Connection—Beam Web

The strength of the bolt group is determined by interpolating AISC *Manual* Table 7-7 for $\text{Angle} = 30°$ and $n = 5$. Note that 30° is used conservatively in order to employ AISC *Manual* Table 7-7. A direct analysis can be performed to obtain an accurate value using the instantaneous center of rotation method.

$$\begin{aligned} e_x &= a + 0.5s \\ &= 9\tfrac{3}{4} \text{ in.} + 0.5(3 \text{ in.}) \\ &= 11.3 \text{ in.} \end{aligned}$$

$C = 3.53$ by interpolation

From AISC *Manual* Table 7-1, the available shear strength per bolt for 1-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) is:

LRFD	ASD
$\phi r_n = 31.8\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 21.2\text{ kips/bolt}$

The available bearing strength of the beam web is determined from AISC *Specification* Equation J3-6b. This equation is applicable in lieu of Equation J3-6a, because plowing of the bolts in the beam web is desirable to provide some flexibility in the connection:

$$\begin{aligned} r_n &= 3.0dt_wF_u \\ &= 3.0(1\text{ in.})(0.415\text{ in.})(65\text{ ksi}) \\ &= 80.9\text{ kips/bolt} \end{aligned} \qquad (\textit{Spec}.\text{ Eq. J3-6b})$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(80.9\text{ kips/bolt})$ $= 60.7\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{80.9\text{ kips/bolt}}{2.00}$ $= 40.5\text{ kips/bolt}$

The available tearout strength of the beam web is determined from *Specification* Equation J3-6d. Similar to the bearing strength determination, this equation is used to allow plowing of the bolts in the beam web, and thus provide some flexibility in the connection.

Because the direction of load on the bolt is unknown, the minimum bolt edge distance is used to determine a worst case available tearout strength (including a ¼-in. tolerance to account for possible beam underrun). If a computer program is available, the true l_e can be calculated based on the instantaneous center of rotation.

$$\begin{aligned} l_c &= l_{eh} - 0.5d_h \\ &= (1\tfrac{3}{4}\text{ in.} - \tfrac{1}{4}\text{ in.}) - 0.5(1\tfrac{1}{8}\text{ in.}) \\ &= 0.938\text{ in.} \end{aligned}$$

$$\begin{aligned} r_n &= 1.5l_ct_wF_u \\ &= 1.5(0.938\text{ in.})(0.415\text{ in.})(65\text{ ksi}) \\ &= 38.0\text{ kips/bolt} \end{aligned} \qquad (\textit{Spec}.\text{ Eq. J3-6d})$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(38.0\text{ kips/bolt})$ $= 28.5\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{38.0\text{ kips/bolt}}{2.00}$ $= 19.0\text{ kips/bolt}$

The tearout strength controls for bolts in the beam web.

The available strength of the bolted connection is determined using the minimum available strength calculated for bolt shear, bearing on the beam web and tearout on the beam web. From AISC *Manual* Equation 7-16, the bolt group eccentricity is accounted for by multiplying the minimum available bolt strength by the bolt coefficient C.

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LRFD	ASD
$\phi R_n = C\phi r_n$ $= 3.53(28.5 \text{ kips/bolt})$ $= 101 \text{ kips} > 96.0 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = C\dfrac{r_n}{\Omega}$ $= 3.53(19.0 \text{ kips/bolt})$ $= 67.1 \text{ kips} > 64.0 \text{ kips}$ **o.k.**

Strength of Bolted Connection—Plate

Note that bolt bearing on the beam web controls over bearing on the plate because the beam web is thinner than the plate; therefore, this limit state will not control.

As was discussed for the beam web, the available tearout strength of the plate is determined from *Specification* Equation J3-6d. The bolt edge distance in the vertical direction controls for this design.

$$\begin{aligned} l_c &= l_{ev} - 0.5d_h \\ &= 1\tfrac{1}{4} \text{ in.} - 0.5(1\tfrac{1}{8} \text{ in.}) \\ &= 0.688 \text{ in.} \end{aligned}$$

$$\begin{aligned} r_n &= 1.5 l_c t F_u \\ &= 1.5(0.688 \text{ in.})(\tfrac{3}{4} \text{ in.})(65 \text{ ksi}) \\ &= 50.3 \text{ kips/bolt} \end{aligned}$$ (*Spec*. Eq. J3-6d)

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(50.3 \text{ kips/bolt})$ $= 37.7 \text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{50.3 \text{ kips/bolt}}{2.00}$ $= 25.2 \text{ kips/bolt}$

Therefore, the available strength of the bolted connection at the beam web, as determined previously, controls.

Shear Yielding Strength of Beam

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the beam is determined as follows:

$$\begin{aligned} A_{gv} &= dt_w \\ &= (18.2 \text{ in.})(0.415 \text{ in.}) \\ &= 7.55 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60 F_y A_{gv} \\ &= 0.60(50 \text{ ksi})(7.55 \text{ in.}^2) \\ &= 227 \text{ kips} \end{aligned}$$ (*Spec*. Eq. J4-3)

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(227 \text{ kips})$ $= 227 \text{ kips} > 75 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{227 \text{ kips}}{1.50}$ $= 151 \text{ kips} > 50 \text{ kips}$ **o.k.**

Tensile Yielding Strength of Beam

From AISC *Specification* Section J4.1(a), the available tensile yielding strength of the beam web is determined as follows:

$$\begin{aligned} R_n &= F_y A_g \\ &= (50\text{ ksi})(17.6\text{ in.}^2) \\ &= 880\text{ kips} \end{aligned} \qquad (\textit{Spec}.\text{ Eq. J4-1})$$

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(880\text{ kips})$ $= 792\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{880\text{ kips}}{1.67}$ $= 527\text{ kips} > 40\text{ kips}$ **o.k.**

Tensile Rupture Strength of Beam

From AISC *Specification* Section J4.1, determine the available tensile rupture strength of the beam. The effective net area is $A_e = A_n U$, where U is determined from AISC *Specification* Table D3.1, Case 2.

$$\begin{aligned} \bar{x} &= \frac{2b_f{}^2 t_f + t_w{}^2(d - 2t_f)}{8b_f t_f + 4t_w(d - 2t_f)} \\ &= \frac{2(7.56\text{ in.})^2(0.695\text{ in.}) + (0.415\text{ in.})^2\left[18.2\text{ in.} - 2(0.695\text{ in.})\right]}{8(7.56\text{ in.})(0.695\text{ in.}) + 4(0.415\text{ in.})\left[18.2\text{ in.} - 2(0.695\text{ in.})\right]} \\ &= 1.18\text{ in.} \end{aligned}$$

$$\begin{aligned} U &= 1 - \frac{\bar{x}}{l} \\ &= 1 - \frac{1.18\text{ in.}}{3.00\text{ in.}} \\ &= 0.607 \end{aligned}$$

$$\begin{aligned} A_n &= A_g - n(d_h + \tfrac{1}{16}\text{ in.})t_w \\ &= 17.6\text{ in.}^2 - 5(1\tfrac{1}{8}\text{ in.} + \tfrac{1}{16}\text{ in.})(0.415\text{ in.}) \\ &= 15.1\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= F_u A_e \\ &= F_u A_n U \\ &= (65\text{ ksi})(15.1\text{ in.}^2)(0.607) \\ &= 596\text{ kips} \end{aligned} \qquad (\textit{Spec}.\text{ Eq. J4-2})$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(596\text{ kips})$ $= 447\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{596\text{ kips}}{2.00}$ $= 298\text{ kips} > 40\text{ kips}$ **o.k.**

Block Shear Rupture of Beam Web

Block shear rupture is only applicable in the direction of the axial load because the beam is uncoped and the limit state is not applicable for an uncoped beam subject to vertical shear. Assuming a U-shaped tearout relative to the axial load, and assuming a horizontal edge distance of l_{eh} = 1¾ in. − ¼ in. = 1½ in. to account for a possible beam underrun of ¼ in., the block shear rupture strength is:

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad (Spec.\ \text{Eq. J4-5})$$

where

$$\begin{aligned} A_{gv} &= (2\text{ shear planes})(s + l_{eh})t_w \\ &= (2\text{ shear planes})(3\text{ in.} + 1\tfrac{1}{2}\text{ in.})(0.415\text{ in.}) \\ &= 3.74\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - (2\text{ shear planes})(1.5)(d_h + \tfrac{1}{16}\text{ in.})t_w \\ &= 3.74\text{ in.}^2 - (2\text{ shear planes})(1.5)(1\tfrac{1}{8}\text{ in.} + \tfrac{1}{16}\text{ in.})(0.415\text{ in.}) \\ &= 2.26\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= \left[12.0\text{ in.} - (n-1)(d_h + \tfrac{1}{16}\text{ in.})\right]t_w \\ &= \left[12.0\text{ in.} - (5-1)(1\tfrac{1}{8}\text{ in.} + \tfrac{1}{16}\text{ in.})\right](0.415\text{ in.}) \\ &= 3.01\text{ in.}^2 \end{aligned}$$

$$U_{bs} = 1.0$$

and

$$\begin{aligned} R_n &= 0.60(65\text{ ksi})(2.26\text{ in.}^2) + 1.0(65\text{ ksi})(3.01\text{ in.}^2) \le 0.60(50\text{ ksi})(3.74\text{ in.}^2) + 1.0(65\text{ ksi})(3.01\text{ in.}^2) \\ &= 284\text{ kips} < 308\text{ kips} \end{aligned}$$

Therefore:

$$R_n = 284\text{ kips}$$

From AISC *Specification* Section J4.3, the available strength for the limit state of block shear rupture of the beam web is:

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(284\text{ kips})$ $= 213\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{284\text{ kips}}{2.00}$ $= 142\text{ kips} > 40\text{ kips}$ **o.k.**

Maximum Plate Thickness

Determine the maximum plate thickness, t_{max}, that will result in the plate yielding before the bolts shear. From AISC *Specification* Table J3.2:

$$F_{nv} = 54 \text{ ksi}$$

From AISC *Manual* Table 7-7 for two column of bolts, Angle = 0°, s = 3 in., and n = 5:

$$C' = 38.7 \text{ in.}$$

$$\begin{aligned} M_{max} &= \frac{F_{nv}}{0.90}\left(A_b C'\right) && \text{(\textit{Manual} Eq. 10-4)} \\ &= \frac{54 \text{ ksi}}{0.90}\left(0.785 \text{ in.}^2\right)\left(38.7 \text{ in.}\right) \\ &= 1{,}820 \text{ kip-in.} \end{aligned}$$

$$\begin{aligned} t_{max} &= \frac{6M_{max}}{F_y l^2} && \text{(\textit{Manual} Eq. 10-3)} \\ &= \frac{6\left(1{,}820 \text{ kip-in.}\right)}{\left(50 \text{ ksi}\right)\left(14\tfrac{1}{2} \text{ in.}\right)^2} \\ &= 1.04 \text{ in.} > \tfrac{3}{4} \text{ in.} \quad \textbf{o.k.} \end{aligned}$$

Flexure Strength of Plate

The required flexural strength of the plate is determined as follows:

LRFD	ASD
$M_u = V_u a$ $= \left(75 \text{ kips}\right)\left(9\tfrac{3}{4} \text{ in.}\right)$ $= 731 \text{ kip-in.}$	$M_a = V_a a$ $= \left(50 \text{ kips}\right)\left(9\tfrac{3}{4} \text{ in.}\right)$ $= 488 \text{ kip-in.}$

The plate is checked for the limit state of buckling using the double-coped beam procedure as given in AISC *Manual* Part 9, where the unbraced length for lateral-torsional buckling, L_b, is taken as the distance from the first column of bolts to the supporting column web and the top cope dimension, d_{ct}, is conservatively taken as the distance from the top of the beam to the first row of bolts.

$$\begin{aligned} C_b &= \left[3 + \ln\left(\frac{L_b}{d}\right)\right]\left(1 - \frac{d_{ct}}{d}\right) \geq 1.84 \\ &= \left[3 + \ln\left(\frac{9\tfrac{3}{4} \text{ in.}}{14\tfrac{1}{2} \text{ in.}}\right)\right]\left(1 - \frac{3\tfrac{1}{8} \text{ in.}}{14\tfrac{1}{2} \text{ in.}}\right) \geq 1.84 \\ &= 2.04 > 1.84 \end{aligned}$$

Therefore:

$$C_b = 2.04$$

The available flexural strength of the plate is determined using AISC *Specification* Section F11 as follows:

For yielding of the plate:

$$M_n = M_p = F_y Z \leq 1.6 F_y S_x \qquad \text{(\textit{Spec}. Eq. F11-1)}$$

$$= (50\text{ ksi})\left[\frac{(\tfrac{3}{4}\text{ in.})(14\tfrac{1}{2}\text{ in.})^2}{4}\right] \leq 1.6(50\text{ ksi})\left[\frac{(\tfrac{3}{4}\text{ in.})(14\tfrac{1}{2}\text{ in.})^2}{6}\right]$$

$$= 1{,}970\text{ kip-in.} < 2{,}100\text{ kip-in.}$$

$$= 1{,}970\text{ kip-in.}$$

For lateral-torsional buckling of the plate:

$$\frac{L_b d}{t^2} = \frac{(9\tfrac{3}{4}\text{ in.})(14\tfrac{1}{2}\text{ in.})}{(\tfrac{3}{4}\text{ in.})^2}$$

$$= 251$$

$$\frac{0.08E}{F_y} = \frac{0.08(29{,}000\text{ ksi})}{50\text{ ksi}}$$

$$= 46.4$$

$$\frac{1.9E}{F_y} = \frac{1.9(29{,}000\text{ ksi})}{50\text{ ksi}}$$

$$= 1{,}100$$

Because $\frac{0.08E}{F_y} < \frac{L_b d}{t^2} \leq \frac{1.9E}{F_y}$, use AISC *Specification* Section F11.2(b):

$$M_y = F_y S_x$$

$$= (50\text{ ksi})\left[\frac{(\tfrac{3}{4}\text{ in.})(14\tfrac{1}{2}\text{ in.})^2}{6}\right]$$

$$= 1{,}310\text{ kip-in.}$$

$$M_n = C_b\left[1.52 - 0.274\left(\frac{L_b d}{t^2}\right)\frac{F_y}{E}\right]M_y \leq M_p \qquad \text{(\textit{Spec}. Eq. F11-2)}$$

$$= 2.04\left[1.52 - 0.274(251)\left(\frac{50\text{ ksi}}{29{,}000\text{ ksi}}\right)\right](1{,}310\text{ kip-in.}) \leq 1{,}970\text{ kip-in.}$$

$$= 3{,}750\text{ kip-in.} > 1{,}970\text{ kip-in.}$$

Therefore:

$$M_n = 1{,}970\text{ kip-in.}$$

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LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(1{,}970 \text{ kip-in.})$ $= 1{,}770 \text{ kip-in.} > 731 \text{ kip-in.}$ **o.k.**	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{1{,}970 \text{ kip-in.}}{1.67}$ $= 1{,}180 \text{ kip-in.} > 488 \text{ kip-in.}$ **o.k.**

Shear Yielding Strength of Plate

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the plate is determined as follows:

$$\begin{aligned} A_{gv} &= lt \\ &= (14\tfrac{1}{2} \text{ in.})(\tfrac{3}{4} \text{ in.}) \\ &= 10.9 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_{nv} &= 0.60 F_y A_{gv} \\ &= 0.60(50 \text{ ksi})(10.9 \text{ in.}^2) \\ &= 327 \text{ kips} \end{aligned}$$ (*Spec*. Eq. J4-3)

LRFD	ASD
$\phi = 1.00$ $\phi R_{nv} = 1.00(327 \text{ kips})$ $= 327 \text{ kips} > 75 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_{nv}}{\Omega} = \dfrac{327 \text{ kips}}{1.50}$ $= 218 \text{ kips} > 50 \text{ kips}$ **o.k.**

Tension Yielding Strength of Plate

From AISC *Specification* Section J4.1(a), the available tensile yielding strength of the plate is determined as follows:

$$\begin{aligned} A_g &= lt \\ &= (14\tfrac{1}{2} \text{ in.})(\tfrac{3}{4} \text{ in.}) \\ &= 10.9 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_{np} &= F_y A_g \\ &= (50 \text{ ksi})(10.9 \text{ in.}) \\ &= 545 \text{ kips} \end{aligned}$$ (from *Spec*. Eq. J4-1)

LRFD	ASD
$\phi = 0.90$ $\phi R_{np} = 0.90(545 \text{ kips})$ $= 491 \text{ kips} > 60 \text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_{np}}{\Omega} = \dfrac{545 \text{ kips}}{1.67}$ $= 326 \text{ kips} > 40 \text{ kips}$ **o.k.**

Interaction of Axial, Flexure and Shear Yielding in Plate

AISC *Specification* Chapter H does not address combined flexure and shear. The method employed here is derived from Chapter H in conjunction with AISC *Manual* Equation 10-5, as follows:

LRFD	ASD
$\dfrac{N_u}{\phi R_{np}} = \dfrac{60\text{ kips}}{491\text{ kips}}$ $= 0.122$ Because $\dfrac{N_u}{\phi R_{np}} < 0.2$: $\left(\dfrac{N_u}{2\phi R_{np}} + \dfrac{V_u a}{\phi M_n}\right)^2 + \left(\dfrac{V_u}{\phi R_{nv}}\right)^2 \le 1$ $= \left[\dfrac{60\text{ kips}}{2(491\text{ kips})} + \dfrac{(75\text{ kips})(9\tfrac{3}{4}\text{ in.})}{1{,}770\text{ kip-in.}}\right]^2$ $+ \left(\dfrac{75\text{ kips}}{327\text{ kips}}\right)^2 \le 1$ $= 0.278 < 1$ **o.k.**	$\dfrac{\Omega N_a}{R_{np}} = \dfrac{40\text{ kips}}{326\text{ kips}}$ $= 0.123$ Because $\dfrac{\Omega N_a}{R_{np}} < 0.2$: $\left(\dfrac{\Omega N_a}{2R_{np}} + \dfrac{\Omega V_a a}{M_n}\right)^2 + \left(\dfrac{\Omega V_a}{R_{nv}}\right)^2 \le 1$ $= \left[\dfrac{40\text{ kips}}{2(326\text{ kips})} + \dfrac{(50\text{ kips})(9\tfrac{3}{4}\text{ in.})}{1{,}180\text{ kip-in.}}\right]^2$ $+ \left(\dfrac{50\text{ kips}}{218\text{ kips}}\right)^2 \le 1$ $= 0.278 < 1$ **o.k.**

Tensile Rupture Strength of Plate

From AISC *Specification* Section J4.1(b), the available tensile rupture strength of the plate is determined as follows:

$$\begin{aligned} A_n &= \left[l - n\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t \\ &= \left[14\tfrac{1}{2}\text{ in.} - (5\text{ bolts})\left(1\tfrac{1}{8}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right]\left(\tfrac{3}{4}\text{ in.}\right) \\ &= 6.42\text{ in.}^2 \end{aligned}$$

AISC *Specification* Table D3.1, Case 1, applies in this case because the tension load is transmitted directly to the cross-sectional element by fasteners; therefore, $U = 1.0$.

$$\begin{aligned} A_e &= A_n U \qquad \text{(}\textit{Spec}\text{. Eq. D3-1)} \\ &= \left(6.42\text{ in.}^2\right)(1.0) \\ &= 6.42\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_{np} &= F_u A_e \qquad \text{(}\textit{Spec}\text{. Eq. J4-2)} \\ &= (65\text{ ksi})\left(6.42\text{ in.}^2\right) \\ &= 417\text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_{np} = 0.75(417\text{ kips})$ $= 313\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_{np}}{\Omega} = \dfrac{417\text{ kips}}{2.00}$ $= 209\text{ kips} > 40\text{ kips}$ **o.k.**

Flexural Rupture of the Plate

The available flexural rupture strength of the plate is determined as follows:

$$
\begin{aligned}
Z_{net} &= \frac{tl^2}{4} - \frac{t}{4}\left[\left(d_h + \tfrac{1}{16}\text{ in.}\right)(s)\left(n^2 - 1\right) + \left(d_h + \tfrac{1}{16}\text{ in.}\right)^2\right] \\
&= \frac{\left(\tfrac{3}{4}\text{ in.}\right)\left(14\tfrac{1}{2}\text{ in.}\right)^2}{4} - \left(\frac{\tfrac{3}{4}\text{ in.}}{4}\right)\left\{\left(1\tfrac{1}{8}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)(3\text{ in.})\left[(5)^2 - 1\right] + \left(1\tfrac{1}{8}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)^2\right\} \\
&= 23.1\text{ in.}^3
\end{aligned}
$$

$$
\begin{aligned}
M_n &= F_u Z_{net} \\
&= (65\text{ ksi})\left(23.1\text{ in.}^3\right) \\
&= 1{,}500\text{ kip-in.}
\end{aligned}
\qquad (\textit{Manual}\text{ Eq. 9-4})
$$

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi M_n = 0.75(1{,}500\text{ kip-in.})$ $= 1{,}130\text{ kip-in.} > 731\text{ kip-in.}$ **o.k.**	$\dfrac{M_n}{\Omega} = \dfrac{1{,}500\text{ kip-in.}}{2.00}$ $= 750\text{ kip-in.} > 488\text{ kip-in.}$ **o.k.**

Shear Rupture Strength of Plate

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the plate is determined as follows:

$$
\begin{aligned}
A_{nv} &= \left[l - n\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t_p \\
&= \left[14\tfrac{1}{2}\text{ in.} - 5\left(1\tfrac{1}{8}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right]\left(\tfrac{3}{4}\text{ in.}\right) \\
&= 6.42\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
R_{nv} &= 0.60 F_u A_{nv} \\
&= 0.60(65\text{ ksi})\left(6.42\text{ in.}^2\right) \\
&= 250\text{ kips}
\end{aligned}
\qquad (\textit{Spec}.\text{ Eq. J4-4})
$$

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_{nv} = 0.75(250\text{ kips})$ $= 188\text{ kips} > 75\text{ kips}$ **o.k.**	$\dfrac{R_{nv}}{\Omega} = \dfrac{250\text{ kips}}{2.00}$ $= 125\text{ kips} > 50\text{ kips}$ **o.k.**

Interaction of Axial, Flexure and Shear Rupture in Plate

AISC *Specification* Chapter H does not address combined flexure and shear. The method employed here is derived from Chapter H in conjunction with AISC *Manual* Equation 10-5, as follows:

LRFD	ASD
$\dfrac{N_u}{\phi R_{np}} = \dfrac{60\text{ kips}}{313\text{ kips}}$ $= 0.192$	$\dfrac{\Omega N_a}{R_{np}} = \dfrac{40\text{ kips}}{209\text{ kips}}$ $= 0.191$

LRFD	ASD
Because $\dfrac{N_u}{\phi R_{np}} < 0.2$: $\left(\dfrac{N_u}{2\phi R_{np}} + \dfrac{V_u a}{\phi M_n}\right)^2 + \left(\dfrac{V_u}{\phi R_{nv}}\right)^2 \leq 1$ $\left[\dfrac{60\text{ kips}}{2(313\text{ kips})} + \dfrac{(75\text{ kips})(9\tfrac{3}{4}\text{ in.})}{1{,}130\text{ kip-in.}}\right]^2 + \left(\dfrac{75\text{ kips}}{188\text{ kips}}\right)^2 \leq 1$ $0.711 < 1$ **o.k.**	Because $\dfrac{\Omega N_a}{R_{np}} < 0.2$: $\left(\dfrac{\Omega N_a}{2R_{np}} + \dfrac{\Omega V_a a}{M_n}\right)^2 + \dfrac{\Omega V_a}{R_{nv}} \leq 1$ $\left[\dfrac{40\text{ kips}}{2(209\text{ kips})} + \dfrac{(50\text{ kips})(9\tfrac{3}{4}\text{ in.})}{750\text{ kip-in.}}\right]^2 + \left(\dfrac{50\text{ kips}}{125\text{ kips}}\right)^2 \leq 1$ $0.716 < 1$ **o.k.**

Block Shear Rupture Strength of Plate—Beam Shear Direction

The nominal strength for the limit state of block shear rupture of the plate, assuming an L-shaped tearout due to the shear load only as shown in Figure II.A-19B-2(a), is determined as follows:

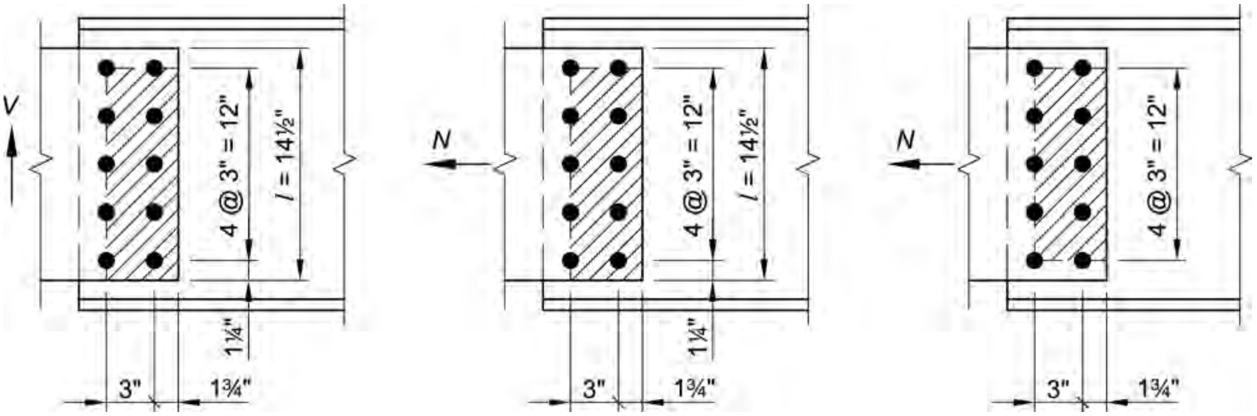
$$R_{bsv} = 0.60F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.60F_y A_{gv} + U_{bs} F_u A_{nt} \qquad (\textit{Spec.}\text{ Eq. J4-5})$$

where

$$\begin{aligned} A_{gv} &= (l - l_{ev})t \\ &= (14\tfrac{1}{2}\text{ in.} - 1\tfrac{1}{4}\text{ in.})(\tfrac{3}{4}\text{ in.}) \\ &= 9.94\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - (n_v - 0.5)(d_h + \tfrac{1}{16}\text{ in.})t \\ &= 9.94\text{ in.}^2 - (5 - 0.5)(1\tfrac{1}{8}\text{ in.} + \tfrac{1}{16}\text{ in.})(\tfrac{3}{4}\text{ in.}) \\ &= 5.93\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= \left[l_{eh} + (n_h - 1)s - (n_h - 0.5)(d_h + \tfrac{1}{16}\text{ in.})\right]t \\ &= \left[1\tfrac{3}{4}\text{ in.} + (2-1)(3\text{ in.}) - (2-0.5)(1\tfrac{1}{8}\text{ in.} + \tfrac{1}{16}\text{ in.})\right](\tfrac{3}{4}\text{ in.}) \\ &= 2.23\text{ in.}^2 \end{aligned}$$



(a) Beam shear direction

(b) Beam axial direction—L-shaped

(c) Beam axial direction—U-shaped

Fig. II.A-19B-2. Block shear rupture of plate.

Because stress is not uniform along the net tensile area, $U_{bs} = 0.5$.

$$\begin{aligned} R_{bsv} &= 0.60(65\text{ ksi})\left(5.93\text{ in.}^2\right)+0.5(65\text{ ksi})\left(2.23\text{ in.}^2\right) \le 0.60(50\text{ ksi})\left(9.94\text{ in.}^2\right)+0.5(65\text{ ksi})\left(2.23\text{ in.}^2\right) \\ &= 304\text{ kips} < 371\text{ kips} \end{aligned}$$

Therefore:

$$R_{bsv} = 304\text{ kips}$$

From AISC *Specification* Section J4.3, the available strength for the limit state of block shear rupture on the plate is:

LRFD	ASD
$\phi = 0.75$ $\begin{aligned} \phi R_{bsv} &= 0.75(304\text{ kips}) \\ &= 228\text{ kips} > 75\text{ kips} \quad \textbf{o.k.} \end{aligned}$	$\Omega = 2.00$ $\begin{aligned} \frac{R_{bsv}}{\Omega} &= \frac{304\text{ kips}}{2.00} \\ &= 152\text{ kips} > 50\text{ kips} \quad \textbf{o.k.} \end{aligned}$

Block Shear Rupture Strength of the Plate—Beam Axial Direction

The plate block shear rupture failure path due to axial load only could occur as an L- or U-shape. Assuming an L-shaped failure path due to axial load only, as shown in Figure II.A-19B-2(b), the available block shear rupture strength of the plate is:

$$R_{bsn} = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad (\textit{Spec.}\text{ Eq. J4-5})$$

where

$$\begin{aligned} A_{gv} &= \left[(n_h-1)s+l_{eh}\right]t \\ &= \left[(2-1)(3\text{ in.})+1\tfrac{3}{4}\text{ in.}\right](\tfrac{3}{4}\text{ in.}) \\ &= 3.56\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv}-(n_h-0.5)(d_h+\tfrac{1}{16}\text{ in.})t \\ &= 3.56\text{ in.}^2-(2-0.5)(1\tfrac{1}{8}\text{ in.}+\tfrac{1}{16}\text{ in})(\tfrac{3}{4}\text{ in.}) \\ &= 2.22\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= \left[l_{ev}+(n_v-1)s-(n_v-0.5)(d_h+\tfrac{1}{16}\text{ in.})\right]t \\ &= \left[1\tfrac{1}{4}\text{ in.}+(5-1)(3\text{ in.})-(5-0.5)(1\tfrac{1}{8}\text{ in.}+\tfrac{1}{16}\text{ in})\right](\tfrac{3}{4}\text{ in.}) \\ &= 5.93\text{ in.}^2 \end{aligned}$$

$$U_{bs} = 1.0$$

and

$$\begin{aligned} R_{bsn} &= 0.60(65\text{ ksi})\left(2.22\text{ in.}^2\right)+1.0(65\text{ ksi})\left(5.93\text{ in.}^2\right) \le 0.60(50\text{ ksi})\left(3.56\text{ in.}^2\right)+1.0(65\text{ ksi})\left(5.93\text{ in.}^2\right) \\ &= 472\text{ kips} < 492\text{ kips} \end{aligned}$$

Therefore:

$$R_{bsn} = 472 \text{ kips}$$

From AISC *Specification* Section J4.3, the available strength for the limit state of block shear rupture on the plate is:

LRFD	ASD
$\phi = 0.75$ $\phi R_{bsn} = 0.75(472 \text{ kips})$ $= 354 \text{ kips} > 60 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_{bsn}}{\Omega} = \dfrac{472 \text{ kips}}{2.00}$ $= 236 \text{ kips} > 40 \text{ kips}$ **o.k.**

Assuming a U-shaped failure path in the plate due to axial load, as shown in Figure II.A-19B-2(c), the available block shear rupture strength of the plate is:

$$R_{bsn} = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad \text{(}\textit{Spec}\text{. Eq. J4-5)}$$

where

$$\begin{aligned} A_{gv} &= (2 \text{ shear planes})\left[l_{eh} + (n_h - 1)s\right]t \\ &= (2 \text{ shear planes})\left[1\tfrac{3}{4} \text{ in.} + (2-1)(3 \text{ in.})\right](\tfrac{3}{4} \text{ in.}) \\ &= 7.13 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - (2 \text{ shear planes})(n_h - 0.5)(d_h + \tfrac{1}{16} \text{ in.})t \\ &= 7.13 \text{ in.}^2 - (2 \text{ shear planes})(2 - 0.5)(1\tfrac{1}{8} \text{ in.} + \tfrac{1}{16} \text{ in.})(\tfrac{3}{4} \text{ in.}) \\ &= 4.46 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= \left[(n_v - 1)s - (n_v - 1)(d_h + \tfrac{1}{16} \text{ in.})\right]t \\ &= \left[(5-1)(3 \text{ in.}) - (5-1)(1\tfrac{1}{8} \text{ in.} + \tfrac{1}{16} \text{ in.})\right](\tfrac{3}{4} \text{ in.}) \\ &= 5.44 \text{ in.}^2 \end{aligned}$$

$$U_{bs} = 1.0$$

and

$$\begin{aligned} R_{bsn} &= 0.60(65 \text{ ksi})(4.46 \text{ in.}^2) + 1.0(65 \text{ ksi})(5.44 \text{ in.}^2) \le 0.60(50 \text{ ksi})(7.13 \text{ in.}^2) + 1.0(65 \text{ ksi})(5.44 \text{ in.}^2) \\ &= 528 \text{ kips} < 568 \text{ kips} \end{aligned}$$

Therefore:

$$R_{bsn} = 528 \text{ kips}$$

From AISC *Specification* Section J4.3, the available strength for the limit state of block shear rupture on the plate is:

LRFD	ASD
$\phi = 0.75$ $\phi R_{bsn} = 0.75(528 \text{ kips})$ $= 396 \text{ kips} > 60 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_{bsn}}{\Omega} = \dfrac{528 \text{ kips}}{2.00}$ $= 264 \text{ kips} > 40 \text{ kips}$ **o.k.**

Block Shear Rupture Strength of Plate—Combined Shear and Axial Interaction

The same L-shaped block shear rupture failure path is loaded by forces in both the shear and axial directions. The interaction of loading in both directions is determined as follows:

LRFD	ASD
$\left(\dfrac{V_u}{\phi R_{bsv}}\right)^2+\left(\dfrac{N_u}{\phi R_{bsn}}\right)^2\le 1$ $\left(\dfrac{75\text{ kips}}{228\text{ kips}}\right)^2+\left(\dfrac{60\text{ kips}}{354\text{ kips}}\right)^2=0.137<1$ **o.k.**	$\left(\dfrac{\Omega V_a}{R_{bsv}}\right)^2+\left(\dfrac{\Omega N_a}{R_{bsn}}\right)^2\le 1$ $\left(\dfrac{50\text{ kips}}{152\text{ kips}}\right)^2+\left(\dfrac{40\text{ kips}}{236\text{ kips}}\right)^2=0.137<1$ **o.k.**

Shear Rupture Strength of Column Web at Weld

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the column web is determined as follows:

$$\begin{aligned}A_{nv}&=2lt_w\\&=2(14\tfrac{1}{2}\text{ in.})(0.440\text{ in.})\\&=12.8\text{ in.}^2\end{aligned}$$

$$\begin{aligned}R_n&=0.60F_uA_v\\&=0.60(65\text{ ksi})(12.8\text{ in.}^2)\\&=499\text{ kips}\end{aligned}$$ (*Spec*. Eq. J4-4)

LRFD	ASD
$\phi=0.75$ $\phi R_n=0.75(499\text{ kips})$ $=374\text{ kips}>75\text{ kips}$ **o.k.**	$\Omega=2.00$ $\dfrac{R_n}{\Omega}=\dfrac{499\text{ kips}}{2.00}$ $=250\text{ kips}>50\text{ kips}$ **o.k.**

Yield Line Analysis on Supporting Column Web

A yield line analysis is used to determine the strength of the column web in the direction of the axial tension load. The yield line and associated dimensions are shown in Figure II.A-19B-3 and the available strength is determined as follows:

$$\begin{aligned}T&=d-2k_{des}\\&=14.0\text{ in.}-2(1.31\text{ in.})\\&=11.4\text{ in.}\end{aligned}$$

$$\begin{aligned}a&=\frac{d}{2}-k_{des}+\frac{t_w}{2}\\&=\frac{14.0\text{ in.}}{2}-1.31\text{ in.}+\frac{0.415\text{ in.}}{2}\\&=5.90\text{ in.}\end{aligned}$$

$$\begin{aligned} b &= \frac{d}{2} - k_{des} - \frac{t_w}{2} - t_p \\ &= \frac{14.0 \text{ in.}}{2} - 1.31 \text{ in.} - \frac{0.415 \text{ in.}}{2} - \tfrac{3}{4} \text{ in.} \\ &= 4.73 \text{ in.} \end{aligned}$$

$$\begin{aligned} c &= t_p \\ &= \tfrac{3}{4} \text{ in.} \end{aligned}$$

$$R_n = \frac{{t_w}^2 F_y}{4}\left[\frac{4\sqrt{2Tab(a+b)} + l(a+b)}{ab}\right] \qquad \text{(\textit{Manual} Eq. 9-31)}$$

$$\begin{aligned} &= \frac{(0.440 \text{ in.})^2 (50 \text{ ksi})}{4}\left[\frac{4\sqrt{2(11.4 \text{ in.})(5.90 \text{ in.})(4.73 \text{ in.})(5.90 \text{ in.} + 4.73 \text{ in.})} + (14\tfrac{1}{2} \text{ in.})(5.90 \text{ in.} + 4.73 \text{ in.})}{(5.90 \text{ in.})(4.73 \text{ in.})}\right] \\ &= 41.9 \text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 1.00$	$\Omega = 1.50$
$\phi R_n = 1.00(41.9 \text{ kips})$ $= 41.9 \text{ kips} < 60 \text{ kips}$ **n.g.**	$\dfrac{R_n}{\Omega} = \dfrac{41.9 \text{ kips}}{1.50}$ $= 27.9 \text{ kips} < 40 \text{ kips}$ **n.g.**

The available column web strength is not adequate to resist the axial force in the beam. The column may be increased in size for an adequate web thickness or reinforced with stiffeners or web doubler plates. For example, a W14×120 column, with $t_w = 0.590$ in., has adequate strength to resist the given forces.

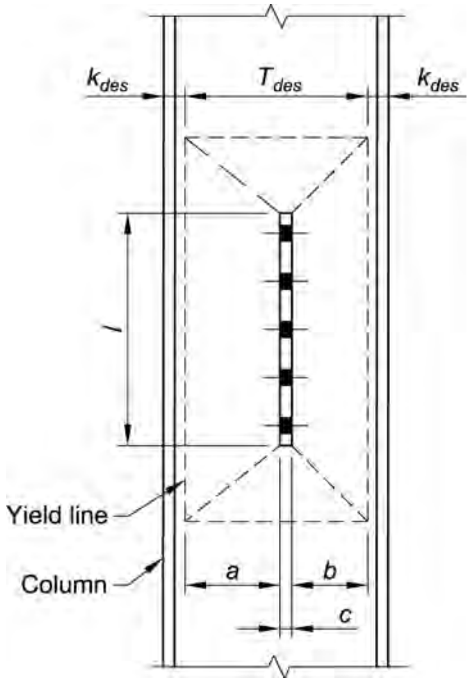


Fig II.A-19B-3. Yield line for column web.

Strength of Weld

A two-sided fillet weld with size of (⅝)t_p = 0.469 in. (use ½-in. fillet welds) is used. As discussed in AISC *Manual* Part 10, this weld size will develop the strength of the shear plate used because the moment generated by this connection is indeterminate.

The available weld strength is determined using AISC *Manual* Equation 8-2a or 8-2b, incorporating the directional strength increase from AISC *Specification* Equation J2-5, as follows:

$$\begin{aligned}\mu &= 1.0 + 0.50\sin^{1.5}\theta \\ &= 1.0 + 0.50\sin^{1.5}(38.7°) \\ &= 1.25\end{aligned}$$

LRFD	ASD
$R_n = (1.392\text{ kip/in.})Dl\mu(2\text{ sides})$ $= (1.392\text{ kip/in.})(8)(14\tfrac{1}{2}\text{ in.})(1.25)(2\text{ sides})$ $= 404\text{ kips} > 96.0\text{ kips}$ **o.k.**	$R_n = (0.928\text{ kip/in.})Dl\mu(2\text{ sides})$ $= (0.928\text{ kip/in.})(8)(14\tfrac{1}{2}\text{ in.})(1.25)(2\text{ sides})$ $= 269\text{ kips} > 64.0\text{ kips}$ **o.k.**

Conclusion

The configuration given does not work due to the inadequate column web. The column would need to be increased in size or reinforced as discussed previously.

Comments: If the applied axial load were in compression, the connection plate would need to be checked for compressive flexural buckling strength as follows. This is required in the case of the extended configuration of a single-plate connection and would not be required for the conventional configuration.

From AISC *Specification* Table C-A-7.1, Case c:

$$K = 1.2$$

$$\begin{aligned}\frac{L_c}{r} &= \frac{KL}{r} \\ &= \frac{1.2(9\tfrac{3}{4}\text{ in.})}{\tfrac{3}{4}\text{ in.}/\sqrt{12}} \\ &= 54.0\end{aligned}$$

As stated in AISC *Specification* Section J4.4, if L_c/r is greater than 25, Chapter E applies. The available critical stress of the plate, ϕF_{cr} or F_{cr}/Ω, is determined using AISC *Manual* Table 4-14 as follows:

LRFD	ASD
$\phi F_{cr} = 36.4\text{ ksi}$	$\dfrac{F_{cr}}{\Omega} = 24.2\text{ ksi}$
$\phi R_n = \phi F_{cr} l t_p$ $= (36.4\text{ ksi})(14\tfrac{1}{2}\text{ in.})(\tfrac{3}{4}\text{ in.})$ $= 396\text{ kips} > 60\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{F_{cr}}{\Omega} l t_p$ $= (24.2\text{ ksi})(14\tfrac{1}{2}\text{ in.})(\tfrac{3}{4}\text{ in.})$ $= 263\text{ kips} > 40\text{ kips}$ **o.k.**

Column Reinforcement

As mentioned there are three options to correct the column web failure. These options are as follows:

1) Use a heavier column. This may not be practical because the steel may have been purchased and perhaps detailed and fabricated before the problem is found.

2) Use a web doubler plate. This plate would be fitted about the shear plate on the same side of the column web as the shear plate. This necessitates a lot of cutting, fitting and welding, and is therefore expensive.

3) Use stiffener or stabilizer plates—also called continuity plates. This is probably the most viable option, but changes the nature of the connection, because the stiffener plates will cause the column to be subjected to a moment. This cannot be avoided, but may be used advantageously.

Option 3 Solution

Because the added stiffeners cause the column to pick-up moment, the moment for which the connection is designed can be reduced.

The connection is designed as a conventional configuration shear plate with axial force for everything to the right of Section A-A as shown in Figure II.A-19B-4. The design to the left of Section A-A is performed following a procedure for Type II stabilizer plates presented in Fortney and Thornton (2016).

As shown in Figure II.A-19B-5, the moment in the shear plate to the left of Section A-A is uncoupled between the stabilizer plates.

$$V_s = \frac{Va'}{l}$$

where

$a' = 7$ in.

$l = 14\frac{1}{2}$ in.

$g = 2\frac{3}{4}$ in.

LRFD	ASD
$V_{us} = \dfrac{V_u a'}{l}$ $= \dfrac{(75 \text{ kips})(7 \text{ in.})}{14\frac{1}{2} \text{ in.}}$ $= 36.2 \text{ kips}$	$V_{as} = \dfrac{V_a a'}{L}$ $= \dfrac{(50 \text{ kips})(7 \text{ in.})}{14\frac{1}{2} \text{ in.}}$ $= 24.1 \text{ kips}$

The force between the shear plate and stabilizer plate is determined as follows:

LRFD	ASD
$F_{up} = V_{us} + \dfrac{N_u}{2}$ $= 36.2 \text{ kips} + \dfrac{60 \text{ kips}}{2}$ $= 66.2 \text{ kips}$	$F_{ap} = V_{as} + \dfrac{N_a}{2}$ $= 24.1 \text{ kips} + \dfrac{40 \text{ kips}}{2}$ $= 44.1 \text{ kips}$

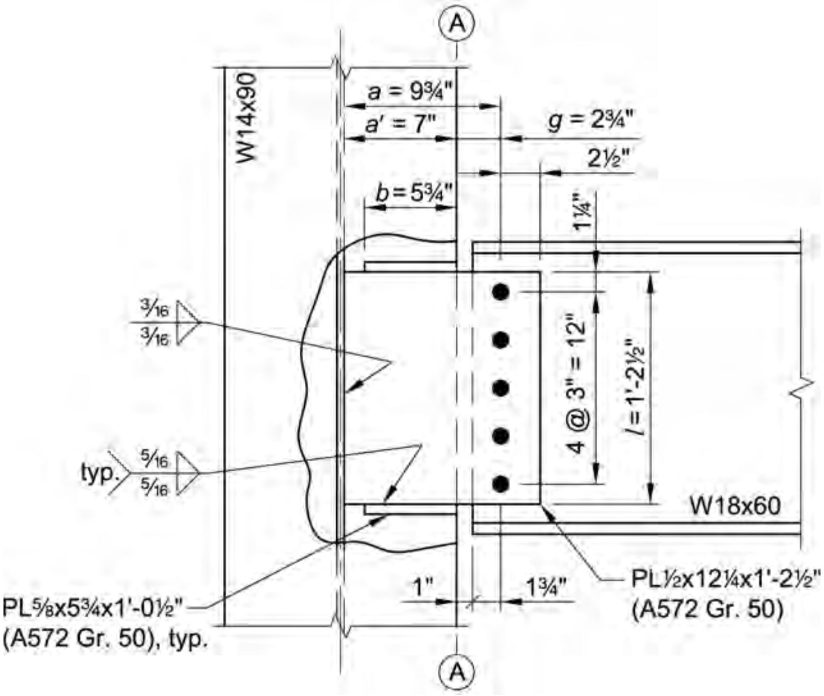


Fig. II.A-19B-4. Design of shear plate with stabilizer plates.

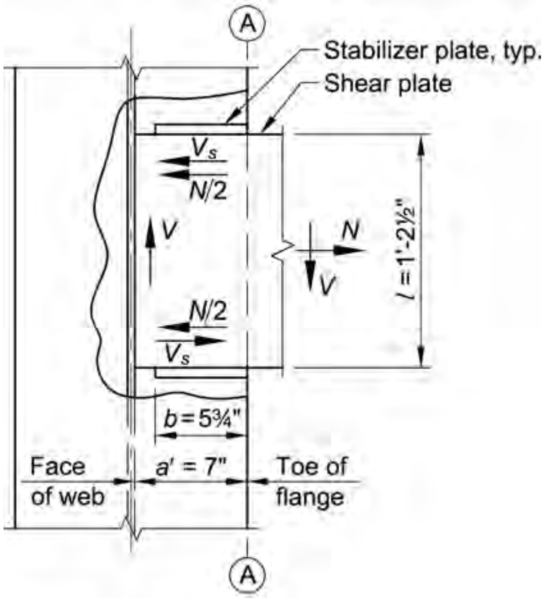


Fig. II.A-19B-5. Forces acting on shear plate.

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Stabilizer Plate Design

The stabilizer plate design is shown in Figure II.A-19B-6. The forces in the stabilizer plate are calculated as follows:

LRFD	ASD
Shear: $V_u = \dfrac{F_{up}}{2}$ $= \dfrac{66.2 \text{ kips}}{2}$ $= 33.1 \text{ kips}$	Shear: $V_a = \dfrac{F_{ap}}{2}$ $= \dfrac{44.1 \text{ kips}}{2}$ $= 22.1 \text{ kips}$
Moment: $M_u = \dfrac{F_{up} w}{4}$ $= \dfrac{(66.2 \text{ kips})(12\tfrac{1}{2}\text{in.})}{4}$ $= 207 \text{ kip-in.}$	Moment: $M_a = \dfrac{F_{ap} w}{4}$ $= \dfrac{(44.1 \text{ kips})(12\tfrac{1}{2}\text{in.})}{4}$ $= 138 \text{ kip-in.}$

Try ⅝-in.-thick stabilizer plates. The available shear strength of the stabilizer plate is determined using AISC *Specification* Section J4.2 as follows:

$$\begin{aligned} A_{nv} &= bt \\ &= (5\tfrac{3}{4} \text{ in.})(\tfrac{5}{8} \text{ in.}) \\ &= 3.59 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60 F_u A_{nv} \\ &= 0.60(65 \text{ ksi})(3.59 \text{ in.}^2) \\ &= 140 \text{ kips} \end{aligned}$$

(*Spec*. Eq. J4-4)

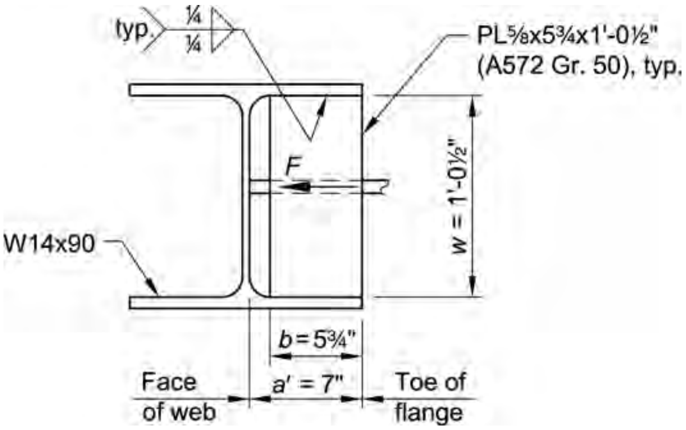

Fig. II.A-19B-6. Stabilizer plate design.

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(140\text{ kips})$ $= 105\text{ kips} > 33.1\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{140\text{ kips}}{2.00}$ $= 70.0\text{ kips} > 22.1\text{ kips}$ **o.k.**

The available flexural strength of the stabilizer plate is determined as follows:

$$\begin{aligned} M_n &= F_y Z_x \\ &= (50\text{ ksi})\left[\frac{(\tfrac{5}{8}\text{ in.})(5\tfrac{3}{4}\text{ in.})^2}{4}\right] \\ &= 258\text{ kip-in.} \end{aligned}$$

LRFD	ASD
$\phi = 0.90$ $\phi M_n = 0.90(258\text{ kip-in.})$ $= 232\text{ kip-in.} > 207\text{ kip-in.}$ **o.k.**	$\Omega = 1.67$ $\dfrac{M_n}{\Omega} = \dfrac{258\text{ kip-in.}}{1.67}$ $= 154\text{ kip-in.} > 138\text{ kip-in.}$ **o.k.**

Stabilizer Plate to Column Weld Design

The required weld size between the stabilizer plate and column flanges is determined using AISC *Manual* Equations 8-2a or 8-2b as follows:

LRFD	ASD
$D_{req} = \dfrac{F_{up}/2}{(2\text{ welds})(1.392\text{ kip/in.})b}$ $= \dfrac{(66.2\text{ kips}/2)}{(2\text{ welds})(1.392\text{ kip/in.})(5\tfrac{3}{4}\text{ in.})}$ $= 2.07\text{ sixteenths}$	$D_{req} = \dfrac{F_{ap}/2}{(2\text{ welds})(0.928\text{ kip/in.})b}$ $= \dfrac{(44.1\text{ kips}/2)}{(2\text{ welds})(0.928\text{ kip/in.})(5\tfrac{3}{4}\text{ in.})}$ $= 2.07\text{ sixteenths}$

The minimum weld size per AISC *Specification* Table J2.4 controls. Use ¼-in. fillet welds.

Shear Plate to Stabilizer Plate Weld Design

The required weld size between the shear plate and stabilizer plates is determined using AISC *Manual* Equations 8-2a or 8-2b as follows:

LRFD	ASD
$D_{req} = \dfrac{F_{up}}{(2\text{ welds})(1.392\text{ kip/in.})l_w}$ $= \dfrac{66.2\text{ kips}}{(2\text{ welds})(1.392\text{ kip/in.})(5\tfrac{3}{4}\text{ in.})}$ $= 4.14\text{ sixteenths}$	$D_{req} = \dfrac{F_{ap}}{(2\text{ welds})(0.928\text{ kip/in.})l_w}$ $= \dfrac{44.1\text{ kips}}{(2\text{ welds})(0.928\text{ kip/in.})(5\tfrac{3}{4}\text{ in.})}$ $= 4.13\text{ sixteenths}$

Use 5⁄16-in. fillet welds.

Strength of Shear Plate at Stabilizer Plate Welds

The minimum shear plate thickness that will match the shear rupture strength of the weld is:

$$\begin{aligned} t_{min} &= \frac{6.19D}{F_u} \\ &= \frac{6.19(4.14)}{65 \text{ ksi}} \\ &= 0.394 \text{ in.} < \tfrac{1}{2} \text{ in.} \quad \textbf{o.k.} \end{aligned}$$

(*Manual* Eq. 9-3)

Shear Plate to Column Web Weld Design

The shear plate to stabilizer plate welds act as “crack arrestors” for the shear plate to column web welds. As shown in Figure II.A-19B-7, the required shear force is V. The required weld size is determined using AISC *Manual* Equations 8-2a or 8-2b as follows:

LRFD	ASD
$V_u = 75 \text{ kips}$	$V_a = 50 \text{ kips}$
$D_{req} = \dfrac{V_u}{(2 \text{ welds})(1.392 \text{ kip/in.})l}$	$D_{req} = \dfrac{V_a}{(2 \text{ welds})(0.928 \text{ kip/in.})l}$
$= \dfrac{75 \text{ kips}}{(2 \text{ welds})(1.392 \text{ kip/in.})(14\tfrac{1}{2} \text{ in.})}$	$= \dfrac{50 \text{ kips}}{(2 \text{ welds})(0.928 \text{ kip/in.})(14\tfrac{1}{2} \text{ in.})}$
$= 1.86 \text{ sixteenths}$	$= 1.86 \text{ sixteenths}$

The minimum weld size per AISC *Specification* Table J2.4 controls. Use $\tfrac{3}{16}$-in. fillet welds.

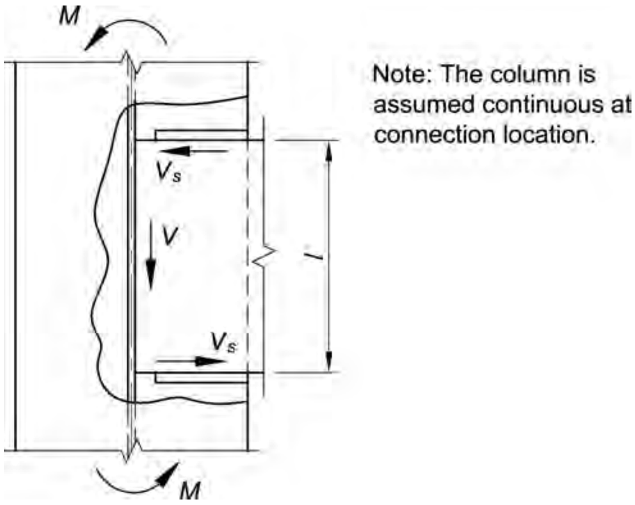

Fig. II.A-19B-7. Moment induced in column.

Strength of Shear Plate at Column Web Welds

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the shear plate is determined as follows:

$$\begin{aligned} A_{nv} &= lt \\ &= (14\tfrac{1}{2}\text{ in.})(\tfrac{1}{2}\text{ in.}) \\ &= 7.25\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60F_uA_{nv} \\ &= 0.60(65\text{ ksi})(7.25\text{ in.}^2) \\ &= 283\text{ kips} \end{aligned}$$ (*Spec*. Eq. J4-4)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(283\text{ kips})$ $= 212\text{ kips} > 75\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{283\text{ kips}}{2.00}$ $= 142\text{ kips} > 50\text{ kips}$ **o.k.**

Moment in Column

The moment in the column is determined as follows:

LRFD	ASD
$2M_u = V_{us}l$ $= (36.2\text{ kips})(14\tfrac{1}{2}\text{ in.})$ $= 525\text{ kip-in.}$ $M_u = 263\text{ kip-in.}$	$2M_a = V_{as}l$ $= (24.1\text{ kips})(14\tfrac{1}{2}\text{ in.})$ $= 349\text{ kip-in.}$ $M_a = 175\text{ kip-in.}$

The column design needs to be reviewed to ensure that this moment does not overload the column.

Reference

Fortney, P. and Thornton, W. (2016), “Analysis and Design of Stabilizer Plates in Single-Plate Shear Connections,” *Engineering Journal*, AISC, Vol. 53, No. 1, pp. 1–18.

EXAMPLE II.A-20 ALL-BOLTED SINGLE-PLATE SHEAR SPLICE

Given:

Verify an all-bolted single-plate shear splice between two ASTM A992 beams, as shown in Figure II.A-20-1, to support the following beam end reactions:

$R_D = 10$ kips
$R_L = 30$ kips

Use ASTM A36 plate.

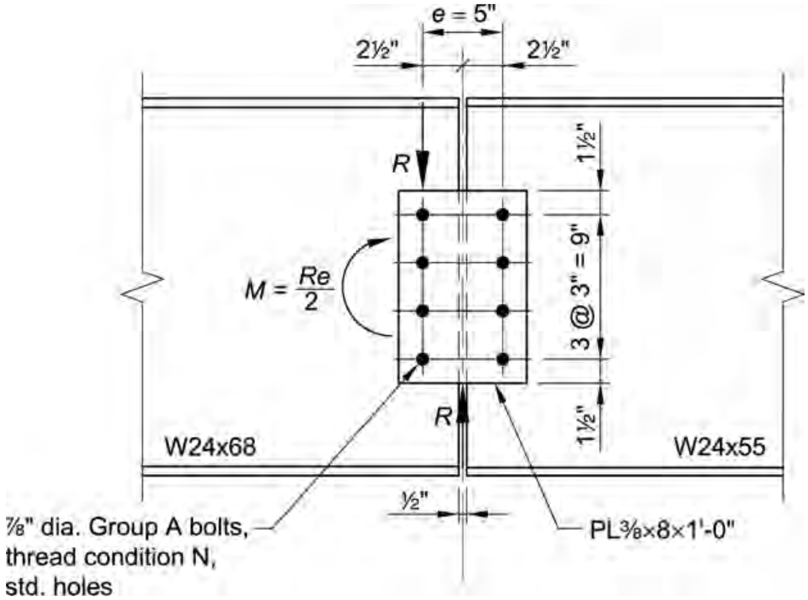

Fig. II.A-20-1. Connection geometry for Example II.A-20.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Beam and column
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Plate
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W24×55
$t_w = 0.395$ in.

Beam
W24×68
$t_w = 0.415$ in.

From AISC *Specification* Table J3.3, for ⅞-in.-diameter bolts with standard holes:

$d_h = {}^{15}\!/_{16}$ in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(10 \text{ kips}) + 1.6(30 \text{ kips})$ $= 60.0 \text{ kips}$	$R_a = 10 \text{ kips} + 30 \text{ kips}$ $= 40.0 \text{ kips}$

Strength of the Bolted Connection—Plate

Note: When the splice is symmetrical, the eccentricity of the shear to the center of gravity of either bolt group is equal to half the distance between the centroids of the bolt groups. Therefore, each bolt group can be designed for the shear, R_u or R_a, and one-half the eccentric moment, R_ue or R_ae.

Using a symmetrical splice, each bolt group will carry one-half the eccentric moment. Thus, the eccentricity on each bolt group is determined as follows:

$$\begin{aligned}\frac{e}{2} &= \frac{5 \text{ in.}}{2}\\ &= 2.50 \text{ in.}\end{aligned}$$

From the Commentary to AISC *Specification* Section J3.6, the strength of the bolt group is taken as the sum of the invividual strengths of the individual fasteners, which may be taken as the lesser of the fastener shear strength per AISC *Specification* Section J3.6, the bearing strength at the bolt hole per AISC *Specification* Section J3.10 or the tearout strength at the bolt hole per AISC *Specification* Section J3.10.

From AISC *Manual* Table 7-1, the available shear strength per bolt for ⅞-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) is:

LRFD	ASD
$\phi r_n = 24.3 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 16.2 \text{ kips/bolt}$

The available bearing strength of the plate per bolt is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration:

$$\begin{aligned}r_n &= 2.4dtF_u \\ &= (2.4)(\tfrac{7}{8} \text{ in.})(\tfrac{3}{8} \text{ in.})(58 \text{ ksi}) \\ &= 45.7 \text{ kips/bolt}\end{aligned}$$ (*Spec*. Eq. J3-6a)

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(45.7 \text{ kips/bolt})$ $= 34.3 \text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{45.7 \text{ kips/bolt}}{2.00}$ $= 22.9 \text{ kips/bolt}$

The available tearout strength of the plate per bolt is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration. Note: The available tearout strength based on edge distance will conservatively be used for all of the bolts.

$$
\begin{aligned}
l_c &= l_{ev} - 0.5(d_h) \\
&= 1\tfrac{1}{2}\text{ in.} - 0.5\left(\tfrac{15}{16}\text{ in.}\right) \\
&= 1.03\text{ in.}
\end{aligned}
$$

$$
\begin{aligned}
r_n &= 1.2 l_c t F_u \qquad \text{(}\textit{Spec}\text{. Eq. J3-6c)} \\
&= 1.2(1.03\text{ in.})\left(\tfrac{3}{8}\text{ in.}\right)(58\text{ ksi}) \\
&= 26.9\text{ kips/bolt}
\end{aligned}
$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(26.9\text{ kips/bolt})$ $= 20.2\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{26.9\text{ kips/bolt}}{2.00}$ $= 13.5\text{ kips/bolt}$

The tearout strength controls over bearing and shear for bolts in the plate.

The available strength of the bolt group is determined by interpolating AISC *Manual* Table 7-6, with $n = 4$, Angle = 0°, and $e_x = 2\tfrac{1}{2}$ in.

$$C = 3.07$$

LRFD	ASD
$C_{min} = \dfrac{R_u}{\phi r_n}$ $= \dfrac{60.0\text{ kips}}{20.2\text{ kips/bolt}}$ $= 2.97 < 3.07$ **o.k.**	$C_{min} = \dfrac{R_a}{r_n / \Omega}$ $= \dfrac{40.0\text{ kips}}{13.5\text{ kips/bolt}}$ $= 2.96 < 3.07$ **o.k.**

Strength of the Bolted Connection—Beam Web

By inspection, bearing and tearout on the webs of the beams will not govern.

Flexural Yielding of Plate

The required flexural strength is determined as follows:

LRFD	ASD
$M_u = \dfrac{R_u e}{2}$ $= \dfrac{(60.0\text{ kips})(5\text{ in.})}{2}$ $= 150\text{ kip-in.}$	$M_a = \dfrac{R_a e}{2}$ $= \dfrac{(40.0\text{ kips})(5\text{ in.})}{2}$ $= 100\text{ kip-in.}$

The available flexural strength is determined as follows:

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LRFD	ASD
$\phi = 0.90$ $\phi M_n = \phi F_y Z_x$ $= 0.90(36\text{ ksi})\left[\dfrac{(3/8\text{ in.})(12\text{ in.})^2}{4}\right]$ $= 437\text{ kip-in.} > 150\text{ kip-in.}$ **o.k.**	$\Omega = 1.67$ $\dfrac{M_n}{\Omega} = \dfrac{F_y Z_x}{\Omega}$ $= \dfrac{36\text{ ksi}}{1.67}\left[\dfrac{(3/8\text{ in.})(12\text{ in.})^2}{4}\right]$ $= 291\text{ kip-in.} > 100\text{ kip-in.}$ **o.k.**

Flexural Rupture of Plate

The net plastic section modulus of the plate, Z_{net}, is determined from AISC *Manual* Table 15-3:

$$Z_{net} = 9.00\text{ in.}^3$$

$$\begin{aligned} M_n &= F_u Z_{net} \\ &= (58\text{ ksi})(9.00\text{ in.}^3) \\ &= 522\text{ kip-in.} \end{aligned} \qquad (\textit{Manual}\text{ Eq. 9-4})$$

LRFD	ASD
$\phi = 0.75$ $\phi M_n = 0.75(522\text{ kip-in.})$ $= 392\text{ kip-in.} > 150\text{ kip-in.}$ **o.k.**	$\Omega = 2.00$ $\phi M_n = \dfrac{522\text{ kip-in.}}{2.00}$ $= 261\text{ kip-in.} > 100\text{ kip-in.}$ **o.k.**

Shear Strength of Plate

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the plate is determined as follows:

$$\begin{aligned} A_{gv} &= lt \\ &= (12\text{ in.})(3/8\text{ in.}) \\ &= 4.50\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60 F_y A_{gv} \\ &= 0.60(36\text{ ksi})(4.50\text{ in.}^2) \\ &= 97.2\text{ kips} \end{aligned} \qquad (\textit{Spec.}\text{ Eq. J4-3})$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(97.2\text{ kips})$ $= 97.2\text{ kips} > 60.0\text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{97.2\text{ kips}}{1.50}$ $= 64.8\text{ kips} > 40.0\text{ kips}$ **o.k.**

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the plate is determined using the net area determined in accordance with AISC *Specification* Section B4.3b.

$$\begin{aligned}A_{nv} &= \left[d - n\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t \\ &= \left[12\text{ in.} - 4\left(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right]\left(\tfrac{3}{8}\text{ in.}\right) \\ &= 3.00\text{ in.}^2\end{aligned}$$

$$R_n = 0.60F_uA_{nv} \qquad (Spec.\text{ Eq. J4-4})$$

$$\begin{aligned} &= 0.60\left(58\text{ ksi}\right)\left(3.00\text{ in.}^2\right) \\ &= 104\text{ kips}\end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(104\text{ kips})$ $= 78.0\text{ kips} > 60.0\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{104\text{ kips}}{2.00}$ $= 52.0\text{ kips} > 40.0\text{ kips}$ **o.k.**

Block Shear Rupture of Plate

The nominal strength for the limit state of block shear rupture is given by AISC *Specification* Section J4.3.

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad (Spec.\text{ Eq. J4-5})$$

The available block shear rupture strength of the plate is determined as follows, using AISC *Manual* Tables 9-3a, 9-3b and 9-3c, and AISC *Specification* Equation J4-5, with $n = 4$, $l_{eh} = l_{ev} = 1\tfrac{1}{2}$ in., and $U_{bs} = 1.0$.

LRFD	ASD
Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{\phi F_uA_{nt}}{t} = 43.5\text{ kip/in.}$	Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{F_uA_{nt}}{\Omega t} = 29.0\text{ kip/in.}$
Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{\phi 0.60F_yA_{gv}}{t} = 170\text{ kip/in.}$	Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{0.60F_yA_{gv}}{\Omega t} = 113\text{ kip/in.}$
Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{\phi 0.60F_uA_{nv}}{t} = 183\text{ kip/in.}$	Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{0.60F_uA_{nv}}{\Omega t} = 122\text{ kip/in.}$

LRFD	ASD
$\phi R_n = \phi 0.60 F_u A_{nv} + \phi U_{bs} F_u A_{nt}$ $\leq \phi 0.60 F_y A_{gv} + \phi U_{bs} F_u A_{nt}$ $= (3\!/\!8 \text{ in.})\left[183 \text{ kip/in.} + (1.0)(43.5 \text{ kip/in.})\right]$ $\leq (3\!/\!8 \text{ in.})\left[170 \text{ kip/in.} + (1.0)(43.5 \text{ kip/in.})\right]$ $= 84.9 \text{ kips} > 80.1 \text{ kips}$ Therefore: $\phi R_n = 80.1 \text{ kips} > 60.0 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{0.60 F_u A_{nv}}{\Omega} + \dfrac{U_{bs} F_u A_{nt}}{\Omega}$ $\leq \dfrac{0.60 F_y A_{gv}}{\Omega} + \dfrac{U_{bs} F_u A_{nt}}{\Omega}$ $= (3\!/\!8 \text{ in.})\left[122 \text{ kip/in.} + (1.0)(29.0 \text{ kip/in.})\right]$ $\leq (3\!/\!8 \text{ in.})\left[113 \text{ kip/in.} + (1.0)(29.0 \text{ kip/in.})\right]$ $= 56.6 \text{ kips} > 53.3 \text{ kips}$ Therefore: $\dfrac{R_n}{\Omega} = 53.3 \text{ kips} > 40.0 \text{ kips}$ **o.k.**

Conclusion

The connection is found to be adequate as given for the applied force.

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EXAMPLE II.A-21 BOLTED/WELDED SINGLE-PLATE SHEAR SPLICE

Given:

Verify a single-plate shear splice between between two ASTM A992 beams, as shown in Figure II.A-21-1, to support the following beam end reactions:

$R_D = 8$ kips
$R_L = 24$ kips

Use an ASTM A36 plate and 70-ksi electrodes.

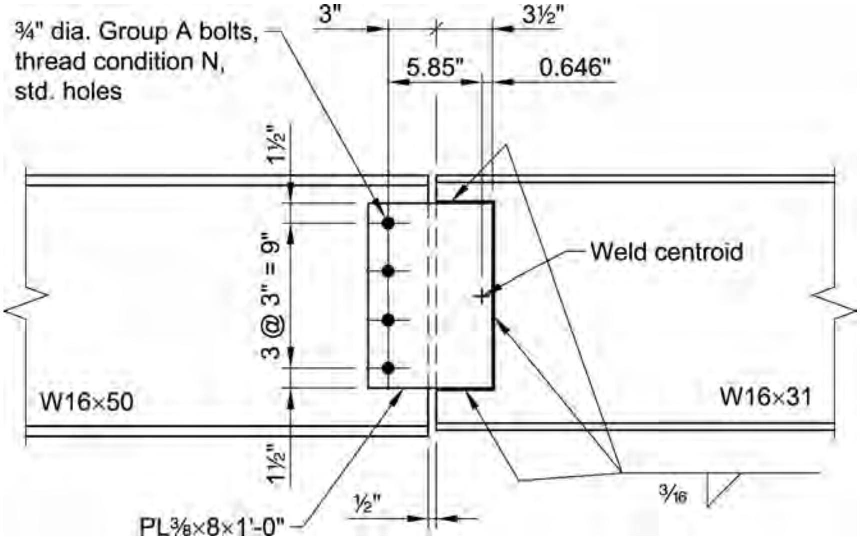

Fig. II.A-21-1. Connection geometry for Example II.A-21.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Beam
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Plate
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W16×31
$t_w = 0.275$ in.

Beam
W16×50
$t_w = 0.380$ in.

From AISC *Specification* Table J3.3, for ¾-in.-diameter bolts with standard holes:

d_h = 13⁄16 in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2\left(8\text{ kips}\right)+1.6\left(24\text{ kips}\right)$ $= 48.0\text{ kips}$	$R_a = 8\text{ kips}+24\text{ kips}$ $= 32.0\text{ kips}$

Strength of the Welded Connection—Plate

Because the splice is unsymmetrical and the weld group is more rigid, it will be designed for the full moment from the eccentric shear.

Use a PL⅜ in.×8 in.×1 ft 0 in. This plate size meets the dimensional and other limitations of a single-plate connection with a conventional configuration from AISC *Manual* Part 10.

Use AISC *Manual* Table 8-8 to determine the weld size.

$$\begin{aligned} k &= \frac{kl}{l} \\ &= \frac{3\tfrac{1}{2}\text{ in.}}{12\text{ in.}} \\ &= 0.292 \end{aligned}$$

Interpolating from AISC *Manual* Table 8-8, with Angle = 0°, and $k = 0.292$:

$$x = 0.0538$$

$$\begin{aligned} xl &= \left(0.0538\right)\left(12\text{ in.}\right) \\ &= 0.646\text{ in.} \end{aligned}$$

$$\begin{aligned} e_x &= al \\ &= 6.50\text{ in.} - 0.646\text{ in.} \\ &= 5.85\text{ in.} \end{aligned}$$

$$\begin{aligned} a &= \frac{al}{l} \\ &= \frac{5.85\text{ in.}}{12\text{ in.}} \\ &= 0.488 \end{aligned}$$

By interpolating AISC *Manual* Table 8-8, with Angle = 0°:

$$C = 2.15$$

From AISC *Manual* Equation 8-21, with C_1 = 1.00 from AISC *Manual* Table 8-3, the required weld size is determined as follows:

LRFD	ASD
$D_{min} = \dfrac{R_u}{\phi C C_1 l}$ $= \dfrac{48.0 \text{ kips}}{0.75(2.15)(1.00)(12 \text{ in.})}$ $= 2.48 \rightarrow 3 \text{ sixteenths}$	$D_{min} = \dfrac{\Omega R_a}{C C_1 l}$ $= \dfrac{(2.00)(32.0 \text{ kips})}{2.15(1.00)(12 \text{ in.})}$ $= 2.48 \rightarrow 3 \text{ sixteenths}$

The minimum required weld size from AISC *Specification* Table J2.4 is 3⁄16 in.

Use a 3⁄16-in. fillet weld.

Shear Rupture of W16×31 Beam Web at Weld

For fillet welds with F_{EXX} = 70 ksi on one side of the connection, the minimum thickness required to match the available shear rupture strength of the connection element to the available shear rupture strength of the base metal is:

$$t_{min} = \frac{3.09D}{F_u} \qquad \text{(}\textit{Manual}\text{ Eq. 9-2)}$$

$$= \frac{3.09(2.48)}{65 \text{ ksi}}$$

$$= 0.118 \text{ in.} < 0.275 \text{ in.} \quad \textbf{o.k.}$$

Strength of the Bolted Connection—Plate

From the Commentary to AISC *Specification* Section J3.6, the strength of the bolt group is taken as the sum of the invividual strengths of the individual fasteners, which may be taken as the lesser of the fastener shear strength per AISC *Specification* Section J3.6, the bearing strength at the bolt hole per AISC *Specification* Section J3.10, or the tearout strength at the bolt hole per AISC *Specification* Section J3.10.

From AISC *Manual* Table 7-1, the available shear strength per bolt for ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) is:

LRFD	ASD
$\phi r_n = 17.9 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 11.9 \text{ kips/bolt}$

The available bearing strength of the plate per bolt is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration:

$$r_n = 2.4dtF_u \qquad \text{(}\textit{Spec.}\text{ Eq. J3-6a)}$$

$$= (2.4)(\tfrac{3}{4} \text{ in.})(\tfrac{3}{8} \text{ in.})(58 \text{ ksi})$$

$$= 39.2 \text{ kips/bolt}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(39.2 \text{ kips/bolt})$ $= 29.4 \text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{39.2 \text{ kips/bolt}}{2.00}$ $= 19.6 \text{ kips/bolt}$

The available tearout strength of the plate per bolt is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration. Note: The available tearout strength based on edge distance will conservatively be used for all of the bolts.

$$\begin{aligned} l_c &= l_{ev} - 0.5\left(d_h\right) \\ &= 1\tfrac{1}{2}\text{ in.} - 0.5\left(\tfrac{13}{16}\text{ in.}\right) \\ &= 1.09\text{ in.} \end{aligned}$$

$$\begin{aligned} r_n &= 1.2 l_c t F_u && \text{(}\textit{Spec.}\text{ Eq. J3-6c)} \\ &= 1.2\left(1.09\text{ in.}\right)\left(\tfrac{3}{8}\text{ in.}\right)\left(58\text{ ksi}\right) \\ &= 28.4\text{ kips/bolt} \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75\left(28.4\text{ kips/bolt}\right)$ $= 21.3\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{28.4\text{ kips/bolt}}{2.00}$ $= 14.2\text{ kips/bolt}$

The bolt shear strength controls for bolts in the plate.

Because the weld group is designed for the full eccentric moment, the bolt group is designed for shear only.

LRFD	ASD
$n_{min} = \dfrac{R_u}{\phi r_n}$ $= \dfrac{48.0\text{ kips}}{17.9\text{ kips/bolt}}$ $= 2.68\text{ bolts} < 4\text{ bolts}$ **o.k.**	$n_{min} = \dfrac{R_a}{r_n / \Omega}$ $= \dfrac{32.0\text{ kips}}{11.9\text{ kips/bolt}}$ $= 2.69\text{ bolts} < 4\text{ bolts}$ **o.k.**

Strength of the Bolted Connection—Beam Web

By inspection, bearing and tearout on the W16×50 beam web will not govern.

Flexural Yielding of Plate

The required flexural strength of the plate is determined as follows:

LRFD	ASD
$M_u = R_u e_x$ $= \left(48.0\text{ kips}\right)\left(5.85\text{ in.}\right)$ $= 281\text{ kip-in.}$	$M_a = R_a e_x$ $= \left(32.0\text{ kips}\right)\left(5.85\text{ in.}\right)$ $= 187\text{ kip-in.}$

The available flexural strength of the plate is determined as follows:

LRFD	ASD
$\phi = 0.90$ $\phi M_n = \phi F_y Z_x$ $= 0.90(36\text{ ksi})\left[\dfrac{(\text{⅜ in.})(12\text{ in.})^2}{4}\right]$ $= 437\text{ kip-in.} > 281\text{ kip-in.}$ **o.k.**	$\Omega = 1.67$ $\dfrac{M_n}{\Omega} = \dfrac{F_y Z_x}{\Omega}$ $= \dfrac{36\text{ ksi}}{1.67}\left[\dfrac{(\text{⅜ in.})(12\text{ in.})^2}{4}\right]$ $= 291\text{ kip-in.} > 187\text{ kip-in.}$ **o.k.**

Shear Strength of Plate

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the plate is determined as follows:

$$\begin{aligned} A_{gv} &= lt \\ &= (12\text{ in.})(\text{⅜ in.}) \\ &= 4.50\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60F_y A_{gv} \\ &= 0.60(36\text{ ksi})(4.50\text{ in.}^2) \\ &= 97.2\text{ kips} \end{aligned}$$ (*Spec*. Eq. J4-3)

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(97.2\text{ kips})$ $= 97.2\text{ kips} > 48.0\text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{97.2\text{ kips}}{1.50}$ $= 64.8\text{ kips} > 32.0\text{ kips}$ **o.k.**

From AISC *Specification* Section J4.2, the available shear rupture strength of the plate is determined using the net area determined in accordance with AISC *Specification* Section B4.3b.

$$\begin{aligned} A_{nv} &= \left[d - n(d_h + \text{¹⁄₁₆ in.})\right]t \\ &= \left[12\text{ in.} - 4(\text{¹³⁄₁₆ in.} + \text{¹⁄₁₆ in.})\right](\text{⅜ in.}) \\ &= 3.19\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60F_u A_{nv} \\ &= 0.60(58\text{ ksi})(3.19\text{ in.}^2) \\ &= 111\text{ kips} \end{aligned}$$ (*Spec*. Eq. J4-4)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(111\text{ kips})$ $= 83.3\text{ kips} > 48.0\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{111\text{ kips}}{2.00}$ $= 55.5\text{ kips} > 32.0\text{ kips}$ **o.k.**

Block Shear Rupture of Plate

The nominal strength for the limit state of block shear rupture is given by AISC *Specification* Section J4.3.

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad \text{(}\textit{Spec.}\text{ Eq. J4-5)}$$

The available block shear rupture strength of the plate is determined as follows, using AISC *Manual* Tables 9-3a, 9-3b and 9-3c, and AISC *Specification* Equation J4-5, with $n = 4$, $l_{eh} = l_{ev} = 1\frac{1}{2}$ in., and $U_{bs} = 1.0$.

LRFD	ASD
Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{\phi F_uA_{nt}}{t} = 46.2 \text{ kip/in.}$	Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{F_uA_{nt}}{\Omega t} = 30.8 \text{ kip/in.}$
Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{\phi 0.60F_yA_{gv}}{t} = 170 \text{ kip/in.}$	Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{0.60F_yA_{gv}}{\Omega t} = 113 \text{ kip/in.}$
Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{\phi 0.60F_uA_{nv}}{t} = 194 \text{ kip/in.}$	Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{0.60F_uA_{nv}}{\Omega t} = 129 \text{ kip/in.}$
$\phi R_n = \phi 0.60F_uA_{nv} + \phi U_{bs}F_uA_{nt}$ $\le \phi 0.60F_yA_{gv} + \phi U_{bs}F_uA_{nt}$ $= (\frac{3}{8} \text{ in.})\left[194 \text{ kip/in.} + (1.0)(46.2 \text{ kip/in.})\right]$ $\le (\frac{3}{8} \text{ in.})\left[170 \text{ kip/in.} + (1.0)(46.2 \text{ kip/in.})\right]$ $= 90.1 \text{ kips} > 81.1 \text{ kips}$	$\dfrac{R_n}{\Omega} = \dfrac{0.60F_uA_{nv}}{\Omega} + \dfrac{U_{bs}F_uA_{nt}}{\Omega}$ $\le \dfrac{0.60F_yA_{gv}}{\Omega} + \dfrac{U_{bs}F_uA_{nt}}{\Omega}$ $= (\frac{3}{8} \text{ in.})\left[129 \text{ kip/in.} + (1.0)(30.8 \text{ kip/in.})\right]$ $\le (\frac{3}{8} \text{ in.})\left[113 \text{ kip/in.} + (1.0)(30.8 \text{ kip/in.})\right]$ $= 59.9 \text{ kips} > 53.9 \text{ kips}$
Therefore: $\phi R_n = 81.1 \text{ kips} > 48.0 \text{ kips}$ **o.k.**	Therefore: $\dfrac{R_n}{\Omega} = 53.9 \text{ kips} > 32.0 \text{ kips}$ **o.k.**

Conclusion

The connection is found to be adequate as given for the applied force.

EXAMPLE II.A-22 BOLTED BRACKET PLATE DESIGN

Given:

Verify the bracket plate to support the loads as shown in Figure II.A-22-1 (loads are per bracket plate). Use ASTM A36 plate. Assume the column has sufficient available strength for the connection.

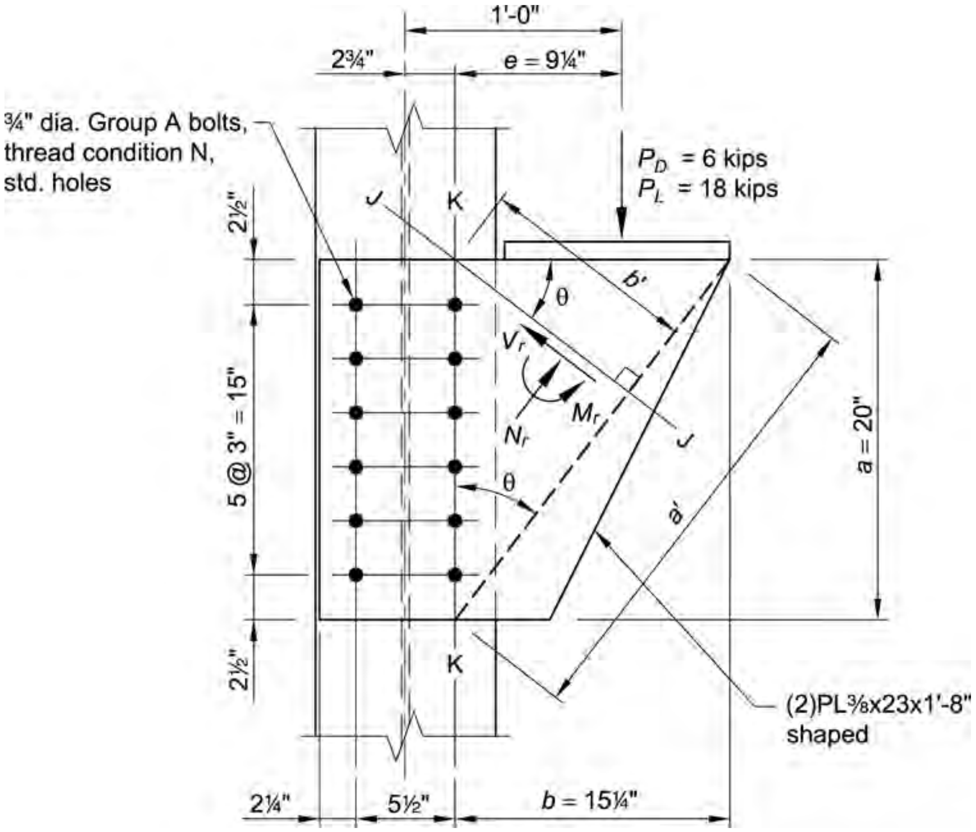

Fig. II.A-22-1. Connection geometry for Example II.A-22.

Solution:

For discussion of the design of a bracket plate, see AISC *Manual* Part 15.

From AISC *Manual* Table 2-5, the material properties are as follows:

Plate
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$P_u = 1.2(6\text{ kips}) + 1.6(18\text{ kips})$ $= 36.0\text{ kips}$	$P_a = 6\text{ kips} + 18\text{ kips}$ $= 24.0\text{ kips}$

From the geometry shown in Figure II.A-22-1 and AISC *Manual* Figure 15-2(b):

$a = 20\text{ in.}$
$b = 15\tfrac{1}{4}\text{ in.}$
$e = 9\tfrac{1}{4}\text{ in.}$

$$\begin{aligned}\theta &= \tan^{-1}\left(\frac{b}{a}\right)\\ &= \tan^{-1}\left(\frac{15\tfrac{1}{4}\text{ in.}}{20\text{ in.}}\right)\\ &= 37.3^\circ\end{aligned}$$

$$\begin{aligned}a' &= \frac{a}{\cos\theta}\\ &= \frac{20\text{ in.}}{\cos 37.3^\circ}\\ &= 25.1\text{ in.}\end{aligned}$$ (*Manual* Eq. 15-17)

$$\begin{aligned}b' &= a\sin\theta\\ &= (20\text{ in.})(\sin 37.3^\circ)\\ &= 12.1\text{ in.}\end{aligned}$$

Strength of the Bolted Connection—Plate

From the Commentary to AISC *Specification* Section J3.6, the strength of the bolt group is taken as the sum of the invividual strengths of the individual fasteners, which may be taken as the lesser of the fastener shear strength per AISC *Specification* Section J3.6, the bearing strength at the bolt hole per AISC *Specification* Section J3.10, or the tearout strength at the bolt hole per AISC *Specification* Section J3.10.

From AISC *Manual* Table 7-1, the available shear strength per bolt for ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) is:

LRFD	ASD
$\phi r_n = 17.9\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 11.9\text{ kips/bolt}$

The available bearing and tearout strength of the plate is determined using AISC *Manual* Table 7-5 conservatively using $l_e = 2$ in. Note: The available bearing and tearout strength based on edge distance will conservatively be used for all of the bolts.

LRFD	ASD
$\phi r_n = (78.3\text{ kip/in.})(\tfrac{3}{8}\text{ in.})$ $= 29.4\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (52.2\text{ kip/in.})(\tfrac{3}{8}\text{ in.})$ $= 19.6\text{ kips/bolt}$

Bolt shear strength controls for bolts in the plate.

The strength of the bolt group is determined by interpolating AISC *Manual* Table 7-8 with Angle = 0°, a 5½ in. gage with $s = 3$ in., $e_x = 12$ in. and $n = 6$:

$C = 4.53$

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$ $= \dfrac{36.0 \text{ kips}}{17.9 \text{ kips/bolt}}$ $= 2.01 < 4.53$ **o.k.**	$C_{min} = \dfrac{\Omega P_a}{r_n}$ $= \dfrac{24.0 \text{ kips}}{11.9 \text{ kips/bolt}}$ $= 2.02 < 4.53$ **o.k.**

Flexural Yielding of Bracket Plate on Section K-K

The required flexural yielding strength of the plate at Section K-K is determined from AISC *Manual* Equation 15-1a or 15-1b as follows:

LRFD	ASD
$M_u = P_u e$ $= (36.0 \text{ kips})(9\tfrac{1}{4} \text{ in.})$ $= 333 \text{ kip-in.}$	$M_a = P_a e$ $= (24.0 \text{ kips})(9\tfrac{1}{4} \text{ in.})$ $= 222 \text{ kip-in.}$

The available flexural yielding strength of the bracket plate is determined as follows:

$$\begin{aligned} M_n &= F_y Z && (\textit{Manual} \text{ Eq. 15-2}) \\ &= (36 \text{ ksi})\left[\frac{(\tfrac{3}{8} \text{ in.})(20 \text{ in.})^2}{4}\right] \\ &= 1{,}350 \text{ kip-in.} \end{aligned}$$

LRFD	ASD
$\phi = 0.90$ $\phi M_n = 0.90(1{,}350 \text{ kip-in.})$ $= 1{,}220 \text{ kip-in.} > 333 \text{ kip-in.}$ **o.k.**	$\Omega = 1.67$ $\dfrac{M_n}{\Omega} = \dfrac{1{,}350 \text{ kip-in.}}{1.67}$ $= 808 \text{ kip-in.} > 222 \text{ kip-in.}$ **o.k.**

Flexural Rupture of Bracket Plate on Section K-K

From AISC *Manual* Table 15-3, for a ⅜-in.-thick bracket plate, with ¾-in. bolts and six bolts in a row, $Z_{net} = 21.5$ in.^{3} Note that AISC *Manual* Table 15-3 conservatively considers $l_{ev} = 1\tfrac{1}{2}$ in. for holes spaced at 3 in.

The available flexural yielding rupture of the bracket plate at Section K-K is determined as follows:

$$\begin{aligned} M_n &= F_u Z_{net} && (\textit{Manual} \text{ Eq. 15-3}) \\ &= (58 \text{ ksi})(21.5 \text{ in.}^3) \\ &= 1{,}250 \text{ kip-in.} \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi M_n = 0.75(1{,}250 \text{ kip-in.})$ $= 938 \text{ kip-in.} > 333 \text{ kip-in.}$ **o.k.**	$\Omega = 2.00$ $\dfrac{M_n}{\Omega} = \dfrac{1{,}250 \text{ kip-in.}}{2.00}$ $= 625 \text{ kip-in.} > 222 \text{ kip-in.}$ **o.k.**

Shear Yielding of Bracket Plate on Section J-J

The required shear strength of the bracket plate on Section J-J is determined from AISC *Manual* Equation 15-6a or 15-6b as follows:

LRFD	ASD
$V_u = P_u \sin\theta$ $= (36.0 \text{ kips})(\sin 37.3°)$ $= 21.8 \text{ kips}$	$V_a = P_a \sin\theta$ $= (24.0 \text{ kips})(\sin 37.3°)$ $= 14.5 \text{ kips}$

The available shear yielding strength of the plate is determined as follows:

$$\begin{aligned} V_n &= 0.6F_y t b' \\ &= 0.6(36 \text{ ksi})(\tfrac{3}{8} \text{ in.})(12.1 \text{ in.}) \\ &= 98.0 \text{ kips} \end{aligned}$$ (*Manual* Eq. 15-7)

LRFD	ASD
$\phi = 1.00$ $\phi V_n = 1.00(98.0 \text{ kips})$ $= 98.0 \text{ kips} > 21.8 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{V_n}{\Omega} = \dfrac{98.0 \text{ kips}}{1.50}$ $= 65.3 \text{ kips} > 14.5 \text{ kips}$ **o.k.**

Local Yielding and Local Buckling of Bracket Plate on Section J-J (see Figure II.A-22-1)

For local yielding:

$$\begin{aligned} F_{cr} &= F_y \\ &= 36 \text{ ksi} \end{aligned}$$ (*Manual* Eq. 15-13)

For local buckling:

$$F_{cr} = QF_y$$ (*Manual* Eq. 15-14)

where

$$\lambda = \frac{\left(\dfrac{b'}{t}\right)\sqrt{F_y}}{5\sqrt{475 + 1{,}120\left(\dfrac{b'}{a'}\right)^2}}$$ (*Manual* Eq. 15-18)

$$\begin{aligned} &= \frac{\left(\dfrac{12.1 \text{ in.}}{\tfrac{3}{8} \text{ in.}}\right)\sqrt{36 \text{ ksi}}}{5\sqrt{475 + 1{,}120\left(\dfrac{12.1 \text{ in.}}{25.1 \text{ in.}}\right)^2}} \\ &= 1.43 \end{aligned}$$

Because $1.41 < \lambda$:

$$Q = \frac{1.30}{\lambda^2} = \frac{1.30}{(1.43)^2} = 0.636 \qquad \text{(}\textit{Manual}\text{ Eq. 15-16)}$$

$$F_{cr} = QF_y = 0.636(36\text{ ksi}) = 22.9\text{ ksi} \qquad \text{(}\textit{Manual}\text{ Eq. 15-14)}$$

Local buckling controls over local yielding.

Interaction of Normal and Flexural Strengths

Check that *Manual* Equation 15-10 is satisfied:

LRFD	ASD
$N_u = P_u \cos\theta$ (*Manual* Eq. 15-9a) $= (36.0\text{ kips})(\cos 37.3°)$ $= 28.6\text{ kips}$	$N_a = P_a \cos\theta$ (*Manual* Eq. 15-9b) $= (24.0\text{ kips})(\cos 37.3°)$ $= 19.1\text{ kips}$
$N_n = F_{cr}tb'$ (*Manual* Eq. 15-11) $= (22.9\text{ ksi})(3/8\text{ in.})(12.1\text{ in.})$ $= 104\text{ kips}$	$N_n = F_{cr}tb'$ (*Manual* Eq. 15-11) $= (22.9\text{ ksi})(3/8\text{ in.})(12.1\text{ in.})$ $= 104\text{ kips}$
$\phi = 0.90$	$\Omega = 1.67$
$N_c = \phi N_n$ $= 0.90(104\text{ kips})$ $= 93.6\text{ kips}$	$N_c = \dfrac{N_n}{\Omega}$ $= \dfrac{104\text{ kips}}{1.67}$ $= 62.3\text{ kips}$
$M_u = P_u e - N_u\left(\dfrac{b'}{2}\right)$ (*Manual* Eq. 15-8a) $= (36.0\text{ kips})(9\tfrac{1}{4}\text{ in.}) - (28.6\text{ kips})\left(\dfrac{12.1\text{ in.}}{2}\right)$ $= 160\text{ kip-in.}$	$M_a = P_a e - N_a\left(\dfrac{b'}{2}\right)$ (*Manual* Eq. 15-8b) $= (24.0\text{ kips})(9\tfrac{1}{4}\text{ in.}) - (19.1\text{ kips})\left(\dfrac{12.1\text{ in.}}{2}\right)$ $= 106\text{ kip-in.}$
$M_n = \dfrac{F_{cr}tb'^2}{4}$ (*Manual* Eq. 15-12) $= \dfrac{(22.9\text{ ksi})(3/8\text{ in.})(12.1\text{ in.})^2}{4}$ $= 314\text{ kip-in.}$	$M_n = \dfrac{F_{cr}tb'^2}{4}$ (*Manual* Eq. 15-12) $= \dfrac{(22.9\text{ ksi})(3/8\text{ in.})(12.1\text{ in.})^2}{4}$ $= 314\text{ kip-in.}$

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LRFD	ASD
$M_c = \phi M_n$ $= 0.90(314 \text{ kip-in.})$ $= 283 \text{ kip-in.}$	$M_c = \dfrac{M_n}{\Omega}$ $= \dfrac{314 \text{ kip-in.}}{1.67}$ $= 188 \text{ kip-in.}$
$\dfrac{N_r}{N_c} + \dfrac{M_r}{M_c} \le 1.0$ (*Manual* Eq. 15-10) $\dfrac{28.6 \text{ kips}}{93.6 \text{ kips}} + \dfrac{160 \text{ kip-in.}}{283 \text{ kip-in.}} = 0.871 < 1.0$ **o.k.**	$\dfrac{N_r}{N_c} + \dfrac{M_r}{M_c} \le 1.0$ (*Manual* Eq. 15-10) $\dfrac{19.1 \text{ kips}}{62.3 \text{ kips}} + \dfrac{106 \text{ kip-in.}}{188 \text{ kip-in.}} = 0.870 < 1.0$ **o.k.**

Shear Strength of Bracket Plate on Section K-K

From AISC *Specification* Section J4.2, the available shear yielding strength of the plate on Section K-K is determined as follows:

$$\begin{aligned} A_{gv} &= at \\ &= (20 \text{ in.})(\tfrac{3}{8} \text{ in.}) \\ &= 7.50 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60 F_y A_{gv} \\ &= 0.60(36 \text{ ksi})(7.50 \text{ in.}^2) \\ &= 162 \text{ kips} \end{aligned}$$ (*Spec*. Eq. J4-3)

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(162 \text{ kips})$ $= 162 \text{ kips} > 36.0 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{162 \text{ kips}}{1.50}$ $= 108 \text{ kips} > 24.0 \text{ kips}$ **o.k.**

From AISC *Specification* Section J4.2, the available shear rupture strength of the plate on Section K-K is determined as follows:

$$\begin{aligned} A_{nv} &= \left[a - n(\tfrac{15}{16} \text{ in.} + \tfrac{1}{16} \text{ in.})\right]t \\ &= \left[20 \text{ in.} - 6(\tfrac{13}{16} \text{ in.} + \tfrac{1}{16} \text{ in.})\right](\tfrac{3}{8} \text{ in.}) \\ &= 5.53 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60 F_u A_{nv} \\ &= 0.60(58 \text{ ksi})(5.53 \text{ in.}^2) \\ &= 192 \text{ kips} \end{aligned}$$ (*Spec*. Eq. J4-4)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(192 \text{ kips}\right)$ $= 144 \text{ kips} > 36.0 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{192 \text{ kips}}{2.00}$ $= 96.0 \text{ kips} > 24.0 \text{ kips}$ **o.k.**

Conclusion

The connection is found to be adequate as given for the applied force.

EXAMPLE II.A-23 WELDED BRACKET PLATE DESIGN

Given:

Verify the welded bracket plate to support the loads as shown in Figure II.A-23-1 (loads are resisted equally by the two bracket plates). Use ASTM A36 plate and 70-ksi electrodes. Assume the column has sufficient available strength for the connection.

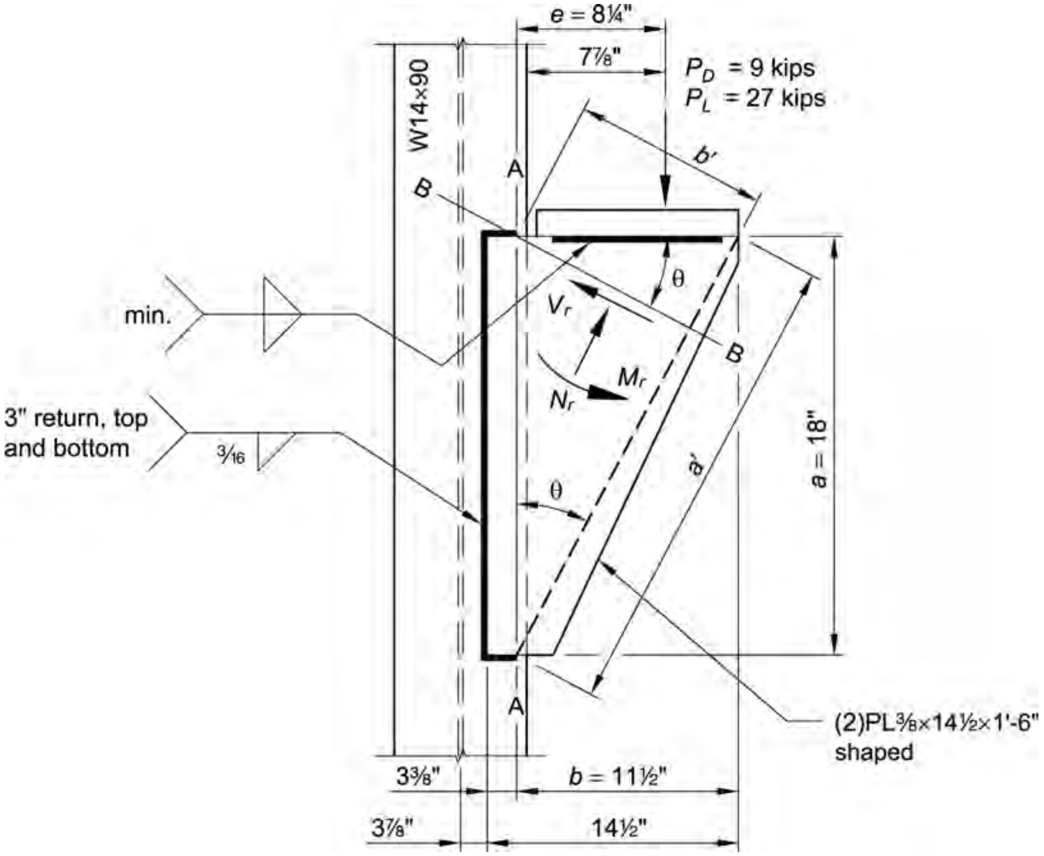

Fig. II.A-23-1. Connection geometry for Example II.A-23.

Solution:

From AISC *Manual* Table 2-5, the material properties are as follows:

Plate
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From ASCE/SEI 7, Chapter 2, the required strength to be resisted by the bracket plates is:

LRFD	ASD
$P_u = 1.2(9 \text{ kips}) + 1.6(27 \text{ kips})$ $= 54.0 \text{ kips}$	$P_a = 9 \text{ kips} + 27 \text{ kips}$ $= 36.0 \text{ kips}$

From the geometry shown in Figure II.A-23-1 and AISC *Manual* Figure 15-2(b):

$$a = 18\text{ in.}$$
$$b = 11\tfrac{1}{2}\text{ in.}$$
$$e = 8\tfrac{1}{4}\text{ in.}$$

$$\begin{aligned}\theta &= \tan^{-1}\left(\frac{b}{a}\right)\\ &= \tan^{-1}\left(\frac{11\tfrac{1}{2}\text{ in.}}{18\text{ in.}}\right)\\ &= 32.6^\circ\end{aligned}$$

$$\begin{aligned}a' &= \frac{a}{\cos\theta}\\ &= \frac{18\text{ in.}}{\cos 32.6^\circ}\\ &= 21.4\text{ in.}\end{aligned}$$ (*Manual* Eq. 15-17)

$$\begin{aligned}b' &= a\sin\theta\\ &= (18\text{ in.})(\sin 32.6^\circ)\\ &= 9.70\text{ in.}\end{aligned}$$

Shear Yielding of Bracket Plate at Section A-A

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the bracket plate at Section A-A, is determined as follows:

$$\begin{aligned}A_{gv} &= (2\text{ plates})at\\ &= (2\text{ plates})(18\text{ in.})(\tfrac{3}{8}\text{ in.})\\ &= 13.5\text{ in.}^2\end{aligned}$$

$$\begin{aligned}R_n &= 0.60F_yA_{gv}\\ &= 0.60(36\text{ ksi})(13.5\text{ in.}^2)\\ &= 292\text{ kips}\end{aligned}$$ (*Spec*. Eq. J4-3)

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(292\text{ kips})$ $= 292\text{ kips} > 54.0\text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{292\text{ kips}}{1.50}$ $= 195\text{ kips} > 36.0\text{ kips}$ **o.k.**

Shear rupture strength is adequate by insection.

Flexural Yielding of Bracket Plate at Section A-A

The required flexural strength of the bracket plate is determined using AISC *Manual* Equation 15-1a or 15-1b as follows:

LRFD	ASD
$M_u = P_u e$ $= (54.0 \text{ kips})(8\tfrac{1}{4} \text{ in.})$ $= 446 \text{ kip-in.}$	$M_a = P_a e$ $= (36.0 \text{ kips})(8\tfrac{1}{4} \text{ in.})$ $= 297 \text{ kip-in.}$

The available flexural strength of the bracket plate is determined using AISC *Manual* Equation 15-2, as follows:

$$\begin{aligned} M_n &= (2 \text{ plates}) F_y Z \\ &= (2 \text{ plates})(36 \text{ ksi})\left[\frac{(\tfrac{3}{8} \text{ in.})(18 \text{ in.})^2}{4}\right] \\ &= 2{,}190 \text{ kip-in.} \end{aligned} \qquad \text{(from } \textit{Manual} \text{ Eq. 15-2)}$$

LRFD	ASD
$\phi = 0.90$ $\phi M_n = 0.90(2{,}190 \text{ kip-in.})$ $= 1{,}970 \text{ kip-in.} > 446 \text{ kip-in.}$ **o.k.**	$\Omega = 1.67$ $\dfrac{M_n}{\Omega} = \dfrac{2{,}190 \text{ kip-in.}}{1.67}$ $= 1{,}310 \text{ kip-in.} > 297 \text{ kip-in.}$ **o.k.**

Weld Strength

Try a C-shaped weld with kl = 3 in. and l = 18 in.

$$\begin{aligned} k &= \frac{kl}{l} \\ &= \frac{3 \text{ in.}}{18 \text{ in.}} \\ &= 0.167 \end{aligned}$$

$$\begin{aligned} xl &= \frac{(kl)^2}{2(kl)+l} \\ &= \frac{(3 \text{ in.})^2}{2(3 \text{ in.})+18 \text{ in.}} \\ &= 0.375 \text{ in.} \end{aligned}$$

$$\begin{aligned} al &= 11\tfrac{1}{4} \text{ in.} - 0.375 \text{ in.} \\ &= 10.9 \text{ in.} \end{aligned}$$

$$\begin{aligned} a &= \frac{al}{l} \\ &= \frac{10.9 \text{ in.}}{18 \text{ in.}} \\ &= 0.606 \end{aligned}$$

Interpolate AISC *Manual* Table 8-8 using Angle = 0°, k = 0.167, and a = 0.606.

$$C = 1.46$$

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From AISC *Manual* Table 8-3:

$$C_1 = 1.00 \text{ (for E70 electrodes)}$$

The required weld size is determined using AISC *Manual* Equation 8-21, as follows:

LRFD	ASD
$\phi = 0.75$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $= \dfrac{54.0\text{ kips}}{0.75(1.46)(1.00)(18\text{ in.})(2\text{ plates})}$ $= 1.37 \rightarrow 3\text{ sixteenths}$	$\Omega = 2.00$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $= \dfrac{2.00(36.0\text{ kips})}{(1.46)(1.00)(18\text{ in.})(2\text{ plates})}$ $= 1.37 \rightarrow 3\text{ sixteenths}$

From AISC *Specification* Section J2.2b(b)(2), the maximum weld size is:

$$\begin{aligned} w_{max} &= \tfrac{3}{8}\text{ in.} - \tfrac{1}{16}\text{ in.} \\ &= \tfrac{5}{16}\text{ in.} > \tfrac{3}{16}\text{ in.} \quad \textbf{o.k.} \end{aligned}$$

From AISC *Specification* Table J2.4, the minimum weld size is:

$$w_{min} = \tfrac{3}{16}\text{ in.}$$

Shear Yielding Strength of Bracket at Section B-B

The required shear strength of the bracket plate at Section B-B is determined from AISC *Manual* Equations 15-6a or 15-6b as follows:

LRFD	ASD
$V_u = P_u \sin\theta$ $= (54.0\text{ kips})(\sin 32.6°)$ $= 29.1\text{ kips}$	$V_a = P_a \sin\theta$ $= (36.0\text{ kips})(\sin 32.6°)$ $= 19.4\text{ kips}$

From AISC *Manual* Part 15, the available shear yielding strength of the bracket plate at Section A-A is determined as follows:

$$\begin{aligned} V_n &= (2\text{ plates})0.6F_y t b' \qquad \text{(from } \textit{Manual} \text{ Eq. 15-7)} \\ &= (2\text{ plates})(0.6)(36\text{ ksi})(\tfrac{3}{8}\text{ in.})(9.70\text{ in.}) \\ &= 157\text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 1.00$ $\phi V_n = 1.00(157\text{ kips})$ $= 157\text{ kips} > 29.1\text{ kips} \quad \textbf{o.k.}$	$\Omega = 1.50$ $\dfrac{V_n}{\Omega} = \dfrac{157\text{ kips}}{1.50}$ $= 105\text{ kips} > 19.4\text{ kips} \quad \textbf{o.k.}$

Bracket Plate Normal and Flexural Strength at Section B-B

From AISC *Manual* Part 15, the required strength of the bracket plate at Section B-B is determined as follows:

LRFD	ASD
$N_u = P_u \cos\theta$ (*Manual* Eq. 15-9a) $= (54.0 \text{ kips})(\cos 32.6°)$ $= 45.5 \text{ kips}$ $M_u = P_u e - N_u \left(\frac{b'}{2}\right)$ (*Manual* Eq. 15-8a) $= (54.0 \text{ kips})(8\tfrac{1}{4} \text{ in.}) - (45.5 \text{ kips})\left(\frac{9.70 \text{ in.}}{2}\right)$ $= 225 \text{ kip-in.}$	$N_a = P_a \cos\theta$ (*Manual* Eq. 15-9b) $= (36.0 \text{ kips})(\cos 32.6°)$ $= 30.3 \text{ kips}$ $M_a = P_a e - N_a \left(\frac{b'}{2}\right)$ (*Manual* Eq. 15-8b) $= (36.0 \text{ kips})(8\tfrac{1}{4} \text{ in.}) - (30.3 \text{ kips})\left(\frac{9.70 \text{ in.}}{2}\right)$ $= 150 \text{ kip-in.}$

For local yielding at the bracket plate:

$$\begin{aligned} F_{cr} &= F_y \\ &= 36 \text{ ksi} \end{aligned} \qquad (\textit{Manual} \text{ Eq. 15-13})$$

For local buckling of the bracket plate:

$$F_{cr} = QF_y \qquad (\textit{Manual} \text{ Eq. 15-14})$$

where

$$\lambda = \frac{\left(\frac{b'}{t}\right)\sqrt{F_y}}{5\sqrt{475 + 1{,}120\left(\frac{b'}{a'}\right)^2}} \qquad (\textit{Manual} \text{ Eq. 15-18})$$

$$= \frac{\left(\frac{9.70 \text{ in.}}{\tfrac{3}{8} \text{ in.}}\right)\sqrt{36 \text{ ksi}}}{5\sqrt{475 + 1{,}120\left(\frac{9.70 \text{ in.}}{21.4 \text{ in.}}\right)^2}}$$

$$= 1.17$$

Since $0.70 < \lambda \le 1.41$:

$$\begin{aligned} Q &= 1.34 - 0.486\lambda \\ &= 1.34 - 0.486(1.17) \\ &= 0.771 \end{aligned} \qquad (\textit{Manual} \text{ Eq. 15-15})$$

$$\begin{aligned} F_{cr} &= QF_y \\ &= 0.771(36 \text{ ksi}) \\ &= 27.8 \text{ ksi} \end{aligned} \qquad (\textit{Manual} \text{ Eq. 15-14})$$

Therefore; local buckling governs over yielding.

The nominal strength of the bracket plate for the limit states of local yielding and local buckling is:

$$\begin{aligned} N_n &= (2 \text{ plates}) F_{cr} t b' \\ &= (2 \text{ plates})(27.8 \text{ ksi})(3/8 \text{ in.})(9.70 \text{ in.}) \\ &= 202 \text{ kips} \end{aligned}$$

(from *Manual* Eq. 15-11)

The nominal flexural strength of the bracket plate for the limit states of local yielding and local buckling is:

$$\begin{aligned} M_n &= (2 \text{ plates}) \frac{F_{cr} t b'^2}{4} \\ &= (2 \text{ plates}) \frac{(27.8 \text{ ksi})(3/8 \text{ in.})(9.70 \text{ in.})^2}{4} \\ &= 490 \text{ kip-in.} \end{aligned}$$

(from *Manual* Eq. 15-12)

LRFD	ASD
$M_r = M_u$ $= 225$ kip-in.	$M_r = M_a$ $= 150$ kip-in.
$\phi = 0.90$	$\Omega = 1.67$
$M_c = \phi M_n$ $= 0.90(490 \text{ kip-in.})$ $= 441 \text{ kip-in.} > 225 \text{ kip-in.}$ **o.k.**	$M_c = \dfrac{M_n}{\Omega}$ $= \dfrac{490 \text{ kip-in.}}{1.67}$ $= 293 \text{ kip-in.} > 150 \text{ kip-in.}$ **o.k.**
$N_r = N_u$ $= 45.5$ kips	$N_r = N_a$ $= 30.3$ kips
$N_c = \phi N_n$ $= 0.90(202 \text{ kips})$ $= 182 \text{ kips} > 45.5 \text{ kips}$ **o.k.**	$N_c = \dfrac{N_n}{\Omega}$ $= \dfrac{202 \text{ kips}}{1.67}$ $= 121 \text{ kips} > 30.3 \text{ kips}$ **o.k.**
$\dfrac{N_r}{N_c} + \dfrac{M_r}{M_c} \leq 1.0$ (*Manual* Eq. 15-10) $\dfrac{45.5 \text{ kips}}{182 \text{ kips}} + \dfrac{225 \text{ kip-in.}}{441 \text{ kip-in.}} = 0.760 < 1.0$ **o.k.**	$\dfrac{N_r}{N_c} + \dfrac{M_r}{M_c} \leq 1.0$ (*Manual* Eq. 15-10) $\dfrac{30.3 \text{ kips}}{121 \text{ kips}} + \dfrac{150 \text{ kip-in.}}{293 \text{ kip-in.}} = 0.762 < 1.0$ **o.k.**

EXAMPLE II.A-24 ECCENTRICALLY LOADED BOLT GROUP (IC METHOD)

Given:

Use AISC *Manual* Table 7-8 to determine the largest eccentric force, acting vertically (0° angle) and at a 15° angle, which can be supported by the available shear strength of the bolts using the instantaneous center of rotation method. Assume that bolt shear controls over bearing and tearout.

Solution A (θ = 0°):

Assume the load is vertical (θ = 0°), as shown in Figure II.A-24-1:

From AISC *Manual* Table 7-1, the available shear strength per bolt for ⅞-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) in single shear is:

LRFD	ASD
$\phi r_n = 24.3$ kips/bolt	$\dfrac{r_n}{\Omega} = 16.2$ kips/bolt

The available strength of the bolt group is determined using AISC *Manual* Table 7-8, with Angle = 0°, a 5½-in. gage with $s = 3$ in., $e_x = 16$ in., and $n = 6$:

$$C = 3.55$$

LRFD	ASD
$\phi R_n = C\phi r_n$ (*Manual* Eq. 7-16) $= 3.55(24.3 \text{ kips/bolt})$ $= 86.3 \text{ kips}$ Thus, P_u must be less than or equal to 86.3 kips.	$\dfrac{R_n}{\Omega} = C\dfrac{r_n}{\Omega}$ (*Manual* Eq. 7-16) $= 3.55(16.2 \text{ kips/bolt})$ $= 57.5 \text{ kips}$ Thus, P_a must be less than or equal to 57.5 kips.

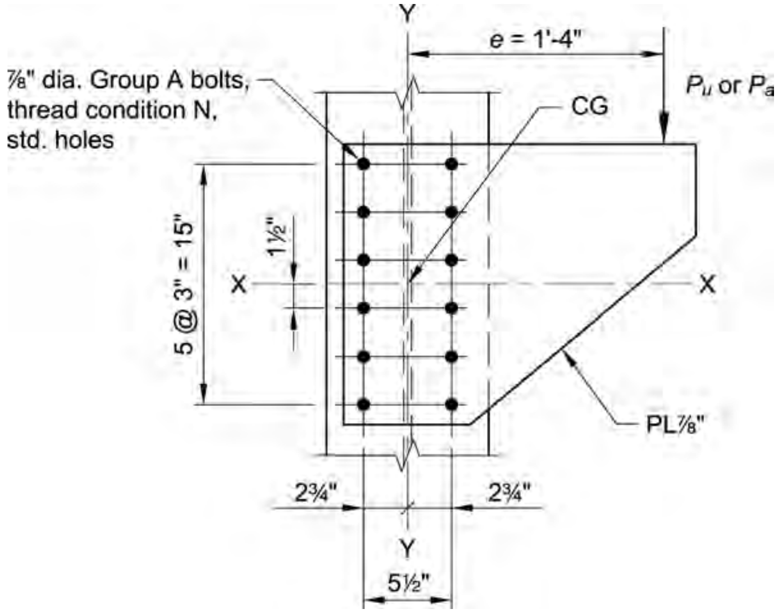

Fig. II.A-24-1. Connection geometry for Example II.A-24—Solution A (θ = 0°).

Note: The eccentricity of the load significantly reduces the shear strength of the bolt group.

Solution B (θ = 15°):

Assume the load acts at an angle of 15° with respect to vertical ($\theta = 15°$), as shown in Figure II.A-24-2:

$$\begin{aligned} e_x &= 16\text{ in.} + (9\text{ in.})(\tan 15°) \\ &= 18.4\text{ in.} \end{aligned}$$

The available strength of the bolt group is determined interpolating from AISC *Manual* Table 7-8, with Angle = 15°, a 5½-in. gage with s = 3 in., e_x = 18.4 in., and n = 6:

$$C = 3.21$$

LRFD	ASD
$\phi R_n = C\phi r_n$ (*Manual* Eq. 7-16) $= 3.21(24.3\text{ kips/bolt})$ $= 78.0\text{ kips}$ Thus, P_u must be less than or equal to 78.0 kips.	$\dfrac{R_n}{\Omega} = C\dfrac{r_n}{\Omega}$ (*Manual* Eq. 7-16) $= 3.21(16.2\text{ kips/bolt})$ $= 52.0\text{ kips}$ Thus, P_a must be less than or equal to 52.0 kips.

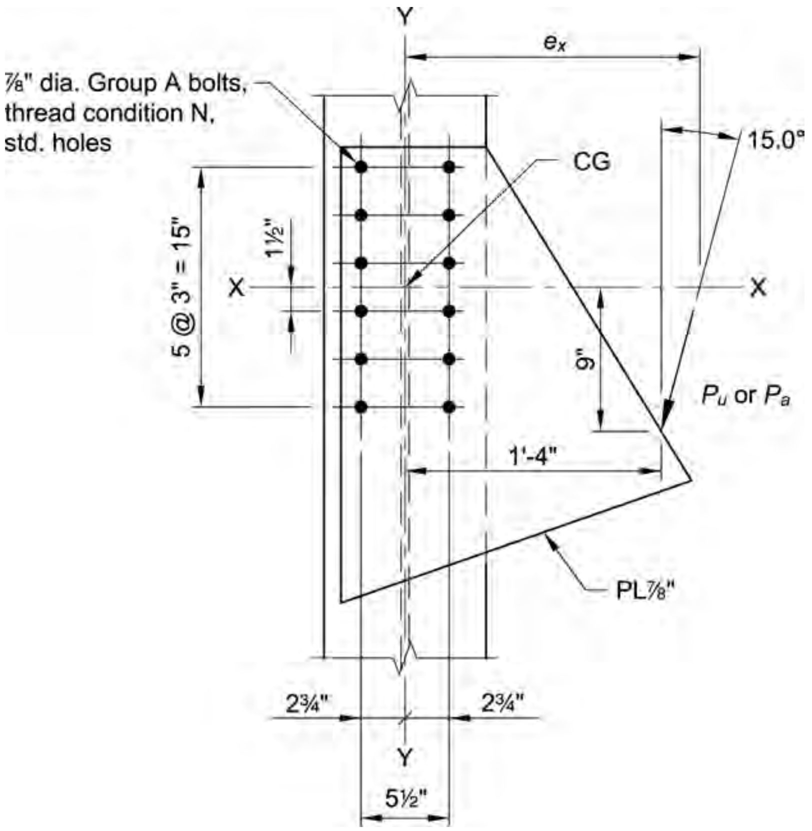

Fig. II.A-24-2. Connection geometry for Example II.A-24—Solution B ($\theta = 15°$).

EXAMPLE II.A-25 ECCENTRICALLY LOADED BOLT GROUP (ELASTIC METHOD)

Given:

Determine the largest eccentric force that can be supported by the available shear strength of the bolts using the elastic method for θ = 0°, as shown in Figure II.A-25-1. Compare the result with that of Example II.A-24. Assume that bolt shear controls over bearing and tearout.

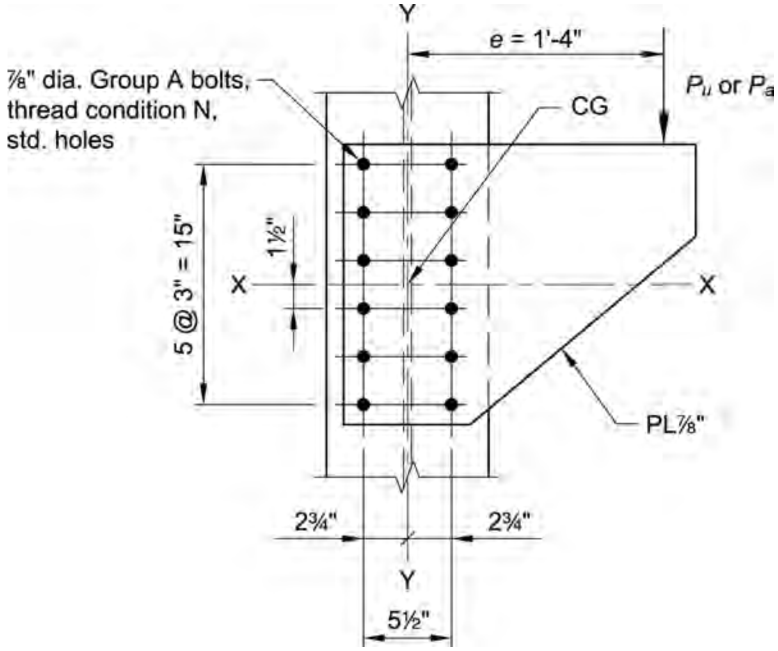

Fig. II.A-25-1. Connection geometry for Example II.A-25.

Solution:

From AISC *Manual* Table 7-1, the available shear strength per bolt for ⅞-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) in single shear is:

LRFD	ASD
$\phi r_n = 24.3$ kips/bolt	$\dfrac{r_n}{\Omega} = 16.2$ kips/bolt

The direct shear force per bolt is determined as follows:

LRFD	ASD
$r_{pxu} = 0$ $r_{pyu} = \dfrac{P_u}{n}$ (from *Manual* Eq. 7-2a) $= \dfrac{P_u}{12}$	$r_{pxa} = 0$ $r_{pya} = \dfrac{P_a}{n}$ (from *Manual* Eq. 7-2b) $= \dfrac{P_a}{12}$

Additional shear force due to eccentricity is determined as follows:

The polar moment of inertia of the bolt group is:

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$$
\begin{aligned}
I_x &\approx \Sigma y^2 \\
&= 4(7.50\text{ in.})^2 + 4(4.50\text{ in.})^2 + 4(1.50\text{ in.})^2 \\
&= 315\text{ in.}^4/\text{in.}^2
\end{aligned}
$$

$$
\begin{aligned}
I_y &\approx \Sigma x^2 \\
&= 12(2.75\text{ in.})^2 \\
&= 90.8\text{ in.}^4/\text{in.}^2
\end{aligned}
$$

$$
\begin{aligned}
I_p &= I_x + I_y \\
&= 315\text{ in.}^4/\text{in.}^2 + 90.8\text{ in.}^4/\text{in.}^2 \\
&= 406\text{ in.}^4/\text{in.}^2
\end{aligned}
$$

LRFD	ASD
$r_{mxu} = \dfrac{P_u e c_y}{I_p}$ (*Manual* Eq. 7-6a) $= \dfrac{P_u(16.0\text{ in.})(7.50\text{ in.})}{406\text{ in.}^4/\text{in.}^2}$ $= 0.296P_u$	$r_{mxa} = \dfrac{P_a e c_y}{I_p}$ (*Manual* Eq. 7-6b) $= \dfrac{P_a(16.0\text{ in.})(7.50\text{ in.})}{406\text{ in.}^4/\text{in.}^2}$ $= 0.296P_a$
$r_{myu} = \dfrac{P_u e c_x}{I_p}$ (*Manual* Eq. 7-7a) $= \dfrac{P_u(16.0\text{ in.})(2.75\text{ in.})}{406\text{ in.}^4/\text{in.}^2}$ $= 0.108P_u$	$r_{mya} = \dfrac{P_a e c_x}{I_p}$ (*Manual* Eq. 7-7b) $= \dfrac{P_a(16.0\text{ in.})(2.75\text{ in.})}{406\text{ in.}^4/\text{in.}^2}$ $= 0.108P_a$
The resultant shear force is determined from AISC *Manual* Equation 7-8a:	The resultant shear force is determined from AISC *Manual* Equation 7-8b:
$r_u = \sqrt{(r_{pxu}+r_{mxu})^2 + (r_{pyu}+r_{myu})^2}$ $= \sqrt{(0+0.296P_u)^2 + \left(\dfrac{P_u}{12}+0.108P_u\right)^2}$ $= 0.352P_u$	$r_a = \sqrt{(r_{pxa}+r_{mxa})^2 + (r_{pya}+r_{mya})^2}$ $= \sqrt{(0+0.296P_a)^2 + \left(\dfrac{P_a}{12}+0.108P_a\right)^2}$ $= 0.352P_a$
Because r_u must be less than or equal to the available strength:	Because r_a must be less than or equal to the available strength:
$P_u \le \dfrac{\phi r_n}{0.352}$ $= \dfrac{24.3\text{ kips/bolt}}{0.352}$ $= 69.0\text{ kips}$	$P_a \le \dfrac{r_n/\Omega}{0.352}$ $= \dfrac{16.2\text{ kips/bolt}}{0.352}$ $= 46.0\text{ kips}$

Note: The elastic method, shown here, is more conservative than the instantaneous center of rotation method, shown in Example II.A-24.

EXAMPLE II.A-26 ECCENTRICALLY LOADED WELD GROUP (IC METHOD)

Given:

Use AISC *Manual* Table 8-8 to determine the largest eccentric force, acting vertically and at a 75° angle, that can be supported by the available shear strength of the weld group, using the instantaneous center of rotation method. Use a ⅜-in. fillet weld and 70-ksi electrodes.

Solution A (θ = 0°):

Assume that the load is vertical (θ = 0°), as shown in Figure II.A-26-1.

$$\begin{aligned} k &= \frac{kl}{l} \\ &= \frac{5 \text{ in.}}{10 \text{ in.}} \\ &= 0.500 \end{aligned}$$

$$\begin{aligned} xl &= \frac{(kl)^2}{2(kl)+l} \\ &= \frac{(5 \text{ in.})^2}{2(5 \text{ in.})+10 \text{ in.}} \\ &= 1.25 \text{ in.} \end{aligned}$$

$$\begin{aligned} &xl + al = 10.0 \text{ in.} \\ &1.25 \text{ in.} + a(10 \text{ in.}) = 10 \text{ in.} \\ &a = 0.875 \end{aligned}$$

$$\begin{aligned} e_x &= al \\ &= 0.875(10 \text{ in.}) \\ &= 8.75 \text{ in.} \end{aligned}$$

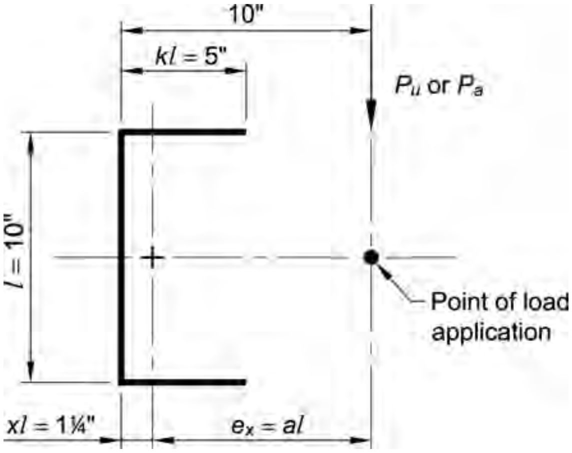

Fig. II.A-26-1. Weld geometry—Solution A (θ = 0°).

The available weld strength is determined using AISC *Manual* Equation 8-21 and interpolating AISC *Manual* Table 8-8, with Angle = 0°, $a = 0.875$, and $k = 0.5$:

$$C = 1.88$$

$$C_1 = 1.00 \text{ (from AISC } \textit{Manual} \text{ Table 8-3)}$$

$$\begin{aligned} R_n &= CC_1Dl \\ &= 1.88(1.00)(6)(10 \text{ in.}) \\ &= 113 \text{ kips} \end{aligned}$$ (*Manual* Eq. 8-21)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(113 \text{ kips}) = 84.8 \text{ kips}$ Thus, P_u must be less than or equal to 84.8 kips.	$\phi = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{113 \text{ kips}}{2.00} = 56.5 \text{ kips}$ Thus, P_a must be less than or equal to 56.5 kips.

Note: The eccentricity of the load significantly reduces the shear strength of this weld group as compared to the concentrically loaded case.

Solution B (θ = 75°):

Assume that the load acts at the same point as in Solution A, but at an angle of 75° with respect to vertical (θ = 75°) as shown in Figure II.A-26-2.

As determined in Solution A:

$$k = 0.500$$
$$a = 0.875$$

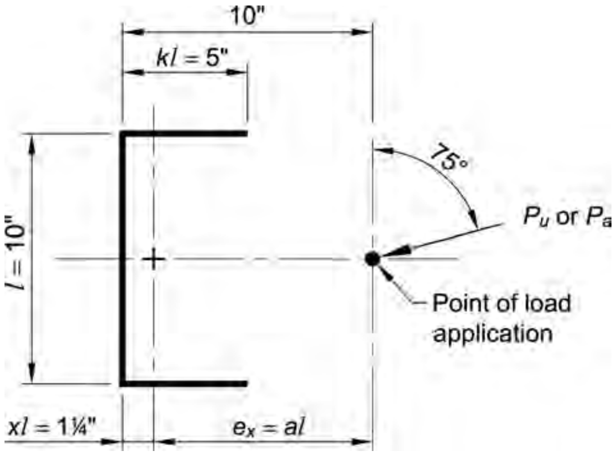

Fig. II.A-26-2. Weld geometry—Solution B (θ = 75°).

The available weld strength is determined using AISC *Manual* Equation 8-21 and interpolating AISC *Manual* Table 8-8, with Angle = 75°, $a = 0.875$, and $k = 0.5$:

$$C = 3.45$$

$C_1 = 1.00$ (from AISC *Manual* Table 8-3)

$$\begin{aligned} R_n &= CC_1Dl \\ &= 3.45(1.00)(6)(10\text{ in.}) \\ &= 207\text{ kips} \end{aligned}$$ (*Manual* Eq. 8-21)

LRFD	ASD
$\phi = 0.75$ $\begin{aligned} \phi R_n &= 0.75(207\text{ kips}) \\ &= 155\text{ kips} \end{aligned}$ Thus, P_u must be less than or equal to 155 kips.	$\phi = 2.00$ $\begin{aligned} \frac{R_n}{\Omega} &= \frac{207\text{ kips}}{2.00} \\ &= 104\text{ kips} \end{aligned}$ Thus, P_a must be less than or equal to 104 kips.

EXAMPLE II.A-27 ECCENTRICALLY LOADED WELD GROUP (ELASTIC METHOD)

Given:

Using the elastic method determine the largest eccentric force that can be supported by the available shear strength of the welds in the connection shown in Figure II.A-27-1. Compare the result with that of Example II.A-26. Use ⅜-in. fillet welds and 70-ksi electrodes.

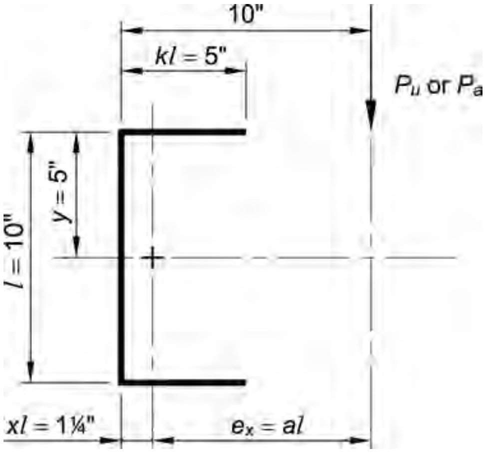

Fig. II.A-27-1. Weld geometry for Example II.A-27.

Solution:

From the weld geometry shown in Figure II.A-27-1 and AISC *Manual* Table 8-8:

$$\begin{aligned} k &= \frac{kl}{l} \\ &= \frac{5 \text{ in.}}{10 \text{ in.}} \\ &= 0.500 \end{aligned}$$

$$\begin{aligned} xl &= \frac{(kl)^2}{2(kl)+l} \\ &= \frac{(5 \text{ in.})^2}{2(5 \text{ in.})+10 \text{ in.}} \\ &= 1.25 \text{ in.} \end{aligned}$$

$$xl + al = 10.0 \text{ in.}$$

$$1.25 \text{ in.} + a(10 \text{ in.}) = 10 \text{ in.}$$

$$a = 0.875$$

$$\begin{aligned} e_x &= al \\ &= 0.875(10 \text{ in.}) \\ &= 8.75 \text{ in.} \end{aligned}$$

Direct Shear Force Per Inch of Weld

LRFD	ASD
$r_{pux} = 0$ $r_{puy} = \dfrac{P_u}{l_{total}}$ (from *Manual* Eq. 8-5a) $= \dfrac{P_u}{20.0 \text{ in.}}$ $= \dfrac{0.0500P_u}{\text{in.}}$	$r_{pax} = 0$ $r_{pay} = \dfrac{P_a}{l_{total}}$ (from *Manual* Eq. 8-5b) $= \dfrac{P_a}{20.0 \text{ in.}}$ $= \dfrac{0.0500P_a}{\text{in.}}$

Additional Shear Force due to Eccentricity

Determine the polar moment of inertia referring to the AISC *Manual* Figure 8-6:

$$
\begin{aligned}
I_x &= \frac{l^3}{12} + 2(kl)(y)^2 \\
&= \frac{(10 \text{ in.})^3}{12} + 2(5 \text{ in.})(5 \text{ in.})^2 \\
&= 333 \text{ in.}^4/\text{in.}
\end{aligned}
$$

$$
\begin{aligned}
I_y &= 2\left[\frac{(kl)^3}{12} + (kl)\left(\frac{kl}{2} - xl\right)^2\right] + l(xl)^2 \\
&= 2\left[\frac{(5 \text{ in.})^3}{12} + (5 \text{ in.})(2.50 \text{ in.} - 1\tfrac{1}{4} \text{ in.})^2\right] + (10 \text{ in.})(1\tfrac{1}{4} \text{ in.})^2 \\
&= 52.1 \text{ in.}^4/\text{in.}
\end{aligned}
$$

$$
\begin{aligned}
I_p &= I_x + I_y \\
&= 333 \text{ in.}^4/\text{in.} + 52.1 \text{ in.}^4/\text{in.} \\
&= 385 \text{ in.}^4/\text{in.}
\end{aligned}
$$

LRFD	ASD
$r_{mux} = \dfrac{P_u e_x c_y}{I_p}$ (from *Manual* Eq. 8-9a) $= \dfrac{P_u(8.75 \text{ in.})(5 \text{ in.})}{385 \text{ in.}^4/\text{in.}}$ $= \dfrac{0.114P_u}{\text{in.}}$	$r_{max} = \dfrac{P_a e_x c_y}{I_p}$ (from *Manual* Eq. 8-9b) $= \dfrac{P_a(8.75 \text{ in.})(5 \text{ in.})}{385 \text{ in.}^4/\text{in.}}$ $= \dfrac{0.114P_a}{\text{in.}}$

LRFD	ASD
$r_{muy} = \frac{P_u e_x c_x}{I_p}$ (from *Manual* Eq. 8-10a) $= \frac{P_u(8.75\text{ in.})(3.75\text{ in.})}{385\text{ in.}^4/\text{in.}}$ $= \frac{0.0852P_u}{\text{in.}}$	$r_{may} = \frac{P_a e_x c_x}{I_p}$ (from *Manual* Eq. 8-10b) $= \frac{P_a(8.75\text{ in.})(3.75\text{ in.})}{385\text{ in.}^4/\text{in.}}$ $= \frac{0.0852P_a}{\text{in.}}$
The resultant shear force is determined using AISC *Manual* Equation 8-11a:	The resultant shear force is determined using *Manual* Equation 8-11b:
$r_u = \sqrt{(r_{pux}+r_{mux})^2+(r_{puy}+r_{muy})^2}$ $= \sqrt{\left(0+\frac{0.114P_u}{\text{in.}}\right)^2+\left(\frac{0.0500P_u}{\text{in.}}+\frac{0.0852P_u}{\text{in.}}\right)^2}$ $= \frac{0.177P_u}{\text{in.}}$	$r_a = \sqrt{(r_{pax}+r_{max})^2+(r_{pay}+r_{may})^2}$ $= \sqrt{\left(0+\frac{0.114P_a}{\text{in.}}\right)^2+\left(\frac{0.0500P_a}{\text{in.}}+\frac{0.0852P_a}{\text{in.}}\right)^2}$ $= \frac{0.177P_a}{\text{in.}}$
Because r_u must be less than or equal to the available strength:	Because r_a must be less than or equal to the available strength:
$r_u = \frac{0.177P_u}{\text{in.}} \le \phi r_n$	$r_a = \frac{0.177P_a}{\text{in.}} \le \frac{r_n}{\Omega}$
Solving for P_u and using AISC *Manual* Equation 8-2a:	Solving for P_a and using AISC *Manual* Equation 8-2b:
$P_u \le \phi r_n\left(\frac{\text{in.}}{0.177}\right)$ $\le (1.392\text{ kip/in.})(6)\left(\frac{\text{in.}}{0.177}\right)$ $\le 47.2\text{ kips}$	$P_a \le \frac{r_n}{\Omega}\left(\frac{\text{in.}}{0.177}\right)$ $\le (0.928\text{ kip/in.})(6)\left(\frac{\text{in.}}{0.177}\right)$ $\le 31.5\text{ kips}$

Note: The strength of the weld group calculated using the elastic method, as shown here, is significantly less than that calculated using the instantaneous center of rotation method in Example II.A-26.

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EXAMPLE II.A-28A ALL-BOLTED SINGLE-ANGLE CONNECTION (BEAM-TO-GIRDER WEB)

Given:

Verify an all-bolted single-angle connection (Case I in AISC *Manual* Table 10-11) between an ASTM A992 W18×35 beam and an ASTM A992 W21×62 girder web, as shown in Figure II.A-28A-1, to support the following beam end reactions:

R_D = 6.5 kips
R_L = 20 kips

The top flange is coped 2 in. deep by 4 in. long, l_{ev} = 1½ in., and l_{eh} = 1¾ in. Use ASTM A36 angle. Use standard angle gages.

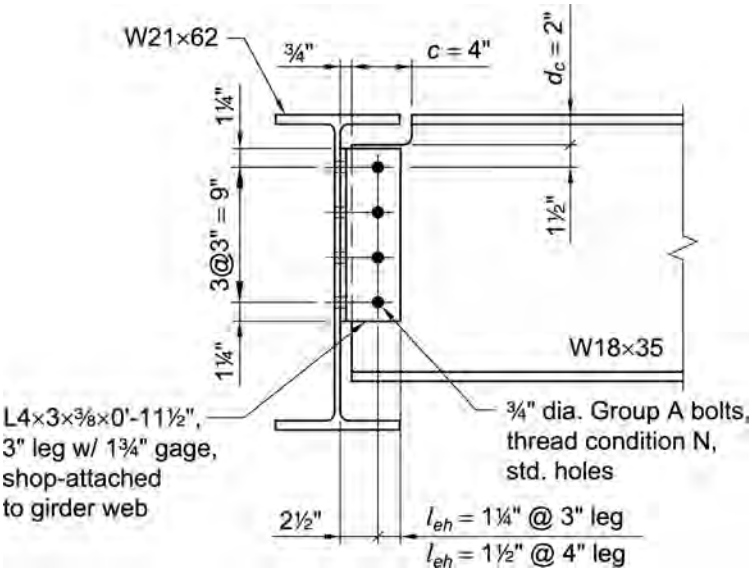

Fig. II.A-28A-1. Connection geometry for Example II.A-28A.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam and girder
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Angle
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-1 the geometric properties are as follows:

Beam
W18×35
t_w = 0.300 in.
d = 17.7 in.

t_f = 0.425 in.

Girder
W21×62
t_w = 0.400 in.

From AISC *Specification* Table J3.3, for ¾-in.-diameter bolts with standard holes:

d_h = 13⁄16 in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(6.5 \text{ kips}) + 1.6(20 \text{ kips})$ $= 39.8 \text{ kips}$	$R_a = 6.5 \text{ kips} + 20 \text{ kips}$ $= 26.5 \text{ kips}$

Strength of the Bolted Connection—Angle

Check eccentricity of connection.

For the 4-in. angle leg attached to the supported beam (W18×35):

e = 2½ in. < 3.00 in., therefore, eccentricity does not need to be considered for this leg. (See AISC *Manual* Figure 10-14)

For the 3-in. angle leg attached to the supporting girder (W21×62):

$$\begin{aligned} e &= 1\tfrac{3}{4} \text{ in.} + \frac{0.300 \text{ in.}}{2} \\ &= 1.90 \text{ in.} \end{aligned}$$

Because e = 1.90 in. < 2½ in., AISC *Manual* Table 10-11 may be conservatively used for bolt shear. From Table 10-11, Case I, with n = 4:

$C = 3.07$

From the Commentary to AISC *Specification* Section J3.6, the strength of the bolt group is taken as the sum of the invividual strengths of the individual fasteners, taken as the lesser of the fastener shear strength per AISC *Specification* Section J3.6, the bearing strength at the bolt hole per AISC *Specification* Section J3.10, or the tearout strength at the bolt hole per AISC *Specification* Section J3.10. In this case, the 3-in. angle leg attached to the supporting girder will control because eccentricity must be taken into consideration and the available strength will be determined based on the bolt group using the eccentrically loaded bolt coefficient, C.

From AISC *Manual* Table 7-1, the available shear strength per bolt for ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) is:

LRFD	ASD
$\phi r_n = 17.9 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 11.9 \text{ kips/bolt}$

The available bearing and tearout strength of the angle at the bottom edge bolt is determined using AISC *Manual* Table 7-5, with l_e = 1¼ in., as follows:

LRFD	ASD
$\phi r_n = (44.0\text{ kip/in.})(\tfrac{3}{8}\text{ in.})$ $= 16.5\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (29.4\text{ kip/in.})(\tfrac{3}{8}\text{ in.})$ $= 11.0\text{ kips/bolt}$

The available bearing and tearout strength of the angle at the interior bolts (not adjacent to the edge) is determined using AISC *Manual* Table 7-4 with $s = 3$ in.

LRFD	ASD
$\phi r_n = (78.3\text{ kip/in.})(\tfrac{3}{8}\text{ in.})$ $= 29.4\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (52.2\text{ kip/in.})(\tfrac{3}{8}\text{ in.})$ $= 19.6\text{ kips/bolt}$

The available strength of the bolted connection at the angle is conservatively determined using the minimum available strength calculated for bolt shear, bearing on the angle, and tearout on the angle. The bolt group eccentricity is accounted for by multiplying the minimum available strength by the bolt coefficient C.

LRFD	ASD
$\phi R_n = C\phi r_n$ $= 3.07(16.5\text{ kips/bolt})$ $= 50.7\text{ kips} > 39.8\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = C\dfrac{r_n}{\Omega}$ $= 3.07(11.0\text{ kips/bolt})$ $= 33.8\text{ kips} > 26.5\text{ kips}$ **o.k.**

Strength of the Bolted Connection—W18×35 Beam Web

The available bearing and tearout strength of the beam web at the top edge bolt is determined using AISC *Manual* Table 7-5, conservatively using $l_e = 1\tfrac{1}{4}$ in., as follows:

LRFD	ASD
$\phi r_n = (49.4\text{ kip/in.})(0.300\text{ in.})$ $= 14.8\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (32.9\text{ kip/in.})(0.300\text{ in.})$ $= 9.87\text{ kips/bolt}$

The available bearing and tearout strength of the beam web at the interior bolts (not adjacent to the edge) is determined using AISC *Manual* Table 7-4 with $s = 3$ in.

LRFD	ASD
$\phi r_n = (87.8\text{ kip/in.})(0.300\text{ in.})$ $= 26.3\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (58.5\text{ kip/in.})(0.300\text{ in.})$ $= 17.6\text{ kips/bolt}$

The available strength of the bolted connection at the beam web is determined by summing the effective strength for each bolt using the minimum available strength calculated for bolt shear, bearing on the web, and tearout on the web.

LRFD	ASD
$\phi R_n = n\phi r_n$ $= (1\text{ bolt})(14.8\text{ kips/bolt})$ $+ (3\text{ bolts})(17.9\text{ kips/bolt})$ $= 68.5\text{ kips} > 39.8\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = n\dfrac{r_n}{\Omega}$ $= (1\text{ bolt})(9.87\text{ kips/bolt})$ $+ (3\text{ bolts})(11.9\text{ kips/bolt})$ $= 45.6\text{ kips} > 26.5\text{ kips}$ **o.k.**

Strength of the Bolted Connection—W21×62 Girder Web

The available bearing and tearout strength of the girder web is determined using AISC *Manual* Table 7-4 with s = 3 in.

LRFD	ASD
$\phi r_n = (87.8\text{ kip/in.})(0.400\text{ in.})$ $= 35.1\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (58.5\text{ kip/in.})(0.400\text{ in.})$ $= 23.4\text{ kips/bolt}$

Therefore; bolt shear controls over bearing or tearout on the girder web and is adequate based on previous calculations.

Shear Strength of Angle

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the angle is determined as follows:

$$
\begin{aligned}
A_{gv} &= lt \\
&= (11\tfrac{1}{2}\text{ in.})(\tfrac{3}{8}\text{ in.}) \\
&= 4.31\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
R_n &= 0.60F_y A_{gv} \\
&= 0.60(36\text{ ksi})(4.31\text{ in.}^2) \\
&= 93.1\text{ kips}
\end{aligned}
$$

(*Spec*. Eq. J4-3)

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(93.1\text{ kips})$ $= 93.1\text{ kips} > 39.8\text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{93.1\text{ kips}}{1.50}$ $= 62.1\text{ kips} > 26.5\text{ kips}$ **o.k.**

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the angle is determined using the net area determined in accordance with AISC *Specification* Section B4.3b.

$$
\begin{aligned}
A_{nv} &= \left[l - n\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t \\
&= \left[11\tfrac{1}{2}\text{ in.} - 4\left(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right](\tfrac{3}{8}\text{ in.}) \\
&= 3.00\text{ in.}^2
\end{aligned}
$$

$$\begin{aligned} R_n &= 0.60F_uA_{nv} \\ &= 0.60\left(58\text{ ksi}\right)\left(3.00\text{ in.}^2\right) \\ &= 104\text{ kips} \end{aligned}$$

(*Spec*. Eq. J4-4)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(104\text{ kips}\right)$ $= 78.0\text{ kips} > 39.8\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{104\text{ kips}}{2.00}$ $= 52.0\text{ kips} > 26.5\text{ kips}$ **o.k.**

Block Shear Rupture of Angle

The nominal strength for the limit state of block shear rupture is given by AISC *Specification* Section J4.3.

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.60F_yA_{gv} + U_{bs}F_uA_{nt}$$

(*Spec*. Eq. J4-5)

The available block shear rupture strength of the 3-in. leg is determined as follows, using AISC *Manual* Tables 9-3a, 9-3b and 9-3c, and AISC *Specification* Equation J4-5, with $n = 4$, $l_{ev} = l_{eh} = 1\tfrac{1}{4}$ in., and $U_{bs} = 1.0$.

LRFD	ASD
Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{\phi F_uA_{nt}}{t} = 35.3\text{ kip/in.}$	Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{F_uA_{nt}}{\Omega t} = 23.6\text{ kip/in.}$
Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{\phi 0.6F_yA_{gv}}{t} = 166\text{ kip/in.}$	Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{0.6F_yA_{gv}}{\Omega t} = 111\text{ kip/in.}$
Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{\phi 0.6F_uA_{nv}}{t} = 188\text{ kip/in.}$	Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{0.6F_uA_{nv}}{\Omega t} = 125\text{ kip/in.}$
$\phi R_n = \phi 0.60F_uA_{nv} + \phi U_{bs}F_uA_{nt}$ $\leq \phi 0.60F_yA_{gv} + \phi U_{bs}F_uA_{nt}$ $= \left(\tfrac{3}{8}\text{ in.}\right)\left[188\text{ kip/in.} + \left(1.0\right)\left(35.3\text{ kip/in.}\right)\right]$ $\leq \left(\tfrac{3}{8}\text{ in.}\right)\left[166\text{ kip/in.} + \left(1.0\right)\left(35.3\text{ kip/in.}\right)\right]$ $= 83.7\text{ kips} > 75.5\text{ kips}$	$\dfrac{R_n}{\Omega} = \dfrac{0.60F_uA_{nv}}{\Omega} + \dfrac{U_{bs}F_uA_{nt}}{\Omega}$ $\leq \dfrac{0.60F_yA_{gv}}{\Omega} + \dfrac{U_{bs}F_uA_{nt}}{\Omega}$ $= \left(\tfrac{3}{8}\text{ in.}\right)\left[125\text{ kip/in.} + \left(1.0\right)\left(23.6\text{ kip/in.}\right)\right]$ $\leq \left(\tfrac{3}{8}\text{ in.}\right)\left[111\text{ kip/in.} + \left(1.0\right)\left(23.6\text{ kip/in.}\right)\right]$ $= 55.7\text{ kips} > 50.5\text{ kips}$

LRFD	ASD
Therefore: $\phi R_n = 75.5 \text{ kips} > 39.8 \text{ kips}$ **o.k.**	Therefore: $\dfrac{R_n}{\Omega} = 50.5 \text{ kips} > 26.5 \text{ kips}$ **o.k.**

Because the edge distance is smaller, block shear rupture is governed by the 3-in. leg.

Flexural Yielding Strength of Angle

The required flexural strength of the support leg of the angle is determined as follows:

LRFD	ASD
$M_u = R_u e$ $= (39.8 \text{ kips})\left(1\tfrac{3}{4} \text{ in.} + \dfrac{0.300 \text{ in.}}{2}\right)$ $= 75.6 \text{ kip-in.}$	$M_a = R_a e$ $= (26.5 \text{ kips})\left(1\tfrac{3}{4} \text{ in.} + \dfrac{0.300 \text{ in.}}{2}\right)$ $= 50.4 \text{ kip-in.}$

The available flexural yielding strength of the support leg of the angle is determined as follows:

$$
\begin{aligned}
M_n &= F_y Z_x \\
&= (36 \text{ ksi})\left[\frac{(\tfrac{3}{8} \text{ in.})(11\tfrac{1}{2} \text{ in.})^2}{4}\right] \\
&= 446 \text{ kip-in.}
\end{aligned}
$$

LRFD	ASD
$\phi = 0.90$ $\phi M_n = 0.90(446 \text{ kip-in.})$ $= 401 \text{ kip-in.} > 75.6 \text{ kip-in.}$ **o.k.**	$\Omega = 1.67$ $\dfrac{M_n}{\Omega} = \dfrac{446 \text{ kip-in.}}{1.67}$ $= 267 \text{ kip-in.} > 50.4 \text{ kip-in.}$ **o.k.**

Flexural Rupture Strength of Angle

The available flexural rupture strength of the support leg of the angle is determined as follows:

$$
\begin{aligned}
Z_{net} &= (\tfrac{3}{8} \text{ in.})\left[\frac{(11\tfrac{1}{2} \text{ in.})^2}{4} - 2(\tfrac{13}{16} \text{ in.} + \tfrac{1}{16} \text{ in.})(4.50 \text{ in.}) - 2(\tfrac{13}{16} \text{ in.} + \tfrac{1}{16} \text{ in.})(1.50 \text{ in.})\right] \\
&= 8.46 \text{ in.}^3
\end{aligned}
$$

$$
\begin{aligned}
M_n &= F_u Z_{net} \qquad \text{(\textit{Manual} Eq. 9-4)} \\
&= (58 \text{ ksi})(8.46 \text{ in.}^3) \\
&= 491 \text{ kip-in.}
\end{aligned}
$$

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LRFD	ASD
$\phi_b = 0.75$ $\phi_b M_n = 0.75(491 \text{ kip-in.})$ $= 368 \text{ kip-in.} > 75.6 \text{ kip-in.}$ **o.k.**	$\Omega_b = 2.00$ $\dfrac{M_n}{\Omega_b} = \dfrac{491 \text{ kip-in.}}{2.00}$ $= 246 \text{ kip-in.} > 50.4 \text{ kip-in.}$ **o.k.**

Flexural Yielding and Buckling of Coped Beam Web

The required flexural strength of the coped section of the beam web is determined using AISC *Manual* Equation 9-5a or 9-5b, as follows:

$$\begin{aligned} e &= c + setback \\ &= 4 \text{ in.} + \tfrac{3}{4} \text{ in.} \\ &= 4.75 \text{ in.} \end{aligned}$$

LRFD	ASD
$M_u = R_u e$ $= (39.8 \text{ kips})(4.75 \text{ in.})$ $= 189 \text{ kip-in.}$	$M_a = R_a e$ $= (26.5 \text{ kips})(4.75 \text{ in.})$ $= 126 \text{ kip-in.}$

The minimum length of the connection elements is one-half of the reduced beam depth, h_o:

$$\begin{aligned} h_o &= d - d_c \quad \text{(from AISC } \textit{Manual} \text{ Figure 9-2)} \\ &= 17.7 \text{ in.} - 2 \text{ in.} \\ &= 15.7 \text{ in.} \end{aligned}$$

$$\begin{aligned} l &> 0.5h_o \\ 11\tfrac{1}{2} \text{ in.} &> 0.5(15.7 \text{ in.}) \\ 11\tfrac{1}{2} \text{ in.} &> 7.85 \text{ in.} \quad \textbf{o.k.} \end{aligned}$$

The available flexural local buckling strength of a beam coped at the top flange is determined as follows:

$$\begin{aligned} \lambda &= \frac{h_o}{t_w} \\ &= \frac{15.7 \text{ in.}}{0.300 \text{ in.}} \\ &= 52.3 \end{aligned} \qquad (\textit{Manual} \text{ Eq. 9-11})$$

$$\begin{aligned} \frac{c}{h_o} &= \frac{4 \text{ in.}}{15.7 \text{ in.}} \\ &= 0.255 \end{aligned}$$

Because $\dfrac{c}{h_o} \le 1.0,$ the plate buckling coefficient, k, is calculated as follows:

$$k = 2.2\left(\frac{h_o}{c}\right)^{1.65} \qquad \text{(\textit{Manual} Eq. 9-13a)}$$

$$= 2.2\left(\frac{15.7\text{ in.}}{4\text{ in.}}\right)^{1.65}$$

$$= 21.0$$

$$\frac{c}{d} = \frac{4\text{ in.}}{17.7\text{ in.}}$$

$$= 0.226$$

Because $\dfrac{c}{d} \le 1.0,$ the buckling adjustment factor, *f*, is calculated as follows:

$$f = 2\left(\frac{c}{d}\right) \qquad \text{(\textit{Manual} Eq. 9-14a)}$$

$$= 2(0.226)$$

$$= 0.452$$

$$k_1 = fk \ge 1.61 \qquad \text{(\textit{Manual} Eq. 9-10)}$$

$$= (0.452)(21.0) \ge 1.61$$

$$= 9.49 > 1.61$$

$$= 9.49$$

$$\lambda_p = 0.475\sqrt{\frac{k_1 E}{F_y}} \qquad \text{(\textit{Manual} Eq. 9-12)}$$

$$= 0.475\sqrt{\frac{(9.49)(29{,}000\text{ ksi})}{50\text{ ksi}}}$$

$$= 35.2$$

$$2\lambda_p = 2(35.2)$$

$$= 70.4$$

Because $\lambda_p < \lambda \le 2\lambda_p$, calculate the nominal flexural strength using AISC *Manual* Equation 9-7.

The plastic section modulus of the coped section, Z_{net}, is determined from Table IV-11 (included in Part IV of this document).

$$Z_{net} = 32.1\text{ in.}^3$$

$$M_p = F_y Z_{net}$$

$$= (50\text{ ksi})(32.1\text{ in.}^3)$$

$$= 1{,}610\text{ kip-in.}$$

From AISC *Manual* Table 9-2:

$$S_{net} = 18.2\text{ in.}^3$$

$$\begin{aligned} M_y &= F_y S_{net} \\ &= (50 \text{ ksi})(18.2 \text{ in.}^3) \\ &= 910 \text{ kip-in.} \end{aligned}$$

$$M_n = M_p - (M_p - M_y)\left(\frac{\lambda}{\lambda_p} - 1\right) \qquad \text{(}\textit{Manual}\text{ Eq. 9-7)}$$

$$\begin{aligned} &= (1{,}610 \text{ kip-in.}) - (1{,}610 \text{ kip-in.} - 910 \text{ kip-in.})\left(\frac{52.3}{35.2} - 1\right) \\ &= 1{,}270 \text{ kip-in.} \end{aligned}$$

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(1{,}270 \text{ kip-in.})$ $= 1{,}140 \text{ kip-in.} > 189 \text{ kip-in.}$ **o.k.**	$\Omega_b = 1.67$ $\frac{M_n}{\Omega_b} = \frac{1{,}270 \text{ kip-in.}}{1.67}$ $= 760 \text{ kip-in.} > 126 \text{ kip-in.}$ **o.k.**

Shear Strength of Beam Web

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the beam web is determined as follows:

$$\begin{aligned} A_{gv} &= h_o t_w \\ &= (15.7 \text{ in.})(0.300 \text{ in.}) \\ &= 4.71 \text{ in.}^2 \end{aligned}$$

$$R_n = 0.60 F_y A_{gv} \qquad \text{(}\textit{Spec}\text{. Eq. J4-3)}$$

$$\begin{aligned} &= 0.60(50 \text{ ksi})(4.71 \text{ in.}^2) \\ &= 141 \text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(141 \text{ kips})$ $= 141 \text{ kips} > 39.8 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{141 \text{ kips}}{1.50}$ $= 94.0 \text{ kips} > 26.5 \text{ kips}$ **o.k.**

Block Shear Rupture of Beam Web

The nominal strength for the limit state of block shear rupture is given by AISC *Specification* Section J4.3.

$$R_n = 0.60 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.60 F_y A_{gv} + U_{bs} F_u A_{nt} \qquad \text{(}\textit{Spec}\text{. Eq. J4-5)}$$

The available block shear rupture strength of the web is determined as follows, using AISC *Manual* Tables 9-3a, 9-3b and 9-3c, and AISC *Specification* Equation J4-5, with $n = 4$, $l_{ev} = 1\frac{1}{2}$ in., $l_{eh} = 1\frac{1}{2}$ in. (including a ¼-in. tolerance to account for possible beam underrun), and $U_{bs} = 1.0$.

LRFD	ASD
Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{\phi F_u A_{nt}}{t} = 51.8 \text{ kip/in.}$	Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{F_u A_{nt}}{\Omega t} = 34.5 \text{ kip/in.}$
Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{\phi 0.60 F_y A_{gv}}{t} = 236 \text{ kip/in.}$	Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{0.60 F_y A_{gv}}{\Omega t} = 158 \text{ kip/in.}$
Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{\phi 0.60 F_u A_{nv}}{t} = 218 \text{ kip/in.}$	Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{0.60 F_u A_{nv}}{\Omega t} = 145 \text{ kip/in.}$
$\begin{aligned} \phi R_n &= \phi 0.60 F_u A_{nv} + \phi U_{bs} F_u A_{nt} \\ &\leq \phi 0.60 F_y A_{gv} + \phi U_{bs} F_u A_{nt} \\ &= (0.300 \text{ in.})\left[218 \text{ kip/in.} + (1.0)(51.8 \text{ kip/in.})\right] \\ &\leq (0.300 \text{ in.})\left[236 \text{ kip/in.} + (1.0)(51.8 \text{ kip/in.})\right] \\ &= 80.9 \text{ kips} < 86.3 \text{ kips} \end{aligned}$	$\begin{aligned} \frac{R_n}{\Omega} &= \frac{0.60 F_u A_{nv}}{\Omega} + \frac{U_{bs} F_u A_{nt}}{\Omega} \\ &\leq \frac{0.60 F_y A_{gv}}{\Omega} + \frac{U_{bs} F_u A_{nt}}{\Omega} \\ &= (0.300 \text{ in.})\left[145 \text{ kip/in.} + (1.0)(34.5 \text{ kip/in.})\right] \\ &\leq (0.300 \text{ in.})\left[158 \text{ kip/in.} + (1.0)(34.5 \text{ kip/in.})\right] \\ &= 53.9 \text{ kips} < 57.8 \text{ kips} \end{aligned}$
Therefore: $\phi R_n = 80.9 \text{ kips} > 39.8 \text{ kips}$ **o.k.**	Therefore: $\dfrac{R_n}{\Omega} = 53.9 \text{ kips} > 26.5 \text{ kips}$ **o.k.**

Conclusion

The connection is found to be adequate as given for the applied load.

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EXAMPLE II.A-28B ALL-BOLTED SINGLE ANGLE CONNECTION—STRUCTURAL INTEGRITY CHECK

Given:

Verify the all-bolted single-angle connection from Example II.A-28A, as shown in Figure II.A-28B-1, for the structural integrity provisions of AISC *Specification* Section B3.9. The connection is verified as a beam end connection. Note that these checks are necessary when design for structural integrity is required by the applicable building code. The angle is ASTM A36 material.

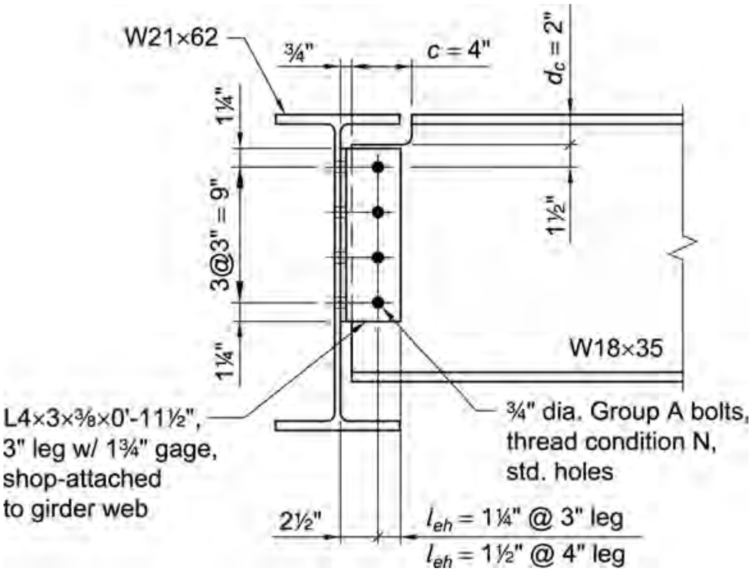

Fig. II.A-28B-1. Connection geometry for Example II.A-28B.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam and Girder
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Angle
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W18x35
$t_w = 0.300$ in.

Girder
W21x62
t_w = 0.400 in.
d = 21.0 in.
k_{des} = 1.12 in.

From AISC *Specification* Table J3.3, the hole diameter for ¾-in.-diameter bolts with standard holes is:

d_h = 13⁄16 in.

From Example II.A-28A, the required shear strength is:

LRFD	ASD
$V_u = 39.8$ kips	$V_a = 26.5$ kips

From AISC *Specification* Section B3.9(b), the required axial tensile strength is:

LRFD	ASD
$T_u = \frac{2}{3}V_u \geq 10$ kips $= \frac{2}{3}(39.8\text{ kips}) > 10$ kips $= 26.5\text{ kips} > 10$ kips $= 26.5$ kips	$T_a = V_a \geq 10$ kips $= 26.5\text{ kips} > 10$ kips $= 26.5$ kips

Bolt Shear

From AISC *Specification* Section J3.6, the nominal bolt shear strength is determined as follows:

F_{nv} = 54 ksi, from AISC *Specification* Table J3.2

$$\begin{aligned} T_n &= nF_{nv}A_b \\ &= (4\text{ bolts})(54\text{ ksi})(0.442\text{ in.}^2) \\ &= 95.5\text{ kips} \end{aligned}$$ (from *Spec*. Eq. J3-1)

Bolt Tension

From AISC *Specification* Section J3.6, the nominal bolt tensile strength is determined as follows:

F_{nt} = 90 ksi, from AISC *Specification* Table J3.2

$$\begin{aligned} T_n &= nF_{nt}A_b \\ &= (4\text{ bolts})(90\text{ ksi})(0.442\text{ in.}^2) \\ &= 159\text{ kips} \end{aligned}$$ (from *Spec*. Eq. J3-1)

Bolt Bearing and Tearout

From AISC *Specification* Section B3.9, for the purpose of satisfying structural integrity requirements inelastic deformations of the connection are permitted; therefore, AISC *Specification* Equations J3-6b and J3-6d are used to determine the nominal bearing and tearout strength.

For bolt bearing on the angle:

$$\begin{aligned} T_n &= (4\text{ bolts})3.0dtF_u \\ &= (4\text{ bolts})(3.0)(\tfrac{3}{4}\text{ in.})(\tfrac{3}{8}\text{ in.})(58\text{ ksi}) \\ &= 196\text{ kips} \end{aligned}$$

(from *Spec.* Eq. J3-6b)

For bolt bearing on the beam web:

$$\begin{aligned} T_n &= (4\text{ bolts})3.0dt_wF_u \\ &= (4\text{ bolts})(3.0)(\tfrac{3}{4}\text{ in.})(0.300\text{ in.})(65\text{ ksi}) \\ &= 176\text{ kips} \end{aligned}$$

(from *Spec.* Eq. J3-6b)

For bolt tearout on the angle:

$$\begin{aligned} l_c &= l_{eh} - 0.5d_h \\ &= 1\tfrac{1}{2}\text{ in.} - 0.5(\tfrac{13}{16}\text{ in.}) \\ &= 1.09\text{ in.} \end{aligned}$$

$$\begin{aligned} T_n &= (4\text{ bolts})1.5l_ctF_u \\ &= (4\text{ bolts})(1.5)(1.09\text{ in.})(\tfrac{3}{8}\text{ in.})(58\text{ ksi}) \\ &= 142\text{ kips} \end{aligned}$$

(from *Spec.* Eq. J3-6d)

For bolt tearout on the beam web (including a ¼-in. tolerance to account for possible beam underrun):

$$\begin{aligned} l_c &= l_{eh} - 0.5d_h \\ &= (1\tfrac{3}{4}\text{ in.} - \tfrac{1}{4}\text{ in.}) - 0.5(\tfrac{13}{16}\text{ in.}) \\ &= 1.09\text{ in.} \end{aligned}$$

$$\begin{aligned} T_n &= (4\text{ bolts})1.5l_ct_wF_u \\ &= (4\text{ bolts})(1.5)(1.09\text{ in.})(0.300\text{ in.})(65\text{ ksi}) \\ &= 128\text{ kips} \end{aligned}$$

(from *Spec.* Eq. J3-6d)

Angle Bending and Prying Action

From AISC *Manual* Part 9, the nominal strength of the angle accounting for prying action is determined as follows:

$$\begin{aligned} b &= gage - \frac{t}{2} \\ &= 1\tfrac{3}{4}\text{ in.} - \frac{\tfrac{3}{8}\text{ in.}}{2} \\ &= 1.56\text{ in.} \end{aligned}$$

$$\begin{aligned} a &= \min\left\{1\tfrac{1}{4}\text{ in.}, 1.25b\right\} \\ &= \min\left\{1\tfrac{1}{4}\text{ in.}, 1.25(1.56\text{ in.})\right\} \\ &= 1.25\text{ in.} \end{aligned}$$

$$\begin{aligned} b' &= b - \frac{d_b}{2} \\ &= 1.56\text{ in.} - \frac{\tfrac{3}{4}\text{ in.}}{2} \\ &= 1.19\text{ in.} \end{aligned}$$

(*Manual* Eq. 9-18)

$$\begin{aligned} a' &= \left(a + \frac{d_b}{2}\right) \le \left(1.25b + \frac{d_b}{2}\right) \\ &= 1.25 + \frac{\tfrac{3}{4}\text{ in.}}{2} \le 1.25(1.56\text{ in.}) + \frac{\tfrac{3}{4}\text{ in.}}{2} \\ &= 1.63\text{ in.} < 2.33\text{ in.} \\ &= 1.63\text{ in.} \end{aligned}$$

(*Manual* Eq. 9-23)

$$\begin{aligned} \rho &= \frac{b'}{a'} \\ &= \frac{1.19\text{ in.}}{1.63\text{ in.}} \\ &= 0.730 \end{aligned}$$

(*Manual* Eq. 9-22)

Note that end distances of 1¼ in. are used on the angles, so *p* is the average pitch of the bolts:

$$\begin{aligned} p &= \frac{l}{n} \\ &= \frac{11\tfrac{1}{2}\text{ in.}}{4} \\ &= 2.88\text{ in.} \end{aligned}$$

Check:

$p < s = 3.00\text{ in.}$ **o.k.**

$$\begin{aligned} d' &= d_h \\ &= \tfrac{13}{16}\text{ in.} \end{aligned}$$

$$\begin{aligned} \delta &= 1 - \frac{d'}{p} \\ &= 1 - \frac{\tfrac{13}{16}\text{ in.}}{2.88\text{ in.}} \\ &= 0.718 \end{aligned}$$

(*Manual* Eq. 9-20)

$$\begin{aligned} B_n &= F_{nt}A_b \\ &= (90\text{ ksi})\left(0.442\text{ in.}^2\right) \\ &= 39.8\text{ kips/bolt} \end{aligned}$$

$$t_c = \sqrt{\frac{4B_n b'}{pF_u}} \qquad \text{(from } \textit{Manual} \text{ Eq. 9-26)}$$

$$= \sqrt{\frac{4\left(39.8 \text{ kips/bolt}\right)\left(1.19 \text{ in.}\right)}{\left(2.88 \text{ in.}\right)\left(58 \text{ ksi}\right)}}$$

$$= 1.06 \text{ in.}$$

$$\alpha' = \frac{1}{\delta\left(1+\rho\right)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] \qquad \text{(} \textit{Manual} \text{ Eq. 9-28)}$$

$$= \frac{1}{0.718\left(1+0.730\right)}\left[\left(\frac{1.06 \text{ in.}}{3/8 \text{ in.}}\right)^2 - 1\right]$$

$$= 5.63$$

Because $\alpha' > 1$:

$$Q = \left(\frac{t}{t_c}\right)^2\left(1+\delta\right)$$

$$= \left(\frac{3/8 \text{ in.}}{1.06 \text{ in.}}\right)^2\left(1+0.718\right)$$

$$= 0.215$$

$$T_n = \left(4 \text{ bolts}\right)B_n Q \qquad \text{(from } \textit{Manual} \text{ Eq. 9-27)}$$

$$= \left(4 \text{ bolts}\right)\left(39.8 \text{ kips/bolt}\right)\left(0.215\right)$$

$$= 34.2 \text{ kips}$$

Block Shear Rupture—Angle

From AISC *Specification* Section J4.3, the nominal block shear rupture strength of the angle with a "U" shaped failure plane is determined as follows:

$$T_n = 0.60F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.60F_y A_{gv} + U_{bs} F_u A_{nt} \qquad \text{(from } \textit{Spec.} \text{ Eq. J4-5)}$$

where

$$A_{gv} = 2l_{eh}t$$

$$= \left(2\right)\left(1\tfrac{1}{2} \text{ in.}\right)\left(\tfrac{3}{8} \text{ in.}\right)$$

$$= 1.13 \text{ in.}^2$$

$$A_{nv} = \left(2\right)\left[l_{eh} - 0.5\left(d_h + \tfrac{1}{16} \text{ in.}\right)\right]t$$

$$= \left(2\right)\left[1\tfrac{1}{2} \text{ in.} - 0.5\left(\tfrac{13}{16} \text{ in.} + \tfrac{1}{16} \text{ in.}\right)\right]\left(\tfrac{3}{8} \text{ in.}\right)$$

$$= 0.797 \text{ in.}^2$$

$$A_{nt} = \left[9.00 \text{ in.} - 4\left(d_h + \tfrac{1}{16} \text{ in.}\right)\right]t$$

$$= \left[9.00 \text{ in.} - 4\left(\tfrac{13}{16} \text{ in.} + \tfrac{1}{16} \text{ in.}\right)\right]\left(\tfrac{3}{8} \text{ in.}\right)$$

$$= 2.06 \text{ in.}^2$$

$$U_{bs} = 1.0$$

$$\begin{aligned} T_n &= 0.60(58\text{ ksi})(0.797\text{ in.}^2) + 1.0(58\text{ ksi})(2.06\text{ in.}^2) \leq 0.60(36\text{ ksi})(1.13\text{ in.}^2) + 1.0(58\text{ ksi})(2.06\text{ in.}^2) \\ &= 147\text{ kips} > 144\text{ kips} \end{aligned}$$

Therefore:

$$T_n = 144\text{ kips}$$

Tensile Yielding of Angle

From AISC *Specification* Section J4.1, the nominal tensile yielding strength of the angle is determined as follows:

$$\begin{aligned} A_g &= lt \\ &= (11\tfrac{1}{2}\text{ in.})(\tfrac{3}{8}\text{ in.}) \\ &= 4.31\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} T_n &= F_y A_g && \text{(from } \textit{Spec.}\text{ Eq. J4-1)} \\ &= (36\text{ ksi})(4.31\text{ in.}^2) \\ &= 155\text{ kips} \end{aligned}$$

Tensile Rupture of Angle

From AISC *Specification* Section J4.1, the nominal tensile rupture strength of the angle is determined as follows:

$$\begin{aligned} A_e &= A_n U && (\textit{Spec.}\text{ Eq. D3-1)} \\ &= \left[l - n\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]tU \\ &= \left[11\tfrac{1}{2}\text{ in.} - 4\left(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right](\tfrac{3}{8}\text{ in.})(1.0) \\ &= 3.00\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} T_n &= F_u A_e && \text{(from } \textit{Spec.}\text{ Eq. J4-2)} \\ &= (58\text{ ksi})(3.00\text{ in.}^2) \\ &= 174\text{ kips} \end{aligned}$$

Block Shear Rupture—Beam Web

From AISC *Specification* Section J4.3, the nominal block shear rupture strength of the beam web with a “U” shaped failure plane is determined as follows (including a ¼-in. tolerance to account for possible beam underrun):

$$T_n = 0.60F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.60F_y A_{gv} + U_{bs} F_u A_{nt} \qquad \text{(from } \textit{Spec.}\text{ Eq. J4-5)}$$

where

$$\begin{aligned} A_{gv} &= 2l_{eh}t_w \\ &= (2)(1\tfrac{3}{4}\text{ in.} - \tfrac{1}{4}\text{ in.})(0.300\text{ in.}) \\ &= 0.900\text{ in.}^2 \end{aligned}$$

$$\begin{aligned}A_{nv} &= (2)\left[l_{eh} - 0.5\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t_w \\ &= (2)\left[\left(1\tfrac{3}{4}\text{ in.} - \tfrac{1}{4}\text{ in.}\right) - 0.5\left(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right](0.300\text{ in.}) \\ &= 0.638\text{ in.}^2\end{aligned}$$

$$\begin{aligned}A_{nt} &= \left[9.00\text{ in.} - 3\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t_w \\ &= \left[9.00\text{ in.} - 3\left(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right](0.300\text{ in.}) \\ &= 1.91\text{ in.}^2\end{aligned}$$

$$U_{bs} = 1.0$$

$$\begin{aligned}T_n &= 0.60(65\text{ ksi})\left(0.638\text{ in.}^2\right) + 1.0(65\text{ ksi})\left(1.91\text{ in.}^2\right) \le 0.60(50\text{ ksi})\left(0.900\text{ in.}^2\right) + 1.0(65\text{ ksi})\left(1.91\text{ in.}^2\right) \\ &= 149\text{ kips} < 151\text{ kips}\end{aligned}$$

Therefore:

$$T_n = 149\text{ kips}$$

Nominal Tensile Strength

The controlling tensile strength, T_n, is the least of those previously calculated:

$$\begin{aligned}T_n &= \min\left\{\begin{array}{l}95.5\text{ kips, } 159\text{ kips, } 196\text{ kips, } 176\text{ kips, } 142\text{ kips, } 128\text{ kips, } 34.2\text{ kips, } 144\text{ kips, } 155\text{ kips,} \\ 174\text{ kips, } 149\text{ kips}\end{array}\right\} \\ &= 34.2\text{ kips}\end{aligned}$$

LRFD	ASD
$T_n = 34.2\text{ kips} > 26.5\text{ kips}$ **o.k.**	$T_n = 34.2\text{ kips} > 26.5\text{ kips}$ **o.k.**

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EXAMPLE II.A-29 BOLTED/WELDED SINGLE-ANGLE CONNECTION (BEAM-TO-COLUMN FLANGE)

Given:

Verify a single-angle connection between an ASTM A992 W16×50 beam and an ASTM A992 W14×90 column flange, as shown in Figure II.A-29-1, to support the following beam end reactions:

$R_D = 9$ kips
$R_L = 27$ kips

Use an ASTM A36 single angle. Use 70-ksi electrode welds to connect the single angle to the column flange.

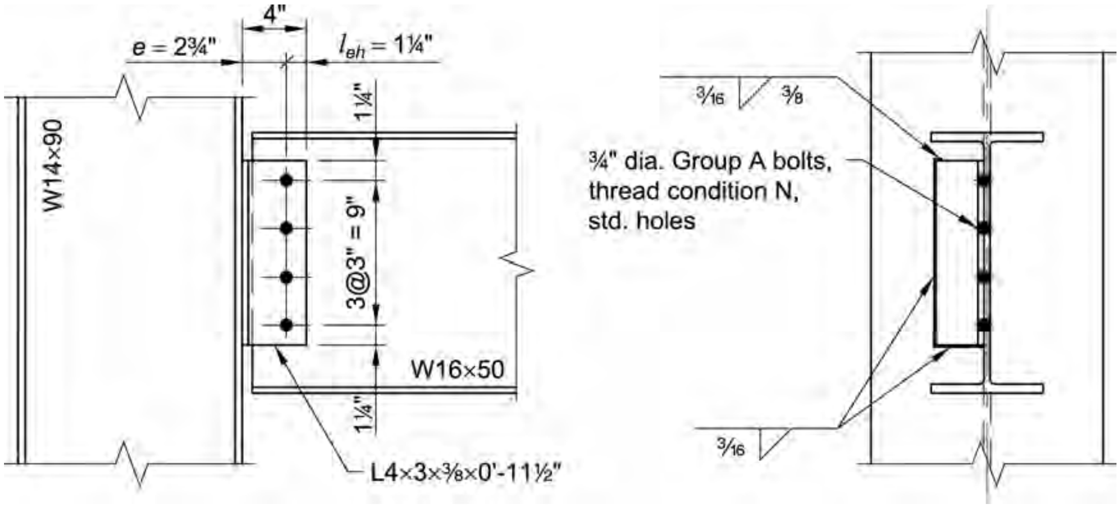

Fig. II.A-29-1. Connection geometry for Example II.A-29.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam and column
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Angle
ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W16×50
$t_w = 0.380$ in.
$d = 16.3$ in.
$t_f = 0.630$ in.

Column
W14×90
$t_f = 0.710$

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(9 \text{ kips}) + 1.6(27 \text{ kips})$ $= 54.0 \text{ kips}$	$R_a = 9 \text{ kips} + 27 \text{ kips}$ $= 36.0 \text{ kips}$

Single Angle, Bolts and Welds

Check eccentricity of the connection.

For the 4-in. angle leg attached to the supported beam:

e = 2¾ in. < 3.00 in., therefore, eccentricity does not need to be considered for this leg.

For the 3-in. angle leg attached to the supporting column flange:

Because the half-web dimension of the W16×50 supported beam is less than ¼ in., AISC *Manual* Table 10-12 may conservatively be used.

Use a four-bolt single-angle (L4×3×⅜).

From AISC *Manual* Table 10-12, the bolt and angle available strength is:

LRFD	ASD
$\phi R_n = 71.4 \text{ kips} > 54.0 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 47.6 \text{ kips} > 36.0 \text{ kips}$ **o.k.**

From AISC *Manual* Table 10-12, the available weld strength for a ³⁄₁₆-in. fillet weld is:

LRFD	ASD
$\phi R_n = 56.6 \text{ kips} > 54.0 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 37.8 \text{ kips} > 36.0 \text{ kips}$ **o.k.**

Support Thickness

The minimum support thickness that matches the column flange strength to the ³⁄₁₆-in. fillet weld strength is:

$$t_{min} = \frac{3.09D}{F_u} \qquad \text{(}\textit{Manual}\text{ Eq. 9-2)}$$

$$= \frac{3.09(3)}{65 \text{ ksi}}$$

$$= 0.143 \text{ in.} < 0.710 \text{ in.} \quad \textbf{o.k.}$$

Note: The minimum thickness values listed in Table 10-12 are for conditions with angles on both sides of the web.

Use a four-bolt single-angle, L4×3×⅜. The 3-in. leg will be shop welded to the column flange and the 4-in. leg will be field bolted to the beam web.

Supported Beam Web

The available bearing and tearout strength of the beam web is determined using AISC *Manual* Table 7-4 with s = 3 in.

LRFD	ASD
$\phi R_n = (4 \text{ bolts})(87.8 \text{ kip/in.})(0.380 \text{ in.})$ $= 133 \text{ kips} > 54.0 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = (4 \text{ bolts})(58.5 \text{ kip/in.})(0.380 \text{ in.})$ $= 88.9 \text{ kips} > 36.0 \text{ kips}$ **o.k.**

Conclusion

The connection is found to be adequate as given for the applied load.

EXAMPLE II.A-30 ALL-BOLTED TEE CONNECTION (BEAM-TO-COLUMN FLANGE)

Given:

Verify an all-bolted tee connection between an ASTM A992 W16×50 beam and an ASTM A992 W14×90 column flange, as shown in Figure II.A-30-1, to support the following beam end reactions:

$R_D = 9$ kips
$R_L = 27$ kips

Use an ASTM A992 WT5×22.5 with a four-bolt connection.

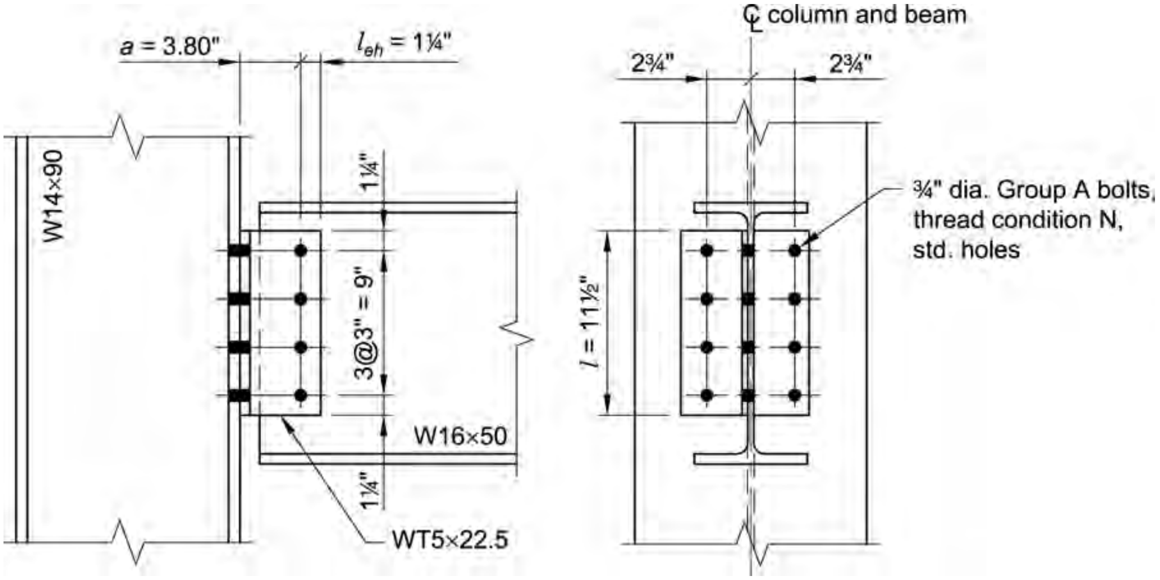

Fig. II.A-30-1. Connection geometry for Example II.A-30.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam, column and tee
ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

From AISC *Manual* Tables 1-1 and 1-8, the geometric properties are as follows:

Beam
W16×50
$t_w = 0.380$ in.
$d = 16.3$ in.
$t_f = 0.630$ in.

Column
W14×90
$t_f = 0.710$ in.

Tee
WT5×22.5
d = 5.05 in.
b_f = 8.02 in.
t_f = 0.620 in.
t_{sw} = 0.350 in.
k_1 = 13⁄16 in. (see W10×45 AISC *Manual* Table 1-1)
k_{des} = 1.12 in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(9\text{ kips}) + 1.6(27\text{ kips})$ $= 54.0\text{ kips}$	$R_a = 9\text{ kips} + 27\text{ kips}$ $= 36.0\text{ kips}$

Limitation on Tee Stem or Beam Web Thickness

See rotational ductility discussion at the beginning of the AISC *Manual* Part 9.

For the tee stem, the maximum tee stem thickness is:

$$\begin{aligned} t_{sw\,max} &= \frac{d}{2} + \tfrac{1}{16}\text{ in.} \\ &= \frac{\tfrac{3}{4}\text{ in.}}{2} + \tfrac{1}{16}\text{ in.} \\ &= 0.438\text{ in.} > 0.350\text{ in.} \quad \textbf{o.k.} \end{aligned}$$

(*Manual* Eq. 9-39)

For W16×50 beam web, the maximum beam web thickness is:

$$\begin{aligned} t_{w\,max} &= \frac{d}{2} + \tfrac{1}{16}\text{ in.} \\ &= \frac{\tfrac{3}{4}\text{ in.}}{2} + \tfrac{1}{16}\text{ in.} \\ &= 0.438\text{ in.} > 0.380\text{ in.} \quad \textbf{o.k.} \end{aligned}$$

(from *Manual* Eq. 9-39)

Limitation on Bolt Diameter for Bolts through Tee Flange

Note: The bolts are not located symmetrically with respect to the centerline of the tee.

b = flexible width in connection element (see AISC *Manual* Figure 9-6)

$$\begin{aligned} &= 2\tfrac{3}{4}\text{ in.} - \frac{t_{sw}}{2} - \frac{t_w}{2} - k_1 \\ &= 2\tfrac{3}{4}\text{ in.} - \frac{0.350\text{ in.}}{2} - \frac{0.380\text{ in.}}{2} - \tfrac{13}{16}\text{ in.} \\ &= 1.57\text{ in.} \end{aligned}$$

$$d_{min} = 0.163 t_f \sqrt{\frac{F_y}{b}\left(\frac{b^2}{l^2}+2\right)} \le 0.69\sqrt{t_{sw}} \qquad \text{(}\textit{Manual}\text{ Eq. 9-38)}$$

$$= 0.163(0.620 \text{ in.})\sqrt{\left(\frac{50 \text{ ksi}}{1.57 \text{ in.}}\right)\left[\frac{(1.57 \text{ in.})^2}{(11\tfrac{1}{2} \text{ in.})^2}+2\right]} \le 0.69\sqrt{0.350 \text{ in.}}$$

$$= 0.810 \text{ in.} > 0.408 \text{ in.}$$

Therefore:

$d_{min} = 0.408 \text{ in.} < \tfrac{3}{4} \text{ in.}$ **o.k.**

Because the connection is rigid at the support, the bolts through the tee stem must be designed for shear, but do not need to be designed for an eccentric moment.

Strength of the Bolted Connection—Tee

From the Commentary to AISC *Specification* Section J3.6, the strength of the bolt group is taken as the sum of the invividual strengths of the individual fasteners, taken as the lesser of the fastener shear strength per AISC *Specification* Section J3.6, the bearing strength at the bolt hole per AISC *Specification* Section J3.10, or the tearout strength at the bolt hole per AISC *Specification* Section J3.10.

From AISC *Manual* Table 7-1, the available shear strength per bolt for ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) is:

LRFD	ASD
$\phi r_n = 17.9 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 11.9 \text{ kips/bolt}$

The available bearing and tearout strength of the tee at the bottom edge bolt is determined using AISC *Manual* Table 7-5, with l_e = 1¼ in., as follows:

LRFD	ASD
$\phi r_n = (49.4 \text{ kip/in.})(0.350 \text{ in.})$ $= 17.3 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (32.9 \text{ kip/in.})(0.350 \text{ in.})$ $= 11.5 \text{ kips/bolt}$

The bearing or tearout strength controls over bolt shear for the bottom edge bolt in the tee.

The available bearing and tearout strength of the tee at the interior bolts (not adjacent to the edge) is determined using AISC *Manual* Table 7-4 with s = 3 in.

LRFD	ASD
$\phi r_n = (87.8 \text{ kip/in.})(0.350 \text{ in.})$ $= 30.7 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (58.5 \text{ kip/in.})(0.350 \text{ in.})$ $= 20.5 \text{ kips/bolt}$

The bolt shear strength controls over bearing or tearout for the interior bolts in the tee.

The strength of the bolt group in the beam web is determined by summing the strength of the individual fasteners as follows:

LRFD	ASD
$\phi R_n = (1\text{ bolt})(17.3\text{ kips/bolt})$ $\quad + (3\text{ bolts})(17.9\text{ kips/bolt})$ $= 71.0\text{ kips} > 54.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = (1\text{ bolt})(11.5\text{ kips/bolt})$ $\quad + (3\text{ bolts})(11.9\text{ kips/bolt})$ $= 47.2\text{ kips} > 36.0\text{ kips}$ **o.k.**

Strength of the Bolted Connection—Beam Web

The available bearing and tearout strength for all bolts in the beam web is determined using AISC *Manual* Table 7-4 with $s = 3$ in.

LRFD	ASD
$\phi r_n = (87.8\text{ kip/in.})(0.380\text{ in.})$ $= 33.4\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (58.5\text{ kip/in.})(0.380\text{ in.})$ $= 22.2\text{ kips/bolt}$

The bolt shear strength controls over bearing or tearout in the beam web; therefore, the beam web is adequate based on previous calculations.

Flexural Yielding of Stem

The flexural yielding strength is checked at the junction of the stem and the fillet. The required flexural strength is determined as follows:

LRFD	ASD
$M_u = P_u e$ $= P_u(a - k_{des})$ $= (54.0\text{ kips})(3.80\text{ in.} - 1.12\text{ in.})$ $= 145\text{ kip-in.}$	$M_a = P_a e$ $= P_a(a - k_{des})$ $= (36.0\text{ kips})(3.80\text{ in.} - 1.12\text{ in.})$ $= 96.5\text{ kip-in.}$

The available flexural strength of the tee stem is determined as follows:

LRFD	ASD
$\phi = 0.90$ $\phi M_n = \phi F_y Z_x$ $= 0.90(50\text{ ksi})\left[\dfrac{(0.350\text{ in.})(11\tfrac{1}{2}\text{ in.})^2}{4}\right]$ $= 521\text{ kip-in.} > 145\text{ kip-in.}$ **o.k.**	$\Omega = 1.67$ $\dfrac{M_n}{\Omega} = \dfrac{F_y Z_x}{\Omega}$ $= \left(\dfrac{50\text{ ksi}}{1.67}\right)\left[\dfrac{(0.350\text{ in.})(11\tfrac{1}{2}\text{ in.})^2}{4}\right]$ $= 346\text{ kip-in.} > 96.5\text{ kip-in.}$ **o.k.**

Shear Strength of Stem

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the tee stem is determined as follows:

$$
\begin{aligned}
A_{gv} &= l t_{sw} \\
&= (11\tfrac{1}{2}\text{ in.})(0.350\text{ in.}) \\
&= 4.03\text{ in.}^2
\end{aligned}
$$

$$\begin{aligned} R_n &= 0.60F_yA_{gv} \\ &= 0.60(50\text{ ksi})(4.03\text{ in.}^2) \\ &= 121\text{ kips} \end{aligned} \qquad (\textit{Spec.}\text{ Eq. J4-3})$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(121\text{ kips})$ $= 121\text{ kips} > 54.0\text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{121\text{ kips}}{1.50}$ $= 80.7\text{ kips} > 36.0\text{ kips}$ **o.k.**

From AISC *Specification* Section J4.2, the available shear rupture strength of the tee stem is determined using the net area determined in accordance with AISC *Specification* Section B4.3b.

$$\begin{aligned} A_{nv} &= \left[l - n\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t_{sw} \\ &= \left[11\tfrac{1}{2}\text{ in.} - 4\left(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right](0.350\text{ in.}) \\ &= 2.80\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60F_uA_{nv} \\ &= 0.60(65\text{ ksi})(2.80\text{ in.}^2) \\ &= 109\text{ kips} \end{aligned} \qquad (\textit{Spec.}\text{ Eq. J4-4})$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(109\text{ kips})$ $= 81.8\text{ kips} > 54.0\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{109\text{ kips}}{2.00}$ $= 54.5\text{ kips} > 36.0\text{ kips}$ **o.k.**

Block Shear Rupture of Stem

The nominal strength for the limit state of block shear rupture is given by AISC *Specification* Section J4.3.

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad (\textit{Spec.}\text{ Eq. J4-5})$$

The available block shear rupture strength of the tee stem is determined as follows, using AISC *Manual* Tables 9-3a, 9-3b and 9-3c, and AISC *Specification* Equation J4-5, with $n = 4$, $l_{eh} = l_{ev} = 1\tfrac{1}{4}$ in., and $U_{bs} = 1.0$.

LRFD	ASD
Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{\phi F_uA_{nt}}{t} = 39.6\text{ kip/in.}$	Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{F_uA_{nt}}{\Omega t} = 26.4\text{ kip/in.}$

LRFD	ASD
Shear yielding component from AISC *Manual* Table 9-3b:	Shear yielding component from AISC *Manual* Table 9-3b:
$\dfrac{\phi 0.60 F_y A_{gv}}{t} = 231 \text{ kip/in.}$	$\dfrac{0.60 F_y A_{gv}}{\Omega t} = 154 \text{ kip/in.}$
Shear rupture component from AISC *Manual* Table 9-3c:	Shear rupture component from AISC *Manual* Table 9-3c:
$\dfrac{\phi 0.60 F_u A_{nv}}{t} = 210 \text{ kip/in.}$	$\dfrac{0.60 F_u A_{nv}}{\Omega t} = 140 \text{ kip/in.}$
The design block shear rupture strength is:	The allowable block shear rupture strength is:
$\phi R_n = \phi 0.60 F_u A_{nv} + \phi U_{bs} F_u A_{nt}$ $\le \phi 0.60 F_y A_{gv} + \phi U_{bs} F_u A_{nt}$ $= (0.350 \text{ in.})[210 \text{ kip/in.} + (1.0)(39.6 \text{ kip/in.})]$ $\le (0.350 \text{ in.})[231 \text{ kip/in.} + (1.0)(39.6 \text{ kip/in.})]$ $= 87.4 \text{ kips} < 94.7 \text{ kips}$	$\dfrac{R_n}{\Omega} = \dfrac{0.60 F_u A_{nv}}{\Omega} + \dfrac{U_{bs} F_u A_{nt}}{\Omega}$ $\le \dfrac{0.60 F_y A_{gv}}{\Omega} + \dfrac{U_{bs} F_u A_{nt}}{\Omega}$ $= (0.350 \text{ in.})[140 \text{ kip/in.} + (1.0)(26.4 \text{ kip/in.})]$ $\le (0.350 \text{ in.})[154 \text{ kip/in.} + (1.0)(26.4 \text{ kip/in.})]$ $= 58.2 \text{ kips} < 63.1 \text{ kips}$
Therefore:	Therefore:
$\phi R_n = 87.4 \text{ kips} > 54.0 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 58.2 \text{ kips} > 36.0 \text{ kips}$ **o.k.**

Because the connection is rigid at the support, the bolts attaching the tee flange to the support must be designed for the shear and the eccentric moment.

Bolt Group at Column

Check bolts for shear and bearing combined with tension due to eccentricity.

The following calculation follows the Case II approach in the Section “Eccentricity Normal to the Plane of the Faying Surface” in Part 7 of the AISC *Manual*. The available shear strength of the bolts is determined as follows:

LRFD	ASD
$\phi r_n = 17.9 \text{ kips/bolt}$ (from AISC *Manual* Table 7-1)	$\dfrac{r_n}{\Omega} = 11.9 \text{ kips/bolt}$ (from AISC *Manual* Table 7-1)
$r_{uv} = \dfrac{P_u}{n}$ (*Manual* Eq. 7-13a) $= \dfrac{54.0 \text{ kips}}{8 \text{ bolts}}$ $= 6.75 \text{ kips/bolt} < 17.9 \text{ kips/bolt}$ **o.k.**	$r_{av} = \dfrac{P_a}{n}$ (*Manual* Eq. 7-13b) $= \dfrac{36.0 \text{ kips}}{8 \text{ bolts}}$ $= 4.50 \text{ kips/bolt} < 11.9 \text{ kips/bolt}$ **o.k.**
$A_b = 0.442 \text{ in.}^2$ (from AISC *Manual* Table 7-1)	$A_b = 0.442 \text{ in.}^2$ (from AISC *Manual* Table 7-1)

LRFD	ASD
$f_{rv} = \dfrac{r_{uv}}{A_b}$ $= \dfrac{6.75 \text{ kips/bolt}}{0.442 \text{ in.}^2}$ $= 15.3 \text{ ksi}$	$f_{rv} = \dfrac{r_{av}}{A_b}$ $= \dfrac{4.50 \text{ kips/bolt}}{0.442 \text{ in.}^2}$ $= 10.2 \text{ ksi}$

The nominal tensile stress modified to include the effects of shear stress is determined from AISC *Specification* Section J3.7 as follows. From AISC *Specification* Table J3.2:

$$F_{nt} = 90 \text{ ksi}$$
$$F_{nv} = 54 \text{ ksi}$$

LRFD	ASD
Tensile force per bolt, r_{ut}: $r_{ut} = \dfrac{P_u e}{n' d_m}$ (*Manual* Eq. 7-14a) $= \dfrac{(54.0 \text{ kips})(3.80 \text{ in.})}{(4 \text{ bolts})(6.00 \text{ in.})}$ $= 8.55 \text{ kips/bolt}$ $\phi = 0.75$ $F'_{nt} = 1.3F_{nt} - \dfrac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt}$ (*Spec*. Eq. J3-3a) $= 1.3(90 \text{ ksi}) - \dfrac{90 \text{ ksi}}{0.75(54 \text{ ksi})}(15.3 \text{ ksi}) \le 90 \text{ ksi}$ $= 83.0 \text{ ksi} < 90 \text{ ksi}$ $= 83.0 \text{ ksi}$ $\phi r_n = \phi F'_{nt} A_b$ (from *Spec*. Eq. J3-2) $= 0.75(83.0 \text{ ksi})(0.442 \text{ in.}^2)$ $= 27.5 \text{ kips/bolt} > 8.55 \text{ kips/bolt}$ **o.k.**	Tensile force per bolt, r_{at}: $r_{at} = \dfrac{P_a e}{n' d_m}$ (*Manual* Eq. 7-14b) $= \dfrac{(36.0 \text{ kips})(3.80 \text{ in.})}{(4 \text{ bolts})(6.00 \text{ in.})}$ $= 5.70 \text{ kips/bolt}$ $\Omega = 2.00$ $F'_{nt} = 1.3F_{nt} - \dfrac{\Omega F_{nt}}{F_{nv}} f_{rv} \le F_{nt}$ (*Spec*. Eq. J3-3b) $= 1.3(90 \text{ ksi}) - \dfrac{2.00(90 \text{ ksi})}{54 \text{ ksi}}(10.2 \text{ ksi}) \le 90 \text{ ksi}$ $= 83.0 \text{ ksi} < 90 \text{ ksi}$ $= 83.0 \text{ ksi}$ $\dfrac{r_n}{\Omega} = \dfrac{F'_{nt} A_b}{\Omega}$ (from *Spec*. Eq. J3-2) $= \dfrac{(83.0 \text{ ksi})(0.442 \text{ in.}^2)}{2.00}$ $= 18.3 \text{ kips/bolt} > 5.70 \text{ kips/bolt}$ **o.k.**

With l_e = 1¼ in. and s = 3 in., the bearing or tearout strength of the tee flange exceeds the single shear strength of the bolts. Therefore, the bearing and tearout strength is adequate.

Prying Action

From AISC *Manual* Part 9, the available tensile strength of the bolts taking prying action into account is determined as follows. By inspection, prying action in the tee will control over prying action in the column.

Note: The bolts are not located symmetrically with respect to the centerline of the tee.

$$a = \frac{b_f}{2} - \frac{t_w}{2} - \frac{t_{sw}}{2} - 2\tfrac{3}{4}\text{ in.}$$
$$= \frac{8.02\text{ in.}}{2} - \frac{0.380\text{ in.}}{2} - \frac{0.350\text{ in.}}{2} - 2\tfrac{3}{4}\text{ in.}$$
$$= 0.895\text{ in.}$$

$$b = 2\tfrac{3}{4}\text{ in.} + \frac{t_w}{2}$$
$$= 2\tfrac{3}{4}\text{ in.} + \frac{0.380\text{ in.}}{2}$$
$$= 2.94\text{ in.}$$

$$a' = \left(a + \frac{d_b}{2}\right) \le \left(1.25b + \frac{d_b}{2}\right) \qquad (\textit{Manual}\text{ Eq. 9-23})$$
$$= 0.895\text{ in.} + \frac{\tfrac{3}{4}\text{ in.}}{2} \le 1.25(2.94\text{ in.}) + \frac{\tfrac{3}{4}\text{ in.}}{2}$$
$$= 1.27\text{ in.} < 4.05\text{ in.}$$
$$= 1.27\text{ in.}$$

$$b' = \left(b - \frac{d_b}{2}\right) \qquad (\textit{Manual}\text{ Eq. 9-18})$$
$$= 2.94\text{ in.} - \frac{\tfrac{3}{4}\text{ in.}}{2}$$
$$= 2.57\text{ in.}$$

$$\rho = \frac{b'}{a'} \qquad (\textit{Manual}\text{ Eq. 9-22})$$
$$= \frac{2.57\text{ in.}}{1.27\text{ in.}}$$
$$= 2.02$$

$$p = l_{ev} + 0.5s$$
$$= 1\tfrac{1}{4}\text{ in.} + 0.5(3\text{ in.})$$
$$= 2.75\text{ in.}$$

Check:

$p \le s$

$2.75\text{ in.} < 3\text{ in.}$ **o.k.**

$p \le l_{ev} + 1.75b$

$2.75\text{ in.} \le 1\tfrac{1}{4}\text{ in.} + 1.75(2.94\text{ in.})$

$2.75\text{ in.} < 6.40\text{ in.}$ **o.k.**

$$d' = d_h$$
$$= \tfrac{13}{16}\text{ in.}$$

$$\delta = 1 - \frac{d'}{p} \qquad \text{(\textit{Manual} Eq. 9-20)}$$

$$= 1 - \frac{{}^{13}\!/_{16}\text{ in.}}{2.75\text{ in.}}$$

$$= 0.705$$

LRFD	ASD
$T_r = r_{ut}$ $= 8.55\text{ kips/bolt}$ $B_c = \phi r_n$ $= 27.5\text{ kips/bolt}$ $\beta = \frac{1}{\rho}\left(\frac{B_c}{T_r} - 1\right)$ (*Manual* Eq. 9-21) $= \frac{1}{2.02}\left[\left(\frac{27.5\text{ kips/bolt}}{8.55\text{ kips/bolt}}\right) - 1\right]$ $= 1.10$ Because $\beta \geq 1$, set $\alpha' = 1.0$. $\phi = 0.90$ $t_{min} = \sqrt{\frac{4T_u b'}{\phi p F_u\left(1 + \delta\alpha'\right)}}$ (*Manual* Eq. 9-19a) $= \sqrt{\frac{4\left(8.55\text{ kips/bolt}\right)\left(2.57\text{ in.}\right)}{0.90\left(2.75\text{ in.}\right)\left(65\text{ ksi}\right)\left[1 + \left(0.705\right)\left(1.0\right)\right]}}$ $= 0.566\text{ in.} < 0.620\text{ in.}$ **o.k.**	$T_r = r_{at}$ $= 5.70\text{ kips/bolt}$ $B_c = \frac{r_n}{\Omega}$ $= 18.3\text{ kips/bolt}$ $\beta = \frac{1}{\rho}\left(\frac{B_c}{T_r} - 1\right)$ (*Manual* Eq. 9-21) $= \frac{1}{2.02}\left[\left(\frac{18.3\text{ kips/bolt}}{5.70\text{ kips/bolt}}\right) - 1\right]$ $= 1.09$ Because $\beta \geq 1$, set $\alpha' = 1.0$. $\Omega = 1.67$ $t_{min} = \sqrt{\frac{\Omega 4T_a b'}{p F_u\left(1 + \delta\alpha'\right)}}$ (*Manual* Eq. 9-19b) $= \sqrt{\frac{1.67\left(4\right)\left(5.70\text{ kips/bolt}\right)\left(2.57\text{ in.}\right)}{\left(2.75\text{ in.}\right)\left(65\text{ ksi}\right)\left[1 + \left(0.705\right)\left(1.0\right)\right]}}$ $= 0.567\text{ in.} < 0.620\text{ in.}$ **o.k.**

Similarly, checks of the tee flange for shear yielding, shear rupture, and block shear rupture will show that the tee flange is adequate.

Bolt Bearing on Column Flange

The available bearing and tearout strength of the column flange is determined using AISC *Manual* Table 7-4 with s = 3 in.

LRFD	ASD
$\phi R_n = \left(8\text{ bolts}\right)\left(87.8\text{ kip/in.}\right)\left(0.710\text{ in.}\right)$ $= 499\text{ kips} > 54.0\text{ kips}$ **o.k.**	$\frac{R_n}{\Omega} = \left(8\text{ bolts}\right)\left(58.5\text{ kip/in.}\right)\left(0.710\text{ in.}\right)$ $= 332\text{ kips} > 36.0\text{ kips}$ **o.k.**

Note: Although the edge distance (a = 0.895 in.) for one row of bolts in the tee flange does not meet the minimum value indicated in AISC *Specification* Table J3.4, based on footnote [a], the edge distance provided is acceptable because the provisions of AISC *Specification* Section J3.10 and J4.4 have been met in this case.

Conclusion

The connection is found to be adequate as given for the applied load.

EXAMPLE II.A-31 BOLTED/WELDED TEE CONNECTION (BEAM-TO-COLUMN FLANGE)

Given:

Verify the tee connection bolted to an ASTM A992 W16×50 supported beam and welded to an ASTM A992 W14×90 supporting column flange, as shown in Figure II.A-31-1, to support the following beam end reactions:

R_D = 6 kips
R_L = 18 kips

Use 70-ksi electrodes. Use an ASTM A992 WT5×22.5 with a four-bolt connection to the beam web.

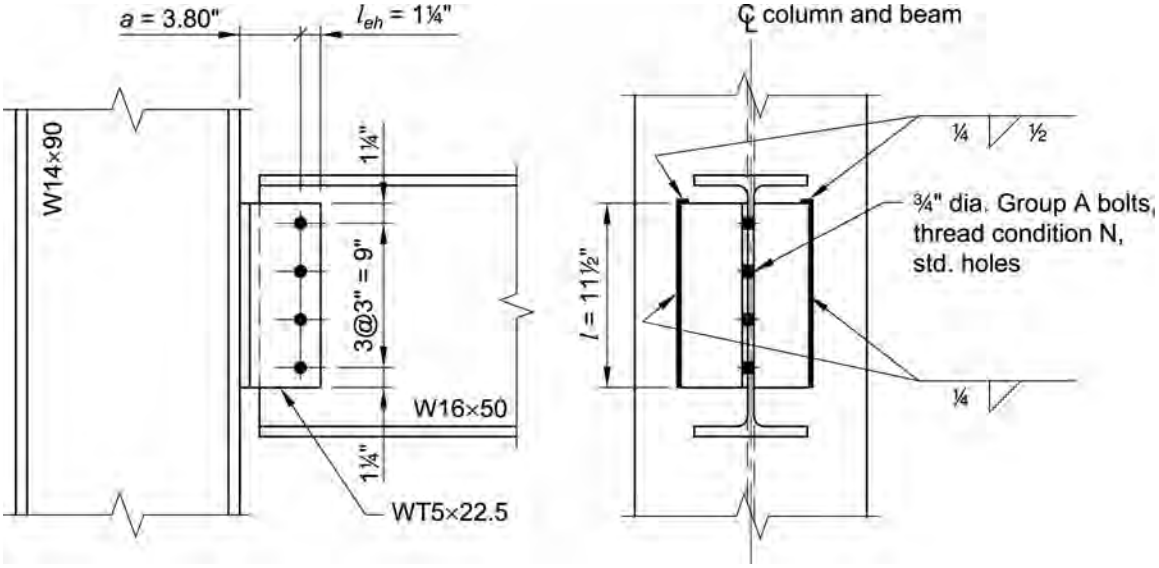

Fig. II.A-31-1. Connection geometry for Example II.A-31.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam, column and tee
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Tables 1-1 and 1-8, the geometric properties are as follows:

Beam
W16×50
t_w= 0.380 in.
d = 16.3 in.
t_f = 0.630 in.

Column
W14×90
t_f = 0.710 in.

Tee
WT5×22.5
$d\ \ = 5.05$ in.
$b_f\ \ = 8.02$ in.
$t_f\ \ = 0.620$ in.
$t_{sw} = 0.350$ in.
$k_1\ \ = 13/16$ in. (see W10×45, AISC *Manual* Table 1-1)
$k_{des} = 1.12$ in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(6 \text{ kips}) + 1.6(18 \text{ kips})$ $= 36.0 \text{ kips}$	$R_a = 6 \text{ kips} + 18 \text{ kips}$ $= 24.0 \text{ kips}$

Limitation on Tee Stem or Beam Web Thickness

See rotational ductility discussion at the beginning of AISC *Manual* Part 9.

For the tee stem, the maximum tee stem thickness is:

$$t_{sw\ max} = \frac{d}{2} + 1/16 \text{ in.} \qquad (\textit{Manual}\text{ Eq. 9-39})$$
$$= \frac{3/4 \text{ in.}}{2} + 1/16 \text{ in.}$$
$$= 0.438 \text{ in.} > 0.350 \text{ in.} \quad \textbf{o.k.}$$

For W16×50 beam web, the maximum beam web thickness is:

$$t_{w\ max} = \frac{d}{2} + 1/16 \text{ in.} \qquad (\textit{Manual}\text{ Eq. 9-39})$$
$$= \frac{3/4 \text{ in.}}{2} + 1/16 \text{ in.}$$
$$= 0.438 \text{ in.} > 0.380 \text{ in.} \quad \textbf{o.k.}$$

Weld Design

b = flexible width in connection element

$$= \frac{b_f - 2k_1}{2}$$
$$= \frac{8.02 \text{ in.} - 2(13/16 \text{ in.})}{2}$$
$$= 3.20 \text{ in.}$$

$$w_{min} = 0.0155\frac{F_y t_f^2}{b}\left(\frac{b^2}{l^2}+2\right) \le \left(5/8\right)t_{sw} \qquad \text{(}\textit{Manual}\text{ Eq. 9-37)}$$

$$= 0.0155\left[\frac{(50\text{ ksi})(0.620\text{ in.})^2}{3.20\text{ in.}}\right]\left[\frac{(3.20\text{ in.})^2}{(11\frac{1}{2}\text{ in.})^2}+2\right] \le \left(5/8\right)(0.350\text{ in.})$$

$$= 0.193\text{ in.} < 0.219\text{ in.}$$

$$= 0.193\text{ in.}$$

The minimum weld size is ¼ in. per AISC *Specification* Table J2.4.

Try ¼-in. fillet welds.

From AISC *Manual* Table 10-2, with $n = 4$, $l = 11\frac{1}{2}$ in., and Welds B = ¼ in.:

LRFD	ASD
$\phi R_n = 79.9\text{ kips} > 36.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 53.3\text{ kips} > 24.0\text{ kips}$ **o.k..**

Use ¼-in. fillet welds.

Supporting Column Flange

From AISC *Manual* Table 10-2, with $n = 4$, $l = 11\frac{1}{2}$ in., and Welds B = ¼ in., the minimum support thickness is 0.190 in.

$$t_f = 0.710\text{ in.} > 0.190\text{ in.} \quad \textbf{o.k.}$$

Strength of Bolted Connection

From the Commentary to AISC *Specification* Section J3.6, the strength of the bolt group is taken as the sum of the invividual strengths of the individual fasteners, taken as the lesser of the fastener shear strength per AISC *Specification* Section J3.6, the bearing strength at the bolt hole per AISC *Specification* Section J3.10, or the tearout strength at the bolt hole per AISC *Specification* Section J3.10. The 3-in. angle leg attached to the supporting girder will control because eccentricity must be taken into consideration.

Because the connection is flexible at the support, the tee stem and bolts must be designed for eccentric shear, where the eccentricity, e_b, is determined as follows:

$$e_b = a$$

$$= d - l_{eh}$$

$$= 5.05\text{ in.} - 1\frac{1}{4}\text{ in.}$$

$$= 3.80\text{ in.}$$

From AISC *Manual* Table 7-6 for Angle = 0°, with $s = 3$ in., $e_x = e_b = 3.80$ in., and $n = 4$:

$$C = 2.45$$

From AISC *Manual* Table 7-1, the available shear strength per bolt for ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) is:

LRFD	ASD
$\phi r_n = 17.9 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 11.9 \text{ kips/bolt}$

The available bearing and tearout strength of the tee at the bottom edge bolt is determined using AISC *Manual* Table 7-5, with l_e = 1¼ in., as follows:

LRFD	ASD
$\phi r_n = (49.4 \text{ kip/in.})(0.350 \text{ in.})$ $= 17.3 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (32.9 \text{ kip/in.})(0.350 \text{ in.})$ $= 11.5 \text{ kips/bolt}$

The available bearing and tearout strength of the tee at the interior bolts (not adjacent to the edge) is determined using AISC *Manual* Table 7-4 with s = 3 in.

LRFD	ASD
$\phi r_n = (87.8 \text{ kip/in.})(0.350 \text{ in.})$ $= 30.7 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (58.5 \text{ kip/in.})(0.350 \text{ in.})$ $= 20.5 \text{ kips/bolt}$

Note: By inspection, bolt bearing on the beam web does not control.

The available strength of the bolted connection is determined from AISC *Manual* Equation 7-16, conservatively using the minimum available strength calculated for bolt shear, bearing on the tee, and tearout on the tee.

LRFD	ASD
$\phi R_n = C\phi r_n$ $= 2.45(17.3 \text{ kips/bolt})$ $= 42.4 \text{ kips} > 36.0 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = C\dfrac{r_n}{\Omega}$ $= 2.45(11.5 \text{ kips/bolt})$ $= 28.2 \text{ kips} > 24.0 \text{ kips}$ **o.k.**

Flexural Yielding of Tee Stem

The required flexural strength of the tee stem is determined as follows:

LRFD	ASD
$M_u = P_u e_b$ $= (36.0 \text{ kips})(3.80 \text{ in.})$ $= 137 \text{ kip-in.}$	$M_a = P_a e_b$ $= (24.0 \text{ kips})(3.80 \text{ in.})$ $= 91.2 \text{ kip-in.}$

The available flexural yielding strength of the tee stem is determined as follows:

LRFD	ASD
$\phi = 0.90$ $\phi M_n = \phi F_y Z_x$ $= 0.90(50\text{ ksi})\left[\dfrac{(0.350\text{ in.})(11\tfrac{1}{2}\text{ in.})^2}{4}\right]$ $= 521\text{ kip-in.} > 137\text{ kip-in.}$ **o.k.**	$\Omega = 1.67$ $\dfrac{M_n}{\Omega} = \dfrac{F_y Z_x}{\Omega}$ $= \dfrac{50\text{ ksi}}{1.67}\left[\dfrac{(0.350\text{ in.})(11\tfrac{1}{2}\text{ in.})^2}{4}\right]$ $= 346\text{ kip-in.} > 91.2\text{ kip-in.}$ **o.k.**

Flexural Rupture of Tee Stem

The available flexural rupture strength of the plate is determined as follows:

$$\begin{aligned} Z_{net} &= (0.350\text{ in.})\left[\frac{(11\tfrac{1}{2}\text{ in.})^2}{4} - 2(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})(4.50\text{ in.}) - 2(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})(1.50\text{ in.})\right] \\ &= 7.90\text{ in.}^3 \end{aligned}$$

$$\begin{aligned} M_n &= F_u Z_{net} \\ &= (65\text{ ksi})(7.90\text{ in.}^3) \\ &= 514\text{ kip-in.} \end{aligned}$$

(*Manual* Eq. 9-4)

LRFD	ASD
$\phi = 0.75$ $\phi M_n = 0.75(514\text{ kip-in.})$ $= 386\text{ kip-in.} > 137\text{ kip-in.}$ **o.k.**	$\Omega = 2.00$ $\dfrac{M_n}{\Omega} = \dfrac{514\text{ kip-in.}}{2.00}$ $= 257\text{ kip-in.} > 91.2\text{ kip-in.}$ **o.k.**

Shear Strength of Stem

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the tee stem is determined as follows:

$$\begin{aligned} A_{gv} &= lt_{sw} \\ &= (11\tfrac{1}{2}\text{ in.})(0.350\text{ in.}) \\ &= 4.03\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60 F_y A_{gv} \\ &= 0.60(50\text{ ksi})(4.03\text{ in.}^2) \\ &= 121\text{ kips} \end{aligned}$$

(*Spec.* Eq. J4-3)

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(121\text{ kips})$ $= 121\text{ kips} > 36.0\text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{121\text{ kips}}{1.50}$ $= 80.7\text{ kips} > 24.0\text{ kips}$ **o.k.**

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the tee stem is determined using the net area determined in accordance with AISC *Specification* Section B4.3b.

$$
\begin{aligned}
A_{nv} &= \left[l - n\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t_{sw} \\
&= \left[11\tfrac{1}{2}\text{ in.} - 4\left(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right](0.350\text{ in.}) \\
&= 2.80\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
R_n &= 0.60F_uA_{nv} \qquad (\textit{Spec}.\text{ Eq. J4-4}) \\
&= 0.60(65\text{ ksi})\left(2.80\text{ in.}^2\right) \\
&= 109\text{ kips}
\end{aligned}
$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(109\text{ kips})$ $= 81.8\text{ kips} > 36.0\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{109\text{ kips}}{2.00}$ $= 54.5\text{ kips} > 24.0\text{ kips}$ **o.k.**

Block Shear Rupture of Stem

The nominal strength for the limit state of block shear rupture is given by AISC *Specification* Section J4.3.

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad (\textit{Spec}.\text{ Eq. J4-5})$$

The available block shear rupture strength of the tee stem is determined as follows, using AISC *Manual* Tables 9-3a, 9-3b and 9-3c, and AISC *Specification* Equation J4-5, with $n = 4$, $l_{eh} = l_{ev} = 1\frac{1}{4}$ in., and $U_{bs} = 1.0$.

LRFD	ASD
Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{\phi F_uA_{nt}}{t} = 39.6\text{ kip/in.}$ Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{\phi 0.60F_yA_{gv}}{t} = 231\text{ kip/in.}$	Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{F_uA_{nt}}{\Omega t} = 26.4\text{ kip/in.}$ Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{0.60F_yA_{gv}}{\Omega t} = 154\text{ kip/in.}$

LRFD	ASD
Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{\phi 0.60 F_u A_{nv}}{t} = 210\text{ kip/in.}$ The design block shear rupture strength is: $\phi R_n = \phi 0.60 F_u A_{nv} + \phi U_{bs} F_u A_{nt}$ $\qquad \leq \phi 0.60 F_y A_{gv} + \phi U_{bs} F_u A_{nt}$ $= (0.350\text{ in.})\left[210\text{ kip/in.} + (1.0)(39.6\text{ kip/in.})\right]$ $\qquad \leq (0.350\text{ in.})\left[231\text{ kip/in.} + (1.0)(39.6\text{ kip/in.})\right]$ $= 87.4\text{ kips} < 94.7\text{ kips}$ Therefore: $\phi R_n = 87.4\text{ kips} > 36.0\text{ kips}$ **o.k.**	Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{0.60 F_u A_{nv}}{\Omega t} = 140\text{ kip/in.}$ The allowable block shear rupture strength is: $\dfrac{R_n}{\Omega} = \dfrac{0.60 F_u A_{nv}}{\Omega} + \dfrac{U_{bs} F_u A_{nt}}{\Omega}$ $\qquad \leq \dfrac{0.60 F_y A_{gv}}{\Omega} + \dfrac{U_{bs} F_u A_{nt}}{\Omega}$ $= (0.350\text{ in.})\left[140\text{ kip/in.} + (1.0)(26.4\text{ kip/in.})\right]$ $\qquad \leq (0.350\text{ in.})\left[154\text{ kip/in.} + (1.0)(26.4\text{ kip/in.})\right]$ $= 58.2\text{ kips} < 63.1\text{ kips}$ Therefore: $\dfrac{R_n}{\Omega} = 58.2\text{ kips} > 24.0\text{ kips}$ **o.k.**

Conclusion

The connection is found to be adequate as given for the applied load.

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Chapter IIB
Fully Restrained (FR) Moment Connections

The design of fully restrained (FR) moment connections is covered in Part 12 of the AISC *Manual*.

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EXAMPLE II.B-1 BOLTED FLANGE-PLATED FR MOMENT CONNECTION (BEAM-TO-COLUMN FLANGE)

Given:

Verify a bolted flange-plated FR moment connection between an ASTM A992 W18×50 beam and an ASTM A992 W14×99 column flange, as shown in Figure II.B-1-1, to transfer the following beam end reactions:

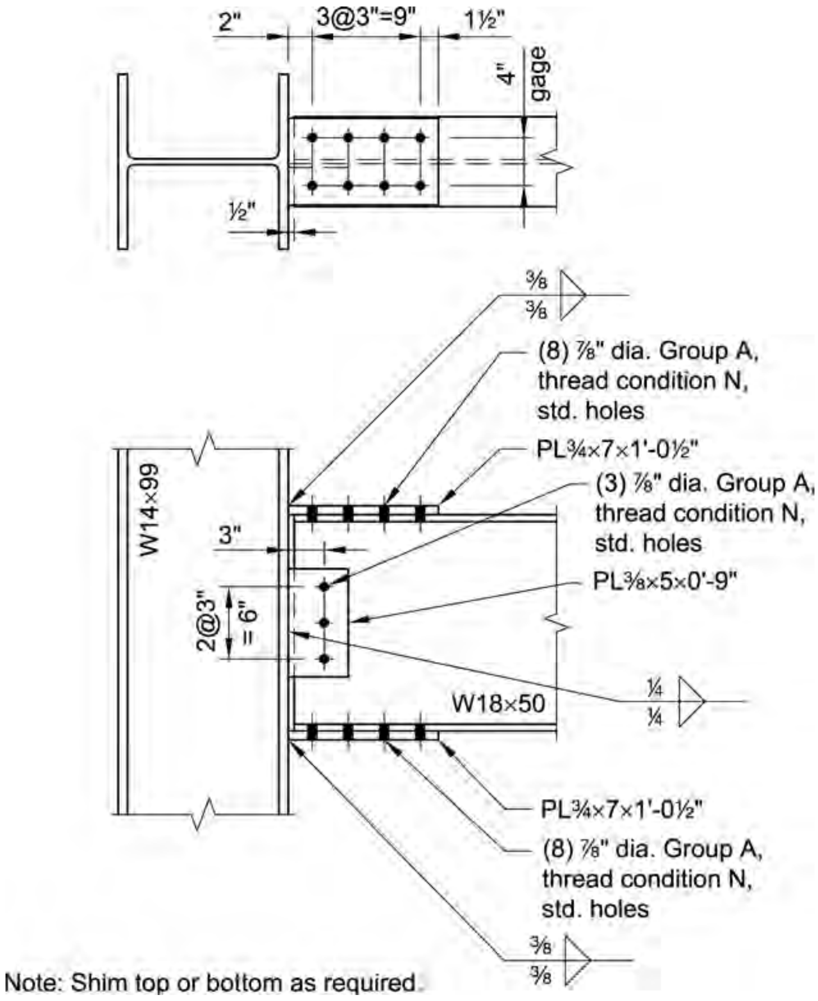
Vertical shear:

$V_D = 7$ kips
$V_L = 21$ kips

Strong-axis moment:

$M_D = 42$ kip-ft
$M_L = 126$ kip-ft

Use 70-ksi electrodes. The flange and web plates are ASTM A36 material. Check the column for stiffening requirements.



Note: Shim top or bottom as required.

Fig. II.B-1-1. Connection geometry for Example II.B-1.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Beam and column
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Plates
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W18×50
d = 18.0 in.
b_f = 7.50 in.
t_f = 0.570 in.
t_w = 0.355 in.
S_x = 88.9 in.^{3}

Column
W14×99
d = 14.2 in.
b_f = 14.6 in.
t_f = 0.780 in.

From AISC *Specification* Table J3.3, the hole diameter for a ⅞-in.-diameter bolt with standard holes is:

$$d_h = {}^{15}\!/\!_{16} \text{ in.}$$

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(7 \text{ kips}) + 1.6(21 \text{ kips})$ $= 42.0 \text{ kips}$ $M_u = 1.2(42 \text{ kip-ft}) + 1.6(126 \text{ kip-ft})$ $= 252 \text{ kip-ft}$	$R_a = 7 \text{ kips} + 21 \text{ kips}$ $= 28.0 \text{ kips}$ $M_a = 42 \text{ kip-ft} + 126 \text{ kip-ft}$ $= 168 \text{ kip-ft}$

Flexural Strength of Beam

From AISC *Specification* Section F13.1, the available flexural strength of the beam is limited according to the limit state of tensile rupture of the tension flange.

$$\begin{aligned} A_{fg} &= b_f t_f \\ &= (7.50 \text{ in.})(0.570 \text{ in.}) \\ &= 4.28 \text{ in.}^2 \end{aligned}$$

The net area of the flange is determined in accordance with AISC *Specification* Section B4.3b.

$$
\begin{aligned}
A_{fn} &= A_{fg} - (2 \text{ bolts})(d_h + \tfrac{1}{16} \text{ in.})t_f \\
&= 4.28 \text{ in.}^2 - (2 \text{ bolts})(\tfrac{15}{16} \text{ in.} + \tfrac{1}{16} \text{ in.})(0.570 \text{ in.}) \\
&= 3.14 \text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
\frac{F_y}{F_u} &= \frac{50 \text{ ksi}}{65 \text{ ksi}} \\
&= 0.769 < 0.8; \text{ therefore, } Y_t = 1.0
\end{aligned}
$$

$$
\begin{aligned}
F_u A_{fn} &= (65 \text{ ksi})(3.14 \text{ in.}^2) \\
&= 204 \text{ kips}
\end{aligned}
$$

$$
\begin{aligned}
Y_t F_y A_{fg} &= 1.0(50 \text{ ksi})(4.28 \text{ in.}^2) \\
&= 214 \text{ kips} > 204 \text{ kips}
\end{aligned}
$$

Therefore, the nominal flexural strength, M_n, at the location of the holes in the tension flange is not greater than:

$$
\begin{aligned}
M_n &= \frac{F_u A_{fn}}{A_{fg}} S_x \qquad (\textit{Spec}. \text{ Eq. F13-1}) \\
&= \left(\frac{204 \text{ kips}}{4.28 \text{ in.}^2}\right)(88.9 \text{ in.}^3) \\
&= 4{,}240 \text{ kip-in. or } 353 \text{ kip-ft}
\end{aligned}
$$

LRFD	ASD
$\phi_b = 0.90$ $\phi M_n = 0.90(353 \text{ kip-ft})$ $= 318 \text{ kip-ft} > 252 \text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $\dfrac{M_n}{\Omega_b} = \dfrac{353 \text{ kip-ft}}{1.67}$ $= 211 \text{ kip-ft} > 168 \text{ kip-ft}$ **o.k.**

Note: The available flexural strength of the beam may be less than that determined based on AISC *Specification* Equation F13-1. Other applicable provisions in AISC *Specification* Chapter F should be checked to possibly determine a lower value for the available flexural strength of the beam.

Single-Plate Web Connection

Strength of the bolted connection—web plate

From the Commentary to AISC *Specification* Section J3.6, the strength of the bolt group is taken as the sum of the strengths of the individual fasteners, which may be taken as the lesser of the fastener shear strength per AISC *Specification* Section J3.6, the bearing strength at the bolt hole per AISC *Specification* Section J3.10, or the tearout strength at the bolt hole per AISC *Specification* Section J3.10.

From AISC *Manual* Table 7-1, the available shear strength per bolt for ⅞-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) is:

LRFD	ASD
$\phi r_n = 24.3 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 16.2 \text{ kips/bolt}$

The available bearing strength of the plate per bolt is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration:

$$\begin{aligned} r_n &= 2.4dtF_u && (\textit{Spec}.\text{ Eq. J3-6a}) \\ &= 2.4(\tfrac{7}{8}\text{ in.})(\tfrac{3}{8}\text{ in.})(58\text{ ksi}) \\ &= 45.7\text{ kips/bolt} \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(45.7\text{ kips/bolt})$ $= 34.3\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{45.7\text{ kips/bolt}}{2.00}$ $= 22.9\text{ kips/bolt}$

The available tearout strength of the plate at the interior bolts is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration.

$$\begin{aligned} l_c &= s - d_h \\ &= 3\text{ in.} - \tfrac{15}{16}\text{ in.} \\ &= 2.06\text{ in.} \end{aligned}$$

$$\begin{aligned} r_n &= 1.2l_c tF_u && (\textit{Spec}.\text{ Eq. J3-6c}) \\ &= 1.2(2.06\text{ in.})(\tfrac{3}{8}\text{ in.})(58\text{ ksi}) \\ &= 53.8\text{ kips/bolt} \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(53.8\text{ kips/bolt})$ $= 40.4\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{53.8\text{ kips/bolt}}{2.00}$ $= 26.9\text{ kips/bolt}$

Therefore, bolt shear controls over bearing or tearout at interior bolts.

The available tearout strength of the plate at the edge bolt is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration.

$$\begin{aligned} l_c &= l_{ev} - 0.5(d_h) \\ &= 1\tfrac{1}{2}\text{ in.} - 0.5(\tfrac{15}{16}\text{ in.}) \\ &= 1.03\text{ in.} \end{aligned}$$

$$\begin{aligned} r_n &= 1.2l_c tF_u && (\textit{Spec}.\text{ Eq. J3-6c}) \\ &= 1.2(1.03\text{ in.})(\tfrac{3}{8}\text{ in.})(58\text{ ksi}) \\ &= 26.9\text{ kips/bolt} \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(26.9 \text{ kips/bolt})$ $= 20.2 \text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{26.9 \text{ kips/bolt}}{2.00}$ $= 13.5 \text{ kips/bolt}$

Therefore, tearout controls over bolt shear or bearing at the edge bolt.

The strength of the bolt group in the plate is determined by summing the strength of the individual fasteners as follows:

LRFD	ASD
$\phi R_n = (1 \text{ bolt})(20.2 \text{ kips/bolt})$ $+ (2 \text{ bolts})(24.3 \text{ kips/bolt})$ $= 68.8 \text{ kips} > 42.0 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = (1 \text{ bolt})(13.5 \text{ kips/bolt})$ $+ (2 \text{ bolts})(16.2 \text{ kips/bolt})$ $= 45.9 \text{ kips} > 28.0 \text{ kips}$ **o.k.**

Strength of the bolted connection—beam web

Because there are no edge bolts, the available bearing and tearout strength of the beam web for all bolts is determined using AISC *Manual* Table 7-4 with s = 3 in.

LRFD	ASD
$\phi r_n = (102 \text{ kip/in.})(0.355 \text{ in.})$ $= 36.2 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (68.3 \text{ kip/in.})(0.355 \text{ in.})$ $= 24.2 \text{ kips/bolt}$

Bolt shear strength is the governing limit state for all bolts at the beam web.

The strength of the bolt group in the beam web is determined by summing the strength of the individual fasteners as follows:

LRFD	ASD
$\phi R_n = (3 \text{ bolts})(24.3 \text{ kips/bolt})$ $= 72.9 \text{ kips} > 42.0 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = (3 \text{ bolts})(16.2 \text{ kips/bolt})$ $= 48.6 \text{ kips} > 28.0 \text{ kips}$ **o.k.**

Shear strength of the web plate

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the plate is determined as follows:

$$
\begin{aligned}
A_{gv} &= lt \\
&= (9 \text{ in.})(\tfrac{3}{8} \text{ in.}) \\
&= 3.38 \text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
R_n &= 0.60F_yA_{gv} \\
&= 0.60(36 \text{ ksi})(3.38 \text{ in.}^2) \\
&= 73.0 \text{ kips}
\end{aligned}
$$

(*Spec*. Eq. J4-3)

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(73.0\text{ kips})$ $= 73.0\text{ kips} > 42.0\text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{73.0\text{ kips}}{1.50}$ $= 48.7\text{ kips} > 28.0\text{ kips}$ **o.k.**

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the plate is determined as follows:

$$\begin{aligned} A_{nv} &= \left[l - n\left(d_h + \tfrac{1}{16}\text{ in.}\right)\right]t \\ &= \left[9\text{ in.} - (3\text{ bolts})\left(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right]\left(\tfrac{3}{8}\text{ in.}\right) \\ &= 2.25\text{ in.}^2 \end{aligned}$$

$$R_n = 0.60F_uA_{nv} \qquad (\textit{Spec}.\text{ Eq. J4-4})$$

$$\begin{aligned} &= 0.60(58\text{ ksi})\left(2.25\text{ in.}^2\right) \\ &= 78.3\text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(78.3\text{ kips})$ $= 58.7\text{ kips} > 42.0\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{78.3\text{ kips}}{2.00}$ $= 39.2\text{ kips} > 28.0\text{ kips}$ **o.k.**

Block shear rupture of the web plate

The nominal strength for the limit state of block shear rupture is given by AISC *Specification* Section J4.3.

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad (\textit{Spec}.\text{ Eq. J4-5})$$

The available block shear rupture strength of the web plate is determined as follows, using AISC *Manual* Tables 9-3a, 9-3b and 9-3c, and AISC *Specification* Equation J4-5, with $n = 3$, $l_{eh} = 2$ in., $l_{ev} = 1\frac{1}{2}$ in., and $U_{bs} = 1.0$.

LRFD	ASD
Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{\phi F_uA_{nt}}{t} = 65.3\text{ kip/in.}$ Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{\phi 0.60F_yA_{gv}}{t} = 121\text{ kip/in.}$	Tension rupture component from AISC *Manual* Table 9-3a: $\dfrac{F_uA_{nt}}{\Omega t} = 43.5\text{ kip/in.}$ Shear yielding component from AISC *Manual* Table 9-3b: $\dfrac{0.60F_yA_{gv}}{\Omega t} = 81.0\text{ kip/in.}$

LRFD	ASD
Shear rupture component from AISC *Manual* Table 9-3c:	Shear rupture component from AISC *Manual* Table 9-3c:
$\dfrac{\phi 0.60 F_u A_{nv}}{t} = 131\text{ kip/in.}$	$\dfrac{0.60 F_u A_{nv}}{\Omega t} = 87.0\text{ kip/in.}$
The design block shear rupture strength is:	The allowable block shear rupture strength is:
$\phi R_n = \phi 0.60 F_u A_{nv} + \phi U_{bs} F_u A_{nt}$ $\leq \phi 0.60 F_y A_{gv} + \phi U_{bs} F_u A_{nt}$ $= \left(\tfrac{3}{8}\text{ in.}\right)\left[131\text{ kip/in.} + (1.0)(65.3\text{ kip/in.})\right]$ $\leq \left(\tfrac{3}{8}\text{ in.}\right)\left[121\text{ kip/in.} + (1.0)(65.3\text{ kip/in.})\right]$ $= 73.6\text{ kips} > 69.9\text{ kips}$	$\dfrac{R_n}{\Omega} = \dfrac{0.60 F_u A_{nv}}{\Omega} + \dfrac{U_{bs} F_u A_{nt}}{\Omega}$ $\leq \dfrac{0.60 F_y A_{gv}}{\Omega} + \dfrac{U_{bs} F_u A_{nt}}{\Omega}$ $= \left(\tfrac{3}{8}\text{ in.}\right)\left[87.0\text{ kip/in.} + (1.0)(43.5\text{ kip/in.})\right]$ $\leq \left(\tfrac{3}{8}\text{ in.}\right)\left[81.0\text{ kip/in.} + (1.0)(43.5\text{ kip/in.})\right]$ $= 48.9\text{ kips} > 46.7\text{ kips}$
Therefore:	Therefore:
$\phi R_n = 69.9\text{ kips} > 42.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 46.7\text{ kips} > 28.0\text{ kips}$ **o.k.**

Weld shear strength of the web plate to the column flange

The available weld strength is determined using AISC *Manual* Equations 8-2a or 8-2b, with the assumption that the weld is in direct shear (the incidental moment in the weld plate due to eccentricity is absorbed by the flange plates).

$D = 4$ (for a ¼-in. fillet weld)

LRFD	ASD
$\phi R_n = (2\text{ welds})(1.392\text{ kip/in.})Dl$ $= (2\text{ welds})(1.392\text{ kip/in.})(4)(9\text{ in.})$ $= 100\text{ kips} > 42.0\text{ kips}$ **o.k.**	$\phi R_n = (2\text{ welds})(0.928\text{ kip/in.})Dl$ $= (2\text{ welds})(0.928\text{ kip/in.})(4)(9\text{ in.})$ $= 66.8\text{ kips} > 28.0\text{ kips}$ **o.k.**

Column flange rupture strength at welds

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the column flange is determined as follows:

$$
\begin{aligned}
A_{nv} &= (2\text{ welds})\,l t_f \\
&= (2\text{ welds})(9\text{ in.})(0.780\text{ in.}) \\
&= 14.0\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
R_n &= 0.60 F_u A_{nv} \\
&= 0.60(65\text{ ksi})\left(14.0\text{ in.}^2\right) \\
&= 546\text{ kips}
\end{aligned}
$$

(*Spec*. Eq. J4-4)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(546\text{ kips})$ $= 410\text{ kips} > 42.0\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{546\text{ kips}}{2.00}$ $= 273\text{ kips} > 28.0\text{ kips}$ **o.k.**

Flange Plate Connection

Flange force

The moment arm between flange forces, d_m, used for verifying the fastener strength is equal to the depth of the beam. This dimension represents the faying surface between the flange of the beam and the tension plate.

LRFD	ASD
$P_{uf} = \dfrac{M_u}{d_m}$ (*Manual* Eq. 12-1a) $= \dfrac{(252\text{ kip-ft})(12\text{ in./ft})}{18.0\text{ in.}}$ $= 168\text{ kips}$	$P_{af} = \dfrac{M_a}{d_m}$ (*Manual* Eq. 12-1b) $= \dfrac{(168\text{ kip-ft})(12\text{ in./ft})}{18.0\text{ in.}}$ $= 112\text{ kips}$

Strength of the bolted connection—flange plate

From the Commentary to AISC *Specification* Section J3.6, the strength of the bolt group is taken as the sum of the strengths of the individual fasteners, which may be taken as the lesser of the fastener shear strength per AISC *Specification* Section J3.6, the bearing strength at the bolt hole per AISC *Specification* Section J3.10, or the tearout strength at the bolt hole per AISC *Specification* Section J3.10.

From AISC *Manual* Table 7-1, the available shear strength per bolt for ⅞-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) is:

LRFD	ASD
$\phi r_n = 24.3\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 16.2\text{ kips/bolt}$

The available bearing strength of the plate per bolt is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration:

$$r_n = 2.4dtF_u \qquad \text{(}\textit{Spec.}\text{ Eq. J3-6a)}$$
$$= 2.4(\tfrac{7}{8}\text{ in.})(\tfrac{3}{4}\text{ in.})(58\text{ ksi})$$
$$= 91.4\text{ kips/bolt}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(91.4\text{ kips/bolt})$ $= 68.6\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{91.4\text{ kips/bolt}}{2.00}$ $= 45.7\text{ kips/bolt}$

The available tearout strength of the plate at the interior bolts is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration.

$$\begin{aligned} l_c &= s - d_h \\ &= 3 \text{ in.} - {}^{15}\!/_{16} \text{ in.} \\ &= 2.06 \text{ in.} \end{aligned}$$

$$\begin{aligned} r_n &= 1.2 l_c t F_u \\ &= 1.2(2.06 \text{ in.})(\tfrac{3}{4} \text{ in.})(58 \text{ ksi}) \\ &= 108 \text{ kips/bolt} \end{aligned}$$

(*Spec*. Eq. J3-6c)

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(108 \text{ kips/bolt})$ $= 81.0 \text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{108 \text{ kips/bolt}}{2.00}$ $= 54.0 \text{ kips/bolt}$

Therefore, bolt shear controls over bearing or tearout at interior bolts.

The available tearout strength of the plate at the edge bolt is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration.

$$\begin{aligned} l_c &= l_{ev} - 0.5(d_h) \\ &= 1\tfrac{1}{2} \text{ in.} - 0.5({}^{15}\!/_{16} \text{ in.}) \\ &= 1.03 \text{ in.} \end{aligned}$$

$$\begin{aligned} r_n &= 1.2 l_c t F_u \\ &= 1.2(1.03 \text{ in.})(\tfrac{3}{4} \text{ in.})(58 \text{ ksi}) \\ &= 53.8 \text{ kips/bolt} \end{aligned}$$

(*Spec*. Eq. J3-6c)

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(53.8 \text{ kips/bolt})$ $= 40.4 \text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{53.8 \text{ kips/bolt}}{2.00}$ $= 26.9 \text{ kips/bolt}$

Therefore, bolt shear controls over bearing or tearout at edge bolts.

The strength of the bolt group in the beam web is determined by summing the strength of the individual fasteners as follows:

LRFD	ASD
$\phi R_n = (8 \text{ bolts})(24.3 \text{ kips/bolt})$ $= 194 \text{ kips} > 168 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = (8 \text{ bolts})(16.2 \text{ kips/bolt})$ $= 130 \text{ kips} > 112 \text{ kips}$ **o.k.**

Strength of the bolted connection—beam flange

The available bearing strength of the flange per bolt is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration:

$$\begin{aligned} r_n &= 2.4dtF_u \\ &= 2.4(\tfrac{7}{8}\text{ in.})(0.570\text{ in.})(65\text{ ksi}) \\ &= 77.8\text{ kips/bolt} \end{aligned}$$ (*Spec*. Eq. J3-6a)

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(77.8\text{ kips/bolt})$ $= 58.4\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{77.8\text{ kips/bolt}}{2.00}$ $= 38.9\text{ kips/bolt}$

The available tearout strength of the flange at the interior bolts is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration.

$$\begin{aligned} l_c &= s - d_h \\ &= 3\text{ in.} - \tfrac{15}{16}\text{ in.} \\ &= 2.06\text{ in.} \end{aligned}$$

$$\begin{aligned} r_n &= 1.2l_ctF_u \\ &= 1.2(2.06\text{ in.})(0.570\text{ in.})(65\text{ ksi}) \\ &= 91.6\text{ kips/bolt} \end{aligned}$$ (*Spec*. Eq. J3-6c)

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(91.6\text{ kips/bolt})$ $= 68.7\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{91.6\text{ kips/bolt}}{2.00}$ $= 45.8\text{ kips/bolt}$

Therefore, bolt shear controls over bearing or tearout at interior bolts.

The available tearout strength of the flange at the edge bolt is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration.

$$\begin{aligned} l_c &= l_{ev} - 0.5(d_h) \\ &= 1\tfrac{1}{2}\text{ in.} - 0.5(\tfrac{15}{16}\text{ in.}) \\ &= 1.03\text{ in.} \end{aligned}$$

$$\begin{aligned} r_n &= 1.2l_ctF_u \\ &= 1.2(1.03\text{ in.})(\tfrac{3}{4}\text{ in.})(58\text{ ksi}) \\ &= 53.8\text{ kips/bolt} \end{aligned}$$ (*Spec*. Eq. J3-6c)

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(53.8\text{ kips/bolt})$ $= 40.4\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{53.8\text{ kips/bolt}}{2.00}$ $= 26.9\text{ kips/bolt}$

Therefore, bolt shear controls over bearing or tearout at edge bolts.

The strength of the bolt group in the flange is determined by summing the strength of the individual fasteners as follows:

LRFD	ASD
$\phi R_n = (8\text{ bolts})(24.3\text{ kips/bolt})$ $= 194\text{ kips} > 168\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = (8\text{ bolts})(16.2\text{ kips/bolt})$ $= 130\text{ kips} > 112\text{ kips}$ **o.k.**

Tensile strength of the flange plate

The moment arm between flange forces, d_m, used for verifying the tensile strength of the flange plate is equal to the depth of the beam plus one plate thickness. This represents the distance between the centerlines of the flange plates at the top and bottom of the beam. From AISC *Manual* Equation 12-1a or 12-1b, the flange force is:

LRFD	ASD
$P_{uf} = \dfrac{M_u}{d_m}$ $= \dfrac{(252\text{ kip-ft})(12\text{ in./ft})}{18.0\text{ in.} + \frac{3}{4}\text{ in.}}$ $= 161\text{ kips}$	$P_{af} = \dfrac{M_a}{d_m}$ $= \dfrac{(168\text{ kip-ft})(12\text{ in./ft})}{18.0\text{ in.} + \frac{3}{4}\text{ in.}}$ $= 108\text{ kips}$

From AISC *Specification* Section J4.1(a), the available tensile yield strength of the flange plate is determined as follows:

$$\begin{aligned} A_g &= bt \\ &= (7\text{ in.})(\tfrac{3}{4}\text{ in.}) \\ &= 5.25\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= F_y A_g \\ &= (36\text{ ksi})(5.25\text{ in.}^2) \\ &= 189\text{ kips} \end{aligned}$$

(*Spec*. Eq. J4-1)

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(189\text{ kips})$ $= 170\text{ kips} > 161\text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{189\text{ kips}}{1.67}$ $= 113\text{ kips} > 108\text{ kips}$ **o.k.**

From AISC *Specification* Section J4.1(b), the available tensile rupture strength of the flange plate is determined as follows:

$$\begin{aligned} A_n &= \left[b - n\left(d_h + \tfrac{1}{16}\text{-in.}\right)\right]t \\ &= \left[7\text{ in.} - (2\text{ bolts})\left(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\right](\tfrac{3}{4}\text{ in.}) \\ &= 3.75\text{ in.}^2 \end{aligned}$$

Table D3.1, Case 1, applies in this case because the tension load is transmitted directly to the cross-sectional element by fasteners; therefore, U = 1.0.

$$\begin{aligned} A_e &= A_n U \qquad\qquad (\textit{Spec}.\text{ Eq. D3-1}) \\ &= \left(3.75\text{ in.}^2\right)(1.0) \\ &= 3.75\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= F_u A_e \qquad\qquad (\textit{Spec}.\text{ Eq. J4-2}) \\ &= (58\text{ ksi})\left(3.75\text{ in.}^2\right) \\ &= 218\text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75(218\text{ kips})$ $= 164\text{ kips} > 161\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{218\text{ kips}}{2.00}$ $= 109\text{ kips} > 108\text{ kips}$ **o.k.**

Flange plate block shear rupture

There are three cases for which block shear rupture of the flange plate must be checked. Case 1, as shown in Figure II.B-1-2(a), involves the tearout of the two blocks outside the two rows of bolt holes in the flange plate; for this case l_{eh} = 1½ in. and l_{ev} = 1½ in. Case 2, as shown in Figure II.B-1-2(b), involves the tearout of the block between the two rows of the holes in the flange plate. AISC *Manual* Tables 9-3a, 9-3b, and 9-3c may be adapted for this calculation by considering the 4 in. width to be comprised of two, 2-in.-wide blocks, where l_{eh} = 2 in. and l_{ev} = 1½ in. Case 1 is more critical than the Case 2 because l_{eh} is smaller. Case 3, as shown in Figure II.B-1-2(c), involves a shear failure through one row of bolts and a tensile failure through the two bolts closest to the column. Therefore, Case 1 and Case 3 will be verified.

Flange plate block shear rupture—Case 1

The nominal strength for the limit state of block shear rupture is given by AISC *Specification* Section J4.3.

$$R_n = 0.60F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.60F_y A_{gv} + U_{bs} F_u A_{nt} \qquad (\textit{Spec}.\text{ Eq. J4-5})$$

The available block shear rupture strength of the flange plate is determined as follows, using AISC *Manual* Tables 9-3a, 9-3b and 9-3c, and AISC *Specification* Equation J4-5, with n = 4, l_{eh} = l_{ev} = 1½ in., and U_{bs} = 1.0.

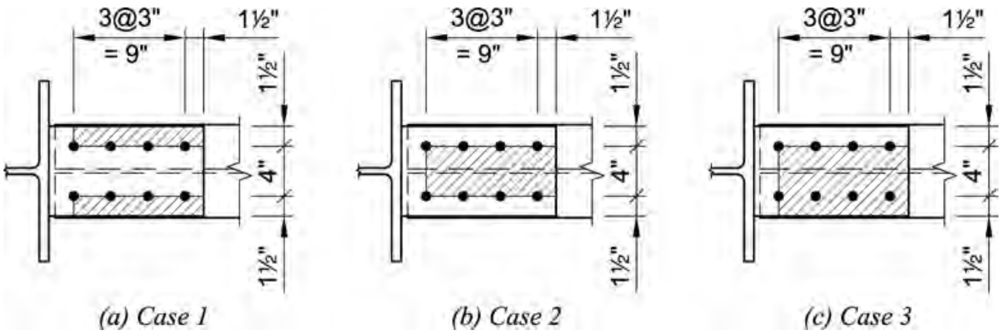
LRFD	ASD
Tension rupture component from AISC *Manual* Table 9-3a:	Tension rupture component from AISC *Manual* Table 9-3a:
$\dfrac{\phi F_u A_{nt}}{t} = 43.5\text{ kip/in.}$	$\dfrac{F_u A_{nt}}{\Omega t} = 29.0\text{ kip/in.}$
Shear yielding component from AISC *Manual* Table 9-3b:	Shear yielding component from AISC *Manual* Table 9-3b:
$\dfrac{\phi 0.60 F_y A_{gv}}{t} = 170\text{ kip/in.}$	$\dfrac{0.60 F_y A_{gv}}{\Omega t} = 113\text{ kip/in.}$

LRFD	ASD
Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{\phi 0.60F_uA_{nv}}{t} = 183\text{ kip/in.}$ The design block shear rupture strength is: $\phi R_n = \phi 0.60F_uA_{nv} + \phi U_{bs}F_uA_{nt}$ $\le \phi 0.60F_yA_{gv} + \phi U_{bs}F_uA_{nt}$ $= (2\text{ planes})(\text{¾ in.})\left[\begin{array}{l}183\text{ kip/in.}\\ \quad +1.0(43.5\text{ kip/in.})\end{array}\right]$ $\le (2\text{ planes})(\text{¾ in.})\left[\begin{array}{l}170\text{ kip/in.}\\ \quad +1.0(43.5\text{ kip/in.})\end{array}\right]$ $= 340\text{ kips} > 320\text{ kips}$ Therefore: $\phi R_n = 320\text{ kips} > 161\text{ kips}$ **o.k.**	Shear rupture component from AISC *Manual* Table 9-3c: $\dfrac{0.60F_uA_{nv}}{\Omega t} = 122\text{ kip/in.}$ The allowable block shear rupture strength is: $\dfrac{R_n}{\Omega} = \dfrac{0.60F_uA_{nv}}{\Omega} + \dfrac{U_{bs}F_uA_{nt}}{\Omega}$ $\le \dfrac{0.60F_yA_{gv}}{\Omega} + \dfrac{U_{bs}F_uA_{nt}}{\Omega}$ $= (2\text{ planes})(\text{¾ in.})\left[\begin{array}{l}122\text{ kip/in.}\\ \quad +1.0(29.0\text{ kip/in.})\end{array}\right]$ $\le (2\text{ planes})(\text{¾ in.})\left[\begin{array}{l}113\text{ kip/in.}\\ \quad +1.0(29.0\text{ kip/in.})\end{array}\right]$ $= 227\text{ kips} > 213\text{ kips}$ Therefore: $\dfrac{R_n}{\Omega} = 213\text{ kips} > 108\text{ kips}$ **o.k.**

Flange plate block shear rupture—Case 3

Because AISC *Manual* Table 9-3a does not include a large enough edge distance, the nominal strength for the limit state of block shear rupture is calculated by directly applying the provisions of AISC *Specification* Section J4.3.

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad (\textit{Spec}.\ \text{Eq. J4-5})$$



(a) Case 1 (b) Case 2 (c) Case 3

Fig. II.B-1-2. Three cases for block shear rupture.

where

$$
\begin{aligned}
A_{gv} &= \left[(n-1)s + l_{ev}\right]t \\
&= \left[(4-1)(3\text{ in.}) + 1\tfrac{1}{2}\text{ in.}\right](\tfrac{3}{4}\text{ in.}) \\
&= 7.88\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
A_{nv} &= A_{gv} - (n-0.5)(d_h + \tfrac{1}{16}\text{ in.})t \\
&= 7.88\text{ in.}^2 - (4-0.5)(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})(\tfrac{3}{4}\text{ in.}) \\
&= 5.26\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
A_{nt} &= \left[gage + l_{eh} - 1.5(d_h + \tfrac{1}{16}\text{ in.})\right]t \\
&= \left[4\text{ in.} + 1\tfrac{1}{2}\text{ in.} - 1.5(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})\right](\tfrac{3}{4}\text{ in.}) \\
&= 3.00\text{ in.}^2
\end{aligned}
$$

$$U_{bs} = 1.0$$

and

$$
\begin{aligned}
R_n &= 0.60(58\text{ ksi})(5.26\text{in.}^2) + 1.0(58\text{ ksi})(3.00\text{ in.}^2) \le 0.60(36\text{ ksi})(7.88\text{ in.}^2) + 1.0(58\text{ ksi})(3.00\text{ in.}^2) \\
&= 357\text{ kips} > 344\text{ kips}
\end{aligned}
$$

Therefore:

$$R_n = 344\text{ kips}$$

From AISC *Specification* Section J4.3, the available strength for the limit state of block shear rupture on the plate is:

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75(344\text{ kips})$ $= 258\text{ kips} > 161\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{344\text{ kips}}{2.00}$ $= 172\text{ kips} > 108\text{ kips}$ **o.k.**

Beam flange block shear rupture

The nominal strength for the limit state of block shear rupture is given by AISC *Specification* Section J4.3.

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad (\textit{Spec}.\text{ Eq. J4-5})$$

The available block shear rupture strength of the beam flange involves the tearout of the two blocks outside the two rows of bolt holes in the flanges. Conservatively use the flange forces that were found for the fastener checks. From AISC *Manual* Tables 9-3a, 9-3b, and 9-3c, and AISC *Specification* Equation J4-5, with n = 4, l_{eh} = 1¾ in., l_{ev} = 1¼ in. (reduced ¼ in. to account for beam underrun), and U_{bs} = 1.0:

LRFD	ASD
Tension rupture component from AISC *Manual* Table 9-3a:	Tension rupture component from AISC *Manual* Table 9-3a:
$\dfrac{\phi F_u A_{nt}}{t} = 60.9 \text{ kip/in.}$	$\dfrac{F_u A_{nt}}{\Omega t} = 40.6 \text{ kip/in.}$
Shear yielding component from AISC *Manual* Table 9-3b:	Shear yielding component from AISC *Manual* Table 9-3b:
$\dfrac{\phi 0.60 F_y A_{gv}}{t} = 231 \text{ kip/in.}$	$\dfrac{0.60 F_y A_{gv}}{\Omega t} = 154 \text{ kip/in.}$
Shear rupture component from AISC *Manual* Table 9-3c:	Shear rupture component from AISC *Manual* Table 9-3c:
$\dfrac{\phi 0.60 F_u A_{nv}}{t} = 197 \text{ kip/in.}$	$\dfrac{0.60 F_u A_{nv}}{\Omega t} = 132 \text{ kip/in.}$
The design block shear rupture strength is:	The allowable block shear rupture strength is:
$\phi R_n = \phi 0.60 F_u A_{nv} + \phi U_{bs} F_u A_{nt}$ $\leq \phi 0.60 F_y A_{gv} + \phi U_{bs} F_u A_{nt}$ $= (2 \text{ planes})(0.570 \text{ in.})\left[\begin{array}{l}197 \text{ kip/in.}\\ +(1.0)(60.9 \text{ kip/in.})\end{array}\right]$ $\leq (2 \text{ planes})(0.570 \text{ in.})\left[\begin{array}{l}231 \text{ kip/in.}\\ +(1.0)(60.9 \text{ kip/in.})\end{array}\right]$ $= 294 \text{ kips} < 333 \text{ kips}$	$\dfrac{R_n}{\Omega} = \dfrac{0.60 F_u A_{nv}}{\Omega} + \dfrac{U_{bs} F_u A_{nt}}{\Omega}$ $\leq \dfrac{0.60 F_y A_{gv}}{\Omega} + \dfrac{U_{bs} F_u A_{nt}}{\Omega}$ $= (2 \text{ planes})(0.570 \text{ in.})\left[\begin{array}{l}132 \text{ kip/in.}\\ +(1.0)(40.6 \text{ kip/in.})\end{array}\right]$ $\leq (2 \text{ planes})(0.570 \text{ in.})\left[\begin{array}{l}154 \text{ kip/in.}\\ +(1.0)(40.6 \text{ kip/in.})\end{array}\right]$ $= 197 \text{ kips} < 222 \text{ kips}$
Therefore:	Therefore:
$\phi R_n = 294 \text{ kips} > 168 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 197 \text{ kips} > 112 \text{ kips}$ **o.k.**

Fillet weld to supporting column flange

The applied load is perpendicular to the weld length (θ = 90°); therefore, the directional strength factor is determined from AISC *Specification* Equation J2-5. This increase factor due to directional strength is incorporated into the weld strength calculation.

$$\begin{aligned} 1.0 + 0.50 \sin^{1.5} \theta &= 1.0 + 0.50 \sin^{1.5} (90°) \\ &= 1.50 \end{aligned}$$

The required fillet weld size is determined using AISC *Manual* Equations 8-2a or 8-2b as follows:

LRFD	ASD
$D_{min} = \dfrac{P_{uf}}{(2\text{ welds})(1.50)(1.392\text{ kip/in.})l}$ $= \dfrac{161\text{ kips}}{(2\text{ welds})(1.50)(1.392\text{ kip/in.})(7\text{ in.})}$ $= 5.51$ Use a ⅜-in. fillet weld on both sides of the flange plate.	$D_{min} = \dfrac{P_{af}}{(2\text{ welds})(1.50)(0.928\text{ kip/in.})l}$ $= \dfrac{108\text{ kips}}{(2\text{ welds})(1.50)(0.928\text{ kip/in.})(7\text{ in.})}$ $= 5.54$ Use a ⅜-in. fillet weld on both sides of the flange plate.

Compression Flange Plate and Connection

From AISC *Specification* Section J4.4, the available strength of the flange plate in compression is determined as follows:

K = 0.65, from AISC *Specification* Commentary Table C-A-7.1
L = 3.00 in. (the distance between adjacent bolt holes)

$$r = \sqrt{\frac{I}{A}}$$

$$= \sqrt{\frac{(7\text{ in.})(\text{¾ in.})^3/12}{(7\text{ in.})(\text{¾ in.})}}$$

$$= 0.217\text{ in.}$$

$$\frac{L_c}{r} = \frac{KL}{r}$$

$$= \frac{0.65(3.00\text{ in.})}{0.217\text{ in.}}$$

$$= 8.99$$

Since $L_c/r \le 25$:

$$P_n = F_y A_g \qquad (\textit{Spec}.\text{ Eq. J4-6})$$

$$= (36\text{ ksi})(7\text{ in.})(\text{¾ in.})$$

$$= 189\text{ kips}$$

LRFD	ASD
$\phi = 0.90$ $\phi P_n = 0.90(189\text{ kips})$ $= 170\text{ kips} > 161\text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{P_n}{\Omega} = \dfrac{189\text{ kips}}{1.67}$ $= 113\text{ kips} > 108\text{ kips}$ **o.k.**

The compression flange plate will be identical to the tension flange plate; a ¾-in.×7-in. plate with eight bolts in two rows of four bolts on a 4-in. gage and ⅜-in. fillet welds to the supporting column flange.

Note: The bolt bearing and shear checks are the same as for the tension flange plate and have found to be adequate in prior calculations. Tension due to load reversal must also be considered in the design of the fillet weld to the supporting column flange. The result is the same as previously calculated for the top flange connection plate.

Flange Local Bending of Column

From AISC *Specification* Section J10.1, the available strength of the column for the limit state of flange local bending is determined as follows:

$$\begin{aligned}0.15b_f &= 0.15(14.6\text{ in.})\\ &= 2.19\text{ in.}\end{aligned}$$

The length of loading (i.e., plate width) is 7 in., which is greater than $0.15b_f$. Thus, flange local bending needs to be checked.

Assume the concentrated force to be resisted is applied at a distance from the column end greater than $10t_f$.

$$\begin{aligned}10t_f &= 10(0.780\text{ in.})\\ &= 7.80\text{ in.}\end{aligned}$$

$$\begin{aligned}R_n &= 6.25F_{yf}t_f^{\,2} \qquad \text{(\textit{Spec}. Eq. J10-1)}\\ &= 6.25(50\text{ ksi})(0.780\text{ in.})^2\\ &= 190\text{ kips}\end{aligned}$$

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(190\text{ kips})$ $= 171\text{ kips} > 161\text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{190\text{ kips}}{1.67}$ $= 114\text{ kips} > 108\text{ kips}$ **o.k.**

Web Local Yielding of Column

Assume the concentrated force to be resisted is applied at a distance from the column end that is greater than the depth of the column. The available strength of the column for the limit state of web local yielding is determined from AISC *Manual* Table 9-4 and AISC *Manual* Equation 9-47a or 9-47b, with $l_b = t =$ ¾ in.

LRFD	ASD
$\phi R_1 = 83.7\text{ kips}$ $\phi R_2 = 24.3\text{ kip/in.}$ $\phi R_n = 2(\phi R_1) + l_b(\phi R_2)$ $= 2(83.7\text{ kips}) + (3/4\text{ in.})(24.3\text{ kip/in.})$ $= 186\text{ kips} > 161\text{ kips}$ **o.k.**	$R_1/\Omega = 55.8\text{ kips}$ $R_2/\Omega = 16.2\text{ kip/in.}$ $\dfrac{R_n}{\Omega} = 2(R_1/\Omega) + l_b(R_2/\Omega)$ $= 2(55.8\text{ kips}) + (3/4\text{ in.})(16.2\text{ kip/in.})$ $= 124\text{ kips} > 108\text{ kips}$ **o.k.**

Web Local Crippling

Assume the concentrated force to be resisted is applied at a distance from the column end that is greater than or equal to one-half of the column depth. The available strength of the column for the limit state of web local crippling is determined from AISC *Manual* Table 9-4 and AISC *Manual* Equation 9-50a or 9-50b, with $l_b = t =$ ¾ in.

LRFD	ASD
$\phi R_3 = 108 \text{ kips}$ $\phi R_4 = 11.2 \text{ kip/in.}$ $\phi R_n = 2\left[\phi R_3 + l_b\left(\phi R_4\right)\right]$ $= 2\left[108 \text{ kips} + \left(3/4 \text{ in.}\right)\left(11.2 \text{ kip/in.}\right)\right]$ $= 233 \text{ kips} > 161 \text{ kips}$ **o.k.**	$R_3/\Omega = 71.8 \text{ kips}$ $R_4/\Omega = 7.44 \text{ kip/in.}$ $\dfrac{R_n}{\Omega} = 2\left[R_3/\Omega + l_b\left(R_4/\Omega\right)\right]$ $= 2\left[71.8 \text{ kips} + \left(3/4 \text{ in.}\right)\left(7.44 \text{ kip/in.}\right)\right]$ $= 155 \text{ kips} > 108 \text{ kips}$ **o.k.**

Note: Web compression buckling (AISC *Specification* Section J10.5) must be checked if another beam is framed into the opposite side of the column at this location.

Web panel zone shear (AISC *Specification* Section J10.6) should also be checked for this column.

For further information, see AISC Design Guide 13 *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications* (Carter, 1999).

EXAMPLE II.B-2 WELDED FLANGE-PLATED FR MOMENT CONNECTION (BEAM-TO-COLUMN FLANGE)

Given:

Verify a welded flange-plated FR moment connection between an ASTM A992 W18×50 beam and an ASTM A992 W14×99 column flange, as shown in Figure II.B-2-1, to transfer the following beam end reactions:

Vertical shear:
V_D = 7 kips
V_L = 21 kips

Strong-axis moment:
M_D = 42 kip-ft
M_L = 126 kip-ft

Use 70-ksi electrodes. The flange plates are ASTM A36 material. Assume the top flange of the beam is in the tension condition due to moment.

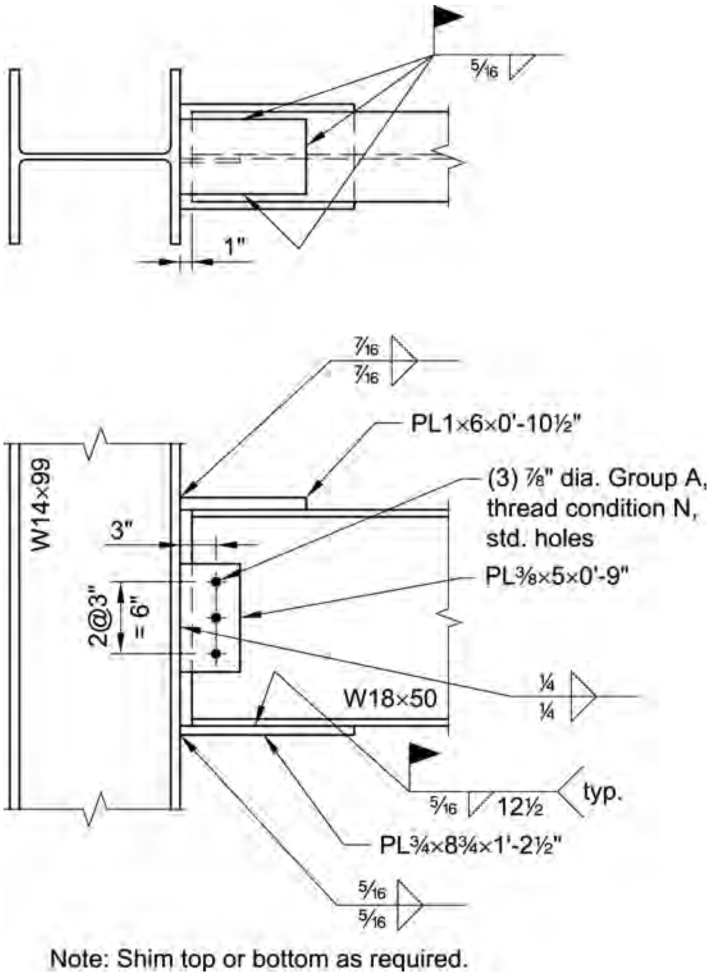


Fig. II.B-2-1. Connection geometry for Example II.B-2.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Beam and column
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Plates
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W18×50
d = 18.0 in.
b_f = 7.50 in.
t_f = 0.570 in.
t_w = 0.355 in.
Z_x = 101 in.^{3}

Column
W14×99
d = 14.2 in.
b_f = 14.6 in.
t_f = 0.780 in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(7\text{ kips}) + 1.6(21\text{ kips})$ $= 42.0\text{ kips}$ $M_u = 1.2(42\text{ kip-ft}) + 1.6(126\text{ kip-ft})$ $= 252\text{ kip-ft}$	$R_a = 7\text{ kips} + 21\text{ kips}$ $= 28.0\text{ kips}$ $M_a = 42\text{ kip-ft} + 126\text{ kip-ft}$ $= 168\text{ kip-ft}$

Single-Plate Web Connection

The single-plate web connection is verified in Example II.B-1.

Note: By inspection, the available effective fastener strength and shear yielding strengths of the beam web are adequate. The beam web is nearly as thick as the web plate and of a higher strength material. Shear rupture and block shear rupture are not limit states for the beam web.

Tension Flange Plate and Connection

Tensile yielding of the flange plate

The top flange plate is specified as a PL1 in. × 6 in. × 0 ft 10½ in. The top beam flange width is b_f = 7.50 in. This provides a shelf dimension of ¾-in. on both sides of the plate for welding.

The moment arm between flange plate forces, d_m, used for verifying the plate strength is equal to the depth of the beam plus one-half the thickness of each of the flange plates. This represents the distance between the centerlines of the flange plates at the top and bottom of the beam.

$$\begin{aligned} d_m &= 18.0 \text{ in.} + \frac{\tfrac{3}{4} \text{ in.}}{2} + \frac{1 \text{ in.}}{2} \\ &= 18.9 \text{ in.} \end{aligned}$$

From AISC *Manual* Equation 12-1a or 12-1b, the flange force is:

LRFD	ASD
$P_{uf} = \dfrac{M_u}{d_m}$ $= \dfrac{(252 \text{ kip-ft})(12 \text{ in./ft})}{18.9 \text{ in.}}$ $= 160 \text{ kips}$	$P_{af} = \dfrac{M_a}{d_m}$ $= \dfrac{(168 \text{ kip-ft})(12 \text{ in./ft})}{18.9 \text{ in.}}$ $= 107 \text{ kips}$

From AISC *Specification* Section J4.1(a), the available tensile yield strength of the flange plate is determined as follows:

$$\begin{aligned} R_n &= F_y A_g \\ &= (36 \text{ ksi})(6 \text{ in.})(1 \text{ in.}) \\ &= 216 \text{ kips} \end{aligned}$$

(*Spec*. Eq. J4-1)

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(216 \text{ kips})$ $= 194 \text{ kips} > 160 \text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{216 \text{ kips}}{1.67}$ $= 129 \text{ kips} > 107 \text{ kips}$ **o.k.**

Fillet weld strength for top flange plate to beam flange

The moment arm between flange forces, d_m, used for verifying the fillet weld strength is equal to the depth of the beam. This dimension represents the faying surface between the flange of the beam and the tension plate. From AISC *Manual* Equation 12-1a or 12-1b, the flange force is:

LRFD	ASD
$P_{uf} = \dfrac{M_u}{d_m}$ $= \dfrac{(252 \text{ kip-ft})(12 \text{ in./ft})}{18.0 \text{ in.}}$ $= 168 \text{ kips}$	$P_{af} = \dfrac{M_a}{d_m}$ $= \dfrac{(168 \text{ kip-ft})(12 \text{ in./ft})}{18.0 \text{ in.}}$ $= 112 \text{ kips}$

A $\tfrac{5}{16}$-in. fillet weld is specified (D = 5). The available strength may be calculated using the provisions from AISC *Specification* Section J2.4(b)(2). The available shear strength of the fillet weld may be calculated using AISC *Specification* Table J2.5.

The length of the longitudinally loaded welds is determined taking into consideration a ¼-in. tolerance to account for possible beam underrun and a weld termination equal to the weld size.

$$\begin{aligned} l &= 10\tfrac{1}{2}\text{ in.} - 1\text{ in. (setback)} - \tfrac{1}{4}\text{ in. (underrun)} - \tfrac{5}{16}\text{ in. (weld termination)} \\ &= 8.94\text{ in.} \end{aligned}$$

$$\begin{aligned} R_{nwl} &= 0.60F_{EXX}\left(\frac{\sqrt{2}}{2}\right)\left(\frac{D}{16}\right)l \\ &= 0.60(70\text{ ksi})\left(\frac{\sqrt{2}}{2}\right)\left(\frac{5}{16}\right)(8.94\text{ in.})(2\text{ welds}) \\ &= 166\text{ kips} \end{aligned}$$

$$\begin{aligned} R_{nwt} &= 0.60F_{EXX}\left(\frac{\sqrt{2}}{2}\right)\left(\frac{D}{16}\right)l \\ &= 0.60(70\text{ ksi})\left(\frac{\sqrt{2}}{2}\right)\left(\frac{5}{16}\right)(6\text{ in.}) \\ &= 55.7\text{ kips} \end{aligned}$$

The combined strength of the fillet weld group may be taken as the larger of the following:

$$\begin{aligned} R_n &= R_{nwl} + R_{nwt} \\ &= 166\text{ kips} + 55.7\text{ kips} \\ &= 222\text{ kips} \end{aligned}$$

(*Spec*. Eq. J2-6a)

$$\begin{aligned} R_n &= 0.85R_{nwl} + 1.5R_{nwt} \\ &= 0.85(166\text{ kips}) + 1.5(55.7\text{ kips}) \\ &= 225\text{ kips} \end{aligned}$$

(*Spec*. Eq. J2-6b)

Therefore:

$R_n = 225\text{ kips}$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(225\text{ kips})$ $= 169\text{ kips} > 168\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{225\text{ kips}}{2.00}$ $= 113\text{ kips} > 112\text{ kips}$ **o.k.**

Connecting elements rupture strength at top flange welds

At the top flange connection, the minimum base metal thickness to match the shear rupture strength of the weld is determined as follows:

$$\begin{aligned} t_{min} &= \frac{3.09D}{F_u} \\ &= \frac{3.09(5)}{65\text{ ksi}} \\ &= 0.238\text{ in.} < 0.570\text{ in. beam flange} \quad \textbf{o.k.} \end{aligned}$$

(*Manual* Eq. 9-2)

$$t_{min} = \frac{3.09D}{F_u} \qquad \text{(\textit{Manual} Eq. 9-2)}$$

$$= \frac{3.09(5)}{58 \text{ ksi}}$$

$$= 0.266 \text{ in.} < 1.00 \text{ in. top flange plate} \quad \textbf{o.k.}$$

Fillet weld at top flange plate to column flange

The applied load is perpendicular to the weld length (θ = 90°), therefore the directional strength factor is determined from AISC *Specification* Equation J2-5. This increase factor due to directional strength is incorporated into the weld strength calculation.

$$1.0 + 0.50\sin^{1.5}\theta = 1.0 + 0.50\sin^{1.5}(90°)$$

$$= 1.50$$

The available strength of fillet welds is determined using AISC *Manual* Equation 8-2a or 8-2b, as follows:

LRFD	ASD
$D_{min} = \dfrac{P_{uf}}{(2 \text{ welds})(1.50)(1.392 \text{ kip/in.})l}$ $= \dfrac{160 \text{ kips}}{(2 \text{ welds})(1.50)(1.392 \text{ kip/in.})(6 \text{ in.})}$ $= 6.39$ Use a 7⁄16-in. fillet weld on both sides of the plate.	$D_{min} = \dfrac{P_{af}}{(2 \text{ welds})(1.50)(0.928 \text{ kip/in.})l}$ $= \dfrac{107 \text{ kips}}{(2 \text{ welds})(1.50)(0.928 \text{ kip/in.})(6 \text{ in.})}$ $= 6.41$ Use a 7⁄16-in. fillet weld on both sides of the plate.

Compression Flange Plate and Connection

Flange plate compressive strength

The bottom flange plate is specified as a PL¾×8¾×1'-2½". The bottom flange width is b_f = 7.50 in. This provides a shelf dimension of ⅝-in. on both sides of the plate for welding.

Assume an underrun dimension of ¼-in. and an additional ½-in. to the start of the weld.

K = 0.65 from AISC *Specification* Commentary Table C-A-7.1
L = 1.75 in.

$$r = \sqrt{\frac{I}{A}}$$

$$= \sqrt{\frac{(8\tfrac{3}{4} \text{ in.})(\tfrac{3}{4} \text{ in.})^3/12}{(8\tfrac{3}{4} \text{ in.})(\tfrac{3}{4} \text{ in.})}}$$

$$= 0.217 \text{ in.}$$

$$\frac{L_c}{r} = \frac{KL}{r}$$

$$= \frac{0.65(1.75 \text{ in.})}{0.217 \text{ in.}}$$

$$= 5.24 < 25$$

Since $L_c/r \le 25$:

$$P_n = F_y A_g \qquad \text{(\textit{Spec}. Eq. J4-6)}$$
$$= (36\text{ ksi})(8\tfrac{3}{4}\text{ in.})(\tfrac{3}{4}\text{ in.})$$
$$= 236\text{ kips}$$

LRFD	ASD
$\phi = 0.90$ $\phi P_n = 0.90(236\text{ kips})$ $= 212\text{ kips} > 160\text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{P_n}{\Omega} = \dfrac{236\text{ kips}}{1.67}$ $= 141\text{ kips} > 107\text{ kips}$ **o.k.**

Fillet weld strength for bottom flange plate to beam flange

The required weld length is determined using AISC *Manual* Equation 8-2a or 8-2b, as follows:

LRFD	ASD
$l_{min} = \dfrac{P_{fu}}{(2\text{ welds})(1.392\text{ kip/in.})D}$ $= \dfrac{168\text{ kips}}{(2\text{ welds})(1.392\text{ kip/in.})(5)}$ $= 12.1\text{ in.}$ Use 12½-in.-long 5⁄16-in. fillet welds.	$l_{min} = \dfrac{P_{fa}}{(2\text{ welds})(0.928\text{ kip/in.})D}$ $= \dfrac{112\text{ kips}}{(2\text{ welds})(0.928\text{ kip/in.})(5)}$ $= 12.1\text{ in.}$ Use 12½-in.-long 5⁄16-in. fillet welds.

Beam bottom flange rupture strength at welds

$$A_{nv} = (2\text{ welds})t_f l$$
$$= (2\text{ welds})(0.570\text{ in.})(12\tfrac{1}{2}\text{ in.})$$
$$= 14.3\text{ in.}^3$$

$$R_n = 0.60 F_u A_{nv} \qquad \text{(\textit{Spec}. Eq. J4-4)}$$
$$= 0.60(65\text{ ksi})(14.3\text{ in.}^2)$$
$$= 558\text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(558\text{ kips})$ $= 419\text{ kips} > 168\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{558\text{ kips}}{2.00}$ $= 279\text{ kips} > 112\text{ kips}$ **o.k.**

Fillet weld at bottom flange plate to column flange

The applied load is perpendicular to the weld length (θ = 90°) therefore the directional strength factor is determined from AISC *Specification* Equation J2-5. This increase factor due to directional strength is incorporated into the weld strength calculation.

$$\begin{aligned}1.0+0.50\sin^{1.5}\theta &= 1.0+0.50\sin^{1.5}(90°)\\ &= 1.50\end{aligned}$$

The available strength of fillet welds is determined using AISC *Manual* Equation 8-2a or 8-2b as follows:

LRFD	ASD
$D_{min} = \dfrac{P_{uf}}{(2\text{ welds})(1.50)(1.392\text{ kip/in.})l}$ $= \dfrac{160\text{ kips}}{(2\text{ welds})(1.50)(1.392\text{ kip/in.})(8\tfrac{3}{4}\text{ in.})}$ $= 4.38$ sixteenths Use 5⁄16-in. fillet welds.	$D_{min} = \dfrac{P_{af}}{(2\text{ welds})(1.50)(0.928\text{ kip/in.})l}$ $= \dfrac{107\text{ kips}}{(2\text{ welds})(1.50)(0.928\text{ kip/in.})(8\tfrac{3}{4}\text{ in.})}$ $= 4.39$ sixteenths Use 5⁄16-in. fillet welds.

See Example II.B-1 for checks of the column under concentrated forces. For further information, see AISC Design Guide 13 *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications*. (Carter, 1999).

Conclusion

The connection is found to be adequate as given for the applied loads.

EXAMPLE II.B-3 DIRECTLY WELDED FLANGE FR MOMENT CONNECTION (BEAM-TO-COLUMN FLANGE)

Given:

Verify a directly welded flange FR moment connection between an ASTM A992 W18×50 beam and an ASTM A992 W14×99 column flange, as shown in Figure II.B-3-1, to transfer the following beam end reactions:

Vertical shear:
V_D = 7 kips
V_L = 21 kips

Strong-axis moment:
M_D = 42 kip-ft
M_L = 126 kip-ft

Use 70-ksi electrodes. Check the column for stiffening requirements.

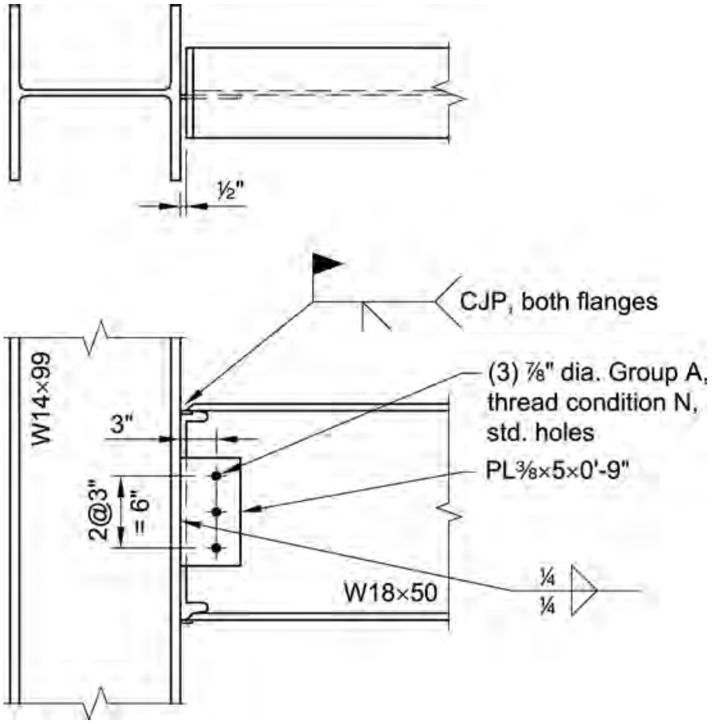


Fig. II.B-3-1. Connection geometry for Example II.B-3.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Beam and column
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Plate
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From ASCE/SEI 7, Chapter 2 , the required strength is:

LRFD	ASD
$R_u = 1.2(7\text{ kips}) + 1.6(21\text{ kips})$ $= 42.0\text{ kips}$ $M_u = 1.2(42\text{ kip-ft}) + 1.6(126\text{ kip-ft})$ $= 252\text{ kip-ft}$	$R_a = 7\text{ kips} + 21\text{ kips}$ $= 28.0\text{ kips}$ $M_a = 42\text{ kip-ft} + 126\text{ kip-ft}$ $= 168\text{ kip-ft}$

The single-plate web connection is verified in Example II.B-1.

Note: By inspection, the available effective fastener strength and shear yielding strengths of the beam web are adequate. The beam web is nearly as thick as the web plate, and of a higher strength material. Shear rupture and block shear rupture are not limit states for the beam web.

Weld of Beam Flange to Column

A complete-joint-penetration groove weld will transfer the entire flange force in tension and compression. It is assumed that the beam is adequate for the applied moment and will carry the tension and compression forces through the flanges.

See Example II.B-1 for checks of the column under concentrated forces. For further information, see AISC Design Guide 13 *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications*. (Carter, 1999).

Conclusion

The connection is found to be adequate as given for the applied loads.

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CHAPTER IIB DESIGN EXAMPLE REFERENCES

Carter, C.J. (1999), *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications*, Design Guide 13, AISC, Chicago, IL.

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Chapter IIC
Bracing and Truss Connections

The design of bracing and truss connections is covered in Part 13 of the AISC *Steel Construction Manual*.

EXAMPLE II.C-1 TRUSS SUPPORT CONNECTION

Given:

The truss end connection shown in Figure II.C-1-1 is designed for the required forces shown in Figure II.C-1-2. Verify the following:

a. The connection requirements between the gusset and the column
b. The required gusset size and the weld requirements connecting the diagonal to the gusset

Use 70-ksi electrodes. The top chord and column are ASTM A992 material. The diagonal member, gusset plate and clip angles are ASTM A36 material.

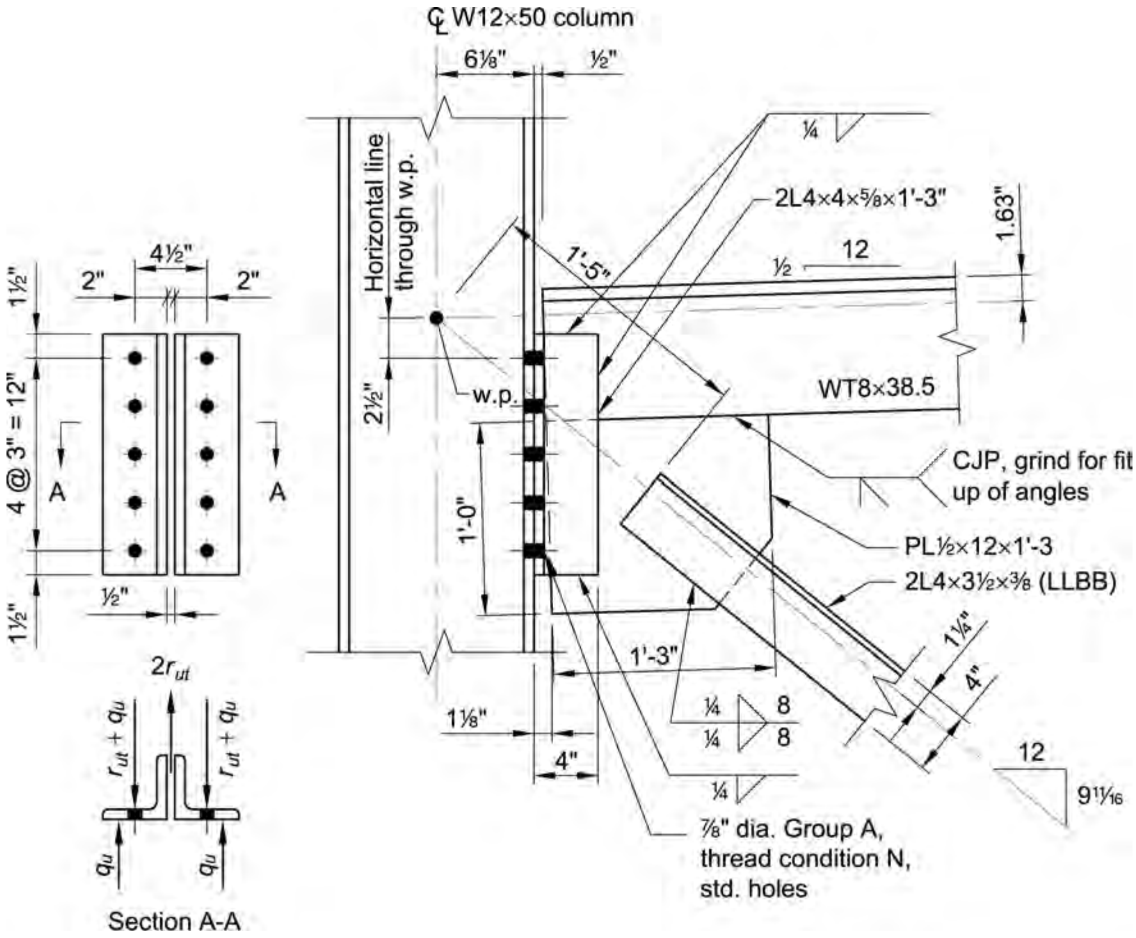

Fig. II.C-1-1. Truss support connection.

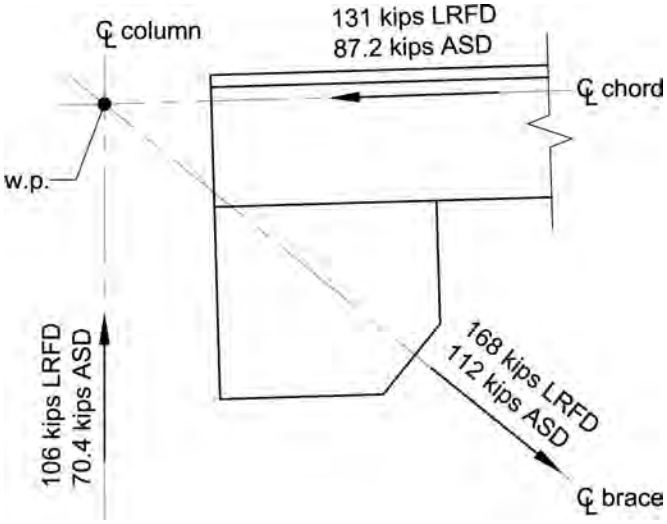

Fig. II.C-1-2. Required forces in members.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Column and top chord
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Diagonal, gusset plate and clip angles
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Tables 1-1, 1-7, 1-8 and 1-15, the geometric properties are as follows:

Top chord
WT8×38.5
d = 8.26 in.
t_w = 0.455 in.
$\bar{y}$ = 1.63 in.

Column
W12×50
d = 12.2 in.
t_f = 0.640 in.
b_f = 8.08 in.
t_w = 0.370 in.

Diagonal brace
2L4×3½×⅜
t = ⅜ in.
A = 5.36 in.^{2}
$\bar{x}$ = 0.947 in. for single angle

Clip angles
2L4×4×⅝
t = ⅝ in.

From Figure II.C-1-2 the required strengths are:

LRFD	ASD
Brace axial load: $R_u = 168$ kips	Brace axial load: $R_a = 112$ kips
Truss end reaction: $R_u = 106$ kips	Truss end reaction: $R_a = 70.4$ kips
Top chord axial load: $R_u = 131$ kips	Top chord axial load: $R_a = 87.2$ kips

Weld Connecting the Diagonal to the Gusset Plate

Note: AISC *Specification* Section J1.7, requiring that the center of gravity of the weld group coincide with the center of gravity of the member, does not apply to end connections of statically loaded single-angle, double-angle and similar members.

From AISC *Specification* Table J2.4, the minimum fillet weld size for ⅜-in. angles attached to a ½-in.-thick gusset plate is:

$$w_{min} = \tfrac{3}{16} \text{ in.}$$

For ¼-in. fillet welds (D = 4), the required weld length is determined from AISC *Manual* Equations 8-2a or 8-2b, as follows:

LRFD	ASD
$l_{req} = \dfrac{R_u}{(4 \text{ welds})(1.392 \text{ kip/in.})(D)}$ $= \dfrac{168 \text{ kips}}{(4 \text{ welds})(1.392 \text{ kip/in.})(4)}$ $= 7.54$ in.	$l_{req} = \dfrac{R_a}{(4 \text{ welds})(0.928 \text{ kip/in.})(D)}$ $= \dfrac{112 \text{ kips}}{(4 \text{ welds})(0.928 \text{ kip/in.})(4)}$ $= 7.54$ in.

Use an 8-in.-long ¼-in. fillet weld at the heel and toe of each angle.

Gusset Shear Rupture at Brace Welds

The minimum plate thickness to match the shear rupture strength of the welds is determined as follows:

$$t_{min} = \frac{6.19D}{F_u} \qquad (\textit{Manual} \text{ Eq. 9-3})$$

$$= \frac{6.19(4)}{58 \text{ ksi}}$$

$$= 0.427$$

Try a ½-in.-thick gusset plate.

Tensile Strength of the Brace

From AISC *Specification* Section D2, the available tensile yielding strength of the brace is determined as follows:

$$\begin{aligned} P_n &= F_y A_g \\ &= (36 \text{ ksi})(5.36 \text{ in.}^2) \\ &= 193 \text{ kips} \end{aligned}$$ (*Spec*. Eq. D2-1)

LRFD	ASD
$\phi_t = 0.90$ $\phi_t P_n = 0.90(193 \text{ kips})$ $= 174 \text{ kips} > 168 \text{ kips}$ **o.k.**	$\Omega_t = 1.67$ $\dfrac{P_n}{\Omega_t} = \dfrac{193 \text{ kips}}{1.67}$ $= 116 \text{ kips} > 112 \text{ kips}$ **o.k.**

From AISC *Specification* Section D2, the available tensile rupture strength of the brace is determined as follows:

$$\begin{aligned} A_n &= A_g \\ &= 5.36 \text{ in.}^2 \end{aligned}$$

The shear lag factor, U, is determined from AISC *Specification* Table D3.1, Case 4:

$$\begin{aligned} U &= \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right) \\ &= \frac{3(8 \text{ in.})^2}{3(8 \text{ in.})^2 + (4 \text{ in.})^2}\left(1 - \frac{0.947 \text{ in.}}{8 \text{ in.}}\right) \\ &= 0.814 \end{aligned}$$

$$\begin{aligned} A_e &= A_n U \\ &= (5.36 \text{ in.}^2)(0.814) \\ &= 4.36 \text{ in.}^2 \end{aligned}$$ (*Spec*. Eq. D3-1)

$$\begin{aligned} P_n &= F_u A_e \\ &= (58 \text{ ksi})(4.36 \text{ in.}^2) \\ &= 253 \text{ kips} \end{aligned}$$ (*Spec*. Eq. D2-2)

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(253 \text{ kips})$ $= 190 \text{ kips} > 168 \text{ kips}$ **o.k.**	$\Omega_t = 2.00$ $\dfrac{P_n}{\Omega_t} = \dfrac{253 \text{ kips}}{2.00}$ $= 127 \text{ kips} > 112 \text{ kips}$ **o.k.**

Use a ½-in.-thick gusset plate. With the brace-to-gusset welds determined, a gusset plate layout as shown in Figure II.C-1-1 can be made.

Strength of the Bolted Connection—Angles

From the Commentary to AISC *Specification* Section J3.6, the strength of the bolt group is taken as the sum of the individual strengths of the individual fasteners, which may be taken as the lesser of the fastener shear strength per AISC *Specification* Section J3.6, the bearing strength at the bolt hole per AISC *Specification* Section J3.10, or the tearout strength at the bolt hole per AISC *Specification* Section J3.10.

The number of ⅞-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) required for shear only is determined as follows:

LRFD	ASD
From AISC *Manual* Table 7-1, the available bolt shear strength is: $\phi r_n = 24.3\text{ kips/bolt}$ $n_{min} = \dfrac{R_u}{(2\text{ bolts/row})\phi r_n}$ $= \dfrac{106\text{ kips}}{(2\text{ bolts/row})(24.3\text{ kips/bolt})}$ $= 2.18\text{ rows}$	From AISC *Manual* Table 7-1, the available bolt shear strength is: $\dfrac{r_n}{\Omega} = 16.2\text{ kips/bolt}$ $n_{min} = \dfrac{R_a}{(2\text{ bolts/row})(r_n/\Omega)}$ $= \dfrac{70.4\text{ kips}}{(2\text{ bolts/row})(16.2\text{ kips/bolt})}$ $= 2.17\text{ rows}$

Use 2L4×4×⅝ clip angles with five pairs of bolts. Note the number of rows of bolts is increased to "square off" the gusset plate.

The available bearing strength of the angles per bolt is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration:

$$\begin{aligned} r_n &= 2.4dtF_u \qquad (\textit{Spec}.\text{ Eq. J3-6a}) \\ &= 2.4(\tfrac{7}{8}\text{ in.})(\tfrac{5}{8}\text{ in.})(58\text{ ksi}) \\ &= 76.1\text{ kips/bolt} \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(76.1\text{ kips/bolt})$ $= 57.1\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{76.1\text{ kips/bolt}}{2.00}$ $= 38.1\text{ kips/bolt}$

The available tearout strength of the angles at edge bolts is determined from AISC *Specification* Section J3.10, with d_h = $^{15}/_{16}$ in. for ⅞-in.-diameter bolts from AISC *Specification* Table J3.3, assuming deformation at service load is a design consideration:

$$\begin{aligned} l_c &= l_e - 0.5d_h \\ &= 1\tfrac{1}{2}\text{ in.} - 0.5(\tfrac{15}{16}\text{ in.}) \\ &= 1.03\text{ in.} \end{aligned}$$

$$\begin{aligned} r_n &= 1.2 l_c t F_u && \text{(}\textit{Spec}\text{. Eq. J3-6c)} \\ &= 1.2(1.03\text{ in.})(\tfrac{5}{8}\text{ in.})(58\text{ ksi}) \\ &= 44.8\text{ kips/bolt} \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(44.8\text{ kips/bolt})$ $= 33.6\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{44.8\text{ kips/bolt}}{2.00}$ $= 22.4\text{ kips/bolt}$

Therefore, bolt shear controls over bolt bearing or tearout at the edge bolts.

The available tearout strength of the angles at interior bolts is determined from AISC *Specification* Section J3.10, assuming deformation at service load is a design consideration:

$$\begin{aligned} l_c &= s - d_h \\ &= 3\text{ in.} - \tfrac{15}{16}\text{ in.} \\ &= 2.06\text{ in.} \end{aligned}$$

$$\begin{aligned} r_n &= 1.2 l_c t F_u && \text{(}\textit{Spec}\text{. Eq. J3-6c)} \\ &= 1.2(2.06\text{ in.})(\tfrac{5}{8}\text{ in.})(58\text{ ksi}) \\ &= 89.6\text{ kips/bolt} \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(89.6\text{ kips/bolt})$ $= 67.2\text{ kips/bolt}$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{89.6\text{ kips/bolt}}{2.00}$ $= 44.8\text{ kips/bolt}$

Therefore, bolt shear controls over bolt bearing or tearout at the interior bolts.

Because bolt shear controls for all the bolts, the connection is acceptable based on previous calculations.

Bolt Shear and Tension Interaction—Bolts Connecting Clip Angles to Column

The eccentric moment about the work point (w.p.) at the faying surface (face of column flange) is determined using an eccentricity equal to half of the column depth.

$$\begin{aligned} e &= \frac{d}{2} \\ &= \frac{12.2\text{ in.}}{2} \\ &= 6.10\text{ in.} \end{aligned}$$

The eccentricity normal to the plane of the faying surface is accounted for using the Case II approach in AISC *Manual* Part 7 for eccentrically loaded bolt groups.

$n' = 4$ bolts (number of bolts above the neutral axis)
$d_m = 9.00$ in. (moment arm between resultant tension force and resultant compressive force)

The maximum tensile force per bolt is determined using AISC *Manual* Equations 7-14a or 7-14b, as follows:

LRFD	ASD
$r_{ut} = \dfrac{P_u e}{n' d_m}$ $= \dfrac{(106\text{ kips})(6.10\text{ in.})}{(4\text{ bolts})(9.00\text{ in.})}$ $= 18.0\text{ kips/bolt}$	$r_{at} = \dfrac{P_a e}{n' d_m}$ $= \dfrac{(70.4\text{ kips})(6.10\text{ in.})}{(4\text{ bolts})(9.00\text{ in.})}$ $= 11.9\text{ kips/bolt}$

The required shear stress per bolt is determined as follows:

$A_b = 0.601\text{ in.}^2$ (from AISC *Manual* Table 7-1)
$n = 10$ bolts

LRFD	ASD
$f_{rv} = \dfrac{R_u}{nA_b}$ $= \dfrac{106\text{ kips}}{(10\text{ bolts})(0.601\text{ in.}^2)}$ $= 17.6\text{ ksi}$	$f_{rv} = \dfrac{R_a}{nA_b}$ $= \dfrac{70.4\text{ kips}}{(10\text{ bolts})(0.601\text{ in.}^2)}$ $= 11.7\text{ ksi}$

The nominal tensile strength modified to include the effects of shear stress is determined from AISC *Specification* Section J3.7 as follows. From AISC *Specification* Table J3.2:

$F_{nt} = 90$ ksi
$F_{nv} = 54$ ksi

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$F'_{nt} = 1.3F_{nt} - \dfrac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt}$ (*Spec*. Eq. J3-3a) $= 1.3(90\text{ ksi}) - \dfrac{90\text{ ksi}}{0.75(54\text{ ksi})}(17.6\text{ ksi}) \le 90\text{ ksi}$ $= 77.9\text{ ksi} < 90\text{ ksi}$	$F'_{nt} = 1.3F_{nt} - \dfrac{\Omega F_{nt}}{F_{nv}} f_{rv} \le F_{nt}$ (*Spec*. Eq. J3-3b) $= 1.3(90\text{ ksi}) - \dfrac{2.00(90\text{ ksi})}{54\text{ ksi}}(11.7\text{ ksi}) \le 90\text{ ksi}$ $= 78.0\text{ ksi} < 90\text{ ksi}$
Therefore:	Therefore:
$F'_{nt} = 77.9\text{ ksi}$	$F'_{nt} = 78.0\text{ ksi}$
$B_c = \phi F'_{nt} A_b$ (from *Spec*. Eq. J3-2) $= 0.75(77.9\text{ ksi})(0.601\text{ in.}^2)$ $= 35.1\text{ kips/bolt} > 18.0\text{ kips/bolt}$ **o.k.**	$B_c = \dfrac{F'_{nt}}{\Omega} A_b$ (from *Spec*. Eq. J3-2) $= \dfrac{78.0\text{ ksi}}{2.00}(0.601\text{ in.}^2)$ $= 23.4\text{ kips/bolt} > 11.9\text{ kips/bolt}$ **o.k.**

Prying Action on Clip Angles

From AISC *Manual* Part 9, the available tensile strength of the bolts in the outstanding angle legs taking prying action into account is determined as follows:

$$\begin{aligned} a &= \frac{b_f - gage}{2} \\ &= \frac{8.08\text{ in.} - 4\tfrac{1}{2}\text{ in.}}{2} \\ &= 1.79\text{ in.} \end{aligned}$$

Note: a is calculated based on the column flange width in this case because it is less than the double angle width.

$$\begin{aligned} b &= \frac{gage - t_p - t}{2} \\ &= \frac{4\tfrac{1}{2}\text{ in.} - \tfrac{1}{2}\text{ in.} - \tfrac{5}{8}\text{ in.}}{2} \\ &= 1.69\text{ in.} \end{aligned}$$

Note: 1¼ in. entering and tightening clearance from AISC *Manual* Table 7-15 is accommodated and the column fillet toe is cleared.

$$\begin{aligned} a' &= \left(a + \frac{d_b}{2}\right) \le \left(1.25b + \frac{d_b}{2}\right) \\ &= 1.79\text{ in.} + \frac{\tfrac{7}{8}\text{ in.}}{2} \le 1.25(1.69\text{ in.}) + \frac{\tfrac{7}{8}\text{ in.}}{2} \\ &= 2.23\text{ in.} < 2.55\text{ in.} \quad \textbf{o.k.} \end{aligned}$$

(*Manual* Eq. 9-23)

$$\begin{aligned} b' &= \left(b - \frac{d_b}{2}\right) \\ &= 1.69\text{ in.} - \frac{\tfrac{7}{8}\text{ in.}}{2} \\ &= 1.25\text{ in.} \end{aligned}$$

(*Manual* Eq. 9-18)

$$\begin{aligned} \rho &= \frac{b'}{a'} \\ &= \frac{1.25\text{ in.}}{2.23\text{ in.}} \\ &= 0.561 \end{aligned}$$

(*Manual* Eq. 9-22)

$$\begin{aligned} p &= \frac{l}{n} \\ &= \frac{15\text{ in.}}{5} \\ &= 3.00\text{ in.} \end{aligned}$$

Check

$$\begin{aligned} &p \le s \\ &3.00\text{ in.} = 3.00\text{ in.} \quad \textbf{o.k.} \end{aligned}$$

$$\delta = 1 - \frac{d'}{p} \qquad \text{(\textit{Manual} Eq. 9-20)}$$

$$= 1 - \frac{^{15}\!/_{16}\text{ in.}}{3.00\text{ in.}}$$

$$= 0.688$$

The angle thickness required to develop the available strength of the bolt with no prying action is determined as follows:

LRFD	ASD
$\phi = 0.90$ $B_c = 35.1$ kips/bolt (calculated previously) $t_c = \sqrt{\dfrac{4B_c b'}{\phi p F_u}}$ (*Manual* Eq. 9-26a) $= \sqrt{\dfrac{4(35.1\text{ kips/bolt})(1.25\text{ in.})}{0.90(3.00\text{ in.})(58\text{ ksi})}}$ $= 1.06$ in.	$\Omega = 1.67$ $B_c = 23.4$ kips/bolt (calculated previously) $t_c = \sqrt{\dfrac{\Omega 4 B_c b'}{p F_u}}$ (*Manual* Eq. 9-26b) $= \sqrt{\dfrac{1.67(4)(23.4\text{ kips/bolt})(1.25\text{ in.})}{(3.00\text{ in.})(58\text{ ksi})}}$ $= 1.06$ in.

$$\alpha' = \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] \qquad \text{(\textit{Manual} Eq. 9-28)}$$

$$= \frac{1}{0.688(1+0.561)}\left[\left(\frac{1.06\text{ in.}}{^5\!/_8\text{ in.}}\right)^2 - 1\right]$$

$$= 1.75$$

Because $\alpha' > 1$, the angles have insufficient strength to develop the bolt strength, therefore:

$$Q = \left(\frac{t}{t_c}\right)^2 (1+\delta)$$

$$= \left(\frac{^5\!/_8\text{ in.}}{1.06\text{ in.}}\right)^2 (1+0.688)$$

$$= 0.587$$

The available tensile strength per bolt, taking prying action into account, is determined using AISC *Manual* Equation 9-27, as follows:

LRFD	ASD
$\phi r_n = B_c Q$ $= (35.1\text{ kips/bolt})(0.587)$ $= 20.6\text{ kips/bolt} > 18.0\text{ kips/bolt}$ **o.k.**	$\dfrac{r_n}{\Omega} = B_c Q$ $= (23.4\text{ kips/bolt})(0.587)$ $= 13.7\text{ kips/bolt} > 11.9\text{ kips/bolt}$ **o.k.**

Shear Strength of Clip Angles

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the angles is determined as follows:

$$
\begin{aligned}
A_{gv} &= (2 \text{ angles})lt \\
&= (2 \text{ angles})(15 \text{ in.})(\tfrac{5}{8} \text{ in.}) \\
&= 18.8 \text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
R_n &= 0.60F_yA_{gv} \qquad \text{(\textit{Spec.} Eq. J4-3)} \\
&= 0.60(36 \text{ ksi})(18.8 \text{ in.}^2) \\
&= 406 \text{ kips}
\end{aligned}
$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(406 \text{ kips})$ $= 406 \text{ kips} > 106 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{406 \text{ kips}}{1.50}$ $= 271 \text{ kips} > 70.4 \text{ kips}$ **o.k.**

From AISC *Specification* Section J4.2, the available shear rupture strength of the angles is determined using the net area determined in accordance with AISC *Specification* Section B4.3b.

$$
\begin{aligned}
A_{nv} &= (2 \text{ angles})\left[l - n(d_h + \tfrac{1}{16} \text{ in.})\right]t \\
&= (2 \text{ angles})\left[15 \text{ in.} - 5(\tfrac{15}{16} \text{ in.} + \tfrac{1}{16} \text{ in.})\right](\tfrac{5}{8} \text{ in.}) \\
&= 12.5 \text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
R_n &= 0.60F_uA_{nv} \qquad \text{(\textit{Spec.} Eq. J4-4)} \\
&= 0.60(58 \text{ ksi})(12.5 \text{ in.}^2) \\
&= 435 \text{ kips}
\end{aligned}
$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(435 \text{ kips})$ $= 326 \text{ kips} > 106 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{435 \text{ kips}}{2.00}$ $= 218 \text{ kips} > 70.4 \text{ kips}$ **o.k.**

Block Shear Rupture of Clip Angles

The available strength for the limit state of block shear rupture of the angles is determined as follows.

$$
R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad \text{(\textit{Spec.} Eq. J4-5)}
$$

where

$$
\begin{aligned}
A_{gv} &= (2 \text{ angles})(l - l_{ev})t \\
&= (2 \text{ angles})(15 \text{ in.} - 1\tfrac{1}{2} \text{ in.})(\tfrac{5}{8} \text{ in.}) \\
&= 16.9 \text{ in.}^2
\end{aligned}
$$

$$\begin{aligned}A_{nv} &= A_{gv} - (2\text{ angles})(n-0.5)(d_h + \tfrac{1}{16}\text{ in.})t\\ &= 16.9\text{ in.}^2 - (2\text{ angles})(5-0.5)(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})(\tfrac{5}{8}\text{ in.})\\ &= 11.3\text{ in.}^2\end{aligned}$$

$$\begin{aligned}A_{nt} &= (2\text{ angles})\left[l_{eh} - 0.5(d_h + \tfrac{1}{16}\text{ in.})\right]t\\ &= (2\text{ angles})\left[2\text{ in.} - 0.5(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})\right](\tfrac{5}{8}\text{ in.})\\ &= 1.88\text{ in.}^2\end{aligned}$$

$$U_{bs} = 1.0$$

and

$$\begin{aligned}R_n &= 0.60(58\text{ ksi})(11.3\text{ in.}^2) + 1.0(58\text{ ksi})(1.88\text{ in.}^2) \le 0.60(36\text{ ksi})(16.9\text{ in.}^2) + 1.0(58\text{ ksi})(1.88\text{ in.}^2)\\ &= 502\text{ kips} > 474\text{ kips}\end{aligned}$$

Therefore:

$$R_n = 474\text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(474\text{ kips})$ $= 356\text{ kips} > 106\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{474\text{ kips}}{2.00}$ $= 237\text{ kips} > 70.4\text{ kips}$ **o.k.**

Prying Action on Column Flange

Using the same procedure as shown previously for the clip angles, the available tensile strength of the bolts, taking prying action into account, is:

LRFD	ASD
$T_c = 18.7\text{ kips} > 18.0\text{ kips}$ **o.k.**	$T_c = 12.4\text{ kips} > 11.9\text{ kips}$ **o.k.**

Strength of the Bolted Connection—Column Flange

By inspection, the applicable limit states will control for the angles; therefore, the column flange is acceptable.

Clip Angle-to-Gusset Plate Connection

With a top chord slope of ½ in 12, the horizontal welds are unequal length as shown in Figure II.C-1-3. The average horizontal length is used in the following calculations.

$$l = 15\text{ in.}$$

$$\begin{aligned}kl &= \frac{3\tfrac{3}{8}\text{ in.} + 2\tfrac{3}{4}\text{ in.}}{2}\\ &= 3.06\end{aligned}$$

$$\begin{aligned} k &= \frac{kl}{l} \\ &= \frac{3.06\text{ in.}}{15\text{ in.}} \\ &= 0.204 \end{aligned}$$

$$\begin{aligned} xl &= \frac{(kl)^2}{l+2(kl)} \\ &= \frac{(3.06\text{ in.})^2}{15\text{ in.}+2(3.06\text{ in.})} \\ &= 0.443\text{ in.} \end{aligned}$$

$$\begin{aligned} al + xl &= 6.10\text{ in.} + 4.00\text{ in.} \\ &= 10.1\text{ in.} \end{aligned}$$

$$\begin{aligned} a &= \frac{10.1\text{ in.} - xl}{l} \\ &= \frac{10.1\text{ in.} - 0.443\text{ in.}}{15\text{ in.}} \\ &= 0.644 \end{aligned}$$

By interpolating AISC *Manual* Table 8-8 with Angle = 0°:

$$C = 1.50$$

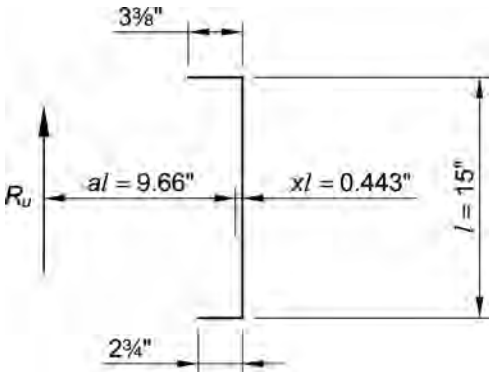

Fig. II.C-1-3. Weld group geometry.

From AISC *Manual* Table 8-8, the minimum required weld size is determined as follows:

LRFD	ASD
$\phi = 0.75$ $D_{min} = \dfrac{R_u}{(2\text{ welds})\phi C C_1 l}$ $= \dfrac{106\text{ kips}}{(2\text{ welds})(0.75)(1.50)(1.0)(15\text{ in.})}$ $= 3.14$ Use ¼-in. fillet welds.	$\Omega = 2.00$ $D_{min} = \dfrac{\Omega R_a}{(2\text{ welds}) C C_1 l}$ $= \dfrac{2.00(70.4\text{ kips})}{2(1.50)(1.0)(15\text{ in.})}$ $= 3.13$ Use ¼-in. fillet welds.

From AISC *Specification* Table J2.4, the minimum weld size for ⅝-in. clip angles attached to a ½-in.-thick gusset plate is:

$$w_{min} = \tfrac{3}{16}\text{ in.} < \tfrac{1}{4}\text{ in.} \quad \textbf{o.k.}$$

Note: Using the average of the horizontal weld lengths provides a reasonable solution when the horizontal welds are close in length. A conservative solution can be determined by using the smaller of the horizontal weld lengths as effective for both horizontal welds. For this example, use kl = 2¾ in., C = 1.43, and D_{min} = 3.29 sixteenths.

Tensile Yielding of Gusset Plate on the Whitmore Section

The gusset plate thickness should match or slightly exceed that of the chord stem. This requirement is satisfied by the ½-in. plate previously selected.

From AISC *Manual* Figure 9-1, the width of the Whitmore section is:

$$\begin{aligned} l_w &= 4.00\text{ in.} + 2(8.00\text{ in.})\tan 30° \\ &= 13.2\text{ in.} \end{aligned}$$

From AISC *Specification* Section J4.1(a), the available tensile yielding strength of the gusset plate is determined as follows:

$$\begin{aligned} A_g &= l_w t \\ &= (13.2\text{ in.})(\tfrac{1}{2}\text{ in.}) \\ &= 6.60\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= F_y A_g \\ &= (36\text{ ksi})(6.60\text{ in.}^2) \\ &= 238\text{ kips} \end{aligned} \qquad (\textit{Spec.}\text{ Eq. J4-1})$$

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(238\text{ kips})$ $= 214\text{ kips} > 168\text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{238\text{ kips}}{1.67}$ $= 143\text{ kips} > 112\text{ kips}$ **o.k.**

Gusset Plate-to-Tee Stem Weld

The interface forces are:

LRFD	ASD
Horizontal shear between gusset and WT:	Horizontal shear between gusset and WT:
$H_{ub} = 131\text{ kips} - (4\text{ bolts})(18.0\text{ kips/bolt})$ $= 59.0\text{ kips}$	$H_{ab} = 87.2\text{ kips} - (4\text{ bolts})(11.9\text{ kips/bolt})$ $= 39.6\text{ kips}$
Vertical tension between gusset and WT:	Vertical tension between gusset and WT:
$V_{ub} = (106\text{ kips})\left(\dfrac{4\text{ bolts}}{10\text{ bolts}}\right)$ $= 42.4\text{ kips}$	$V_{ab} = (70.4\text{ kips})\left(\dfrac{4\text{ bolts}}{10\text{ bolts}}\right)$ $= 28.2\text{ kips}$
Compression between WT and column:	Compression between WT and column:
$C_{ub} = (4\text{ bolts})(18.0\text{ kips/bolt})$ $= 72.0\text{ kips}$	$C_{ab} = (4\text{ bolts})(11.9\text{ kips/bolt})$ $= 47.6\text{ kips}$
Summing moments about the face of the column at the workline of the top chord:	Summing moments about the face of the column at the workline of the top chord:
$M_{ub} = C_{ub}(2\tfrac{1}{2}\text{ in.} + 1.50\text{ in.})$ $+ H_{ub}(d - \bar{y})$ $- V_{ub}\left(\dfrac{gusset\ width}{2} + setback\right)$ $= (72.0\text{ kips})(2\tfrac{1}{2}\text{ in.} + 1.50\text{ in.})$ $+ (59.0\text{ kips})(8.26\text{ in.} - 1.63\text{ in.})$ $- (42.4\text{ kips})\left(\dfrac{15.0\text{ in.}}{2} + \tfrac{1}{2}\text{ in.}\right)$ $= 340\text{ kip-in.}$	$M_{ab} = C_{ab}(2\tfrac{1}{2}\text{ in.} + 1.50\text{ in.})$ $+ H_{ab}(d - \bar{y})$ $- V_{ab}\left(\dfrac{gusset\ width}{2} + setback\right)$ $= (47.6\text{ kips})(2\tfrac{1}{2}\text{ in.} + 1.50\text{ in.})$ $+ (39.6\text{ kips})(8.26\text{ in.} - 1.63\text{ in.})$ $- (28.2\text{ kips})\left(\dfrac{15.0\text{ in.}}{2} + \tfrac{1}{2}\text{ in.}\right)$ $= 227\text{ kip-in.}$

A CJP weld should be used along the interface between the gusset plate and the tee stem. The weld should be ground smooth under the clip angles.

The gusset plate width depends upon the diagonal connection. From a scaled layout, the gusset plate must be 1 ft 3 in. wide.

The gusset plate depth depends upon the connection angles. From a scaled layout, the gusset plate must extend 12 in. below the tee stem.

Use a PL½×12 in.×1 ft 3 in.

Conclusion

The connection is found to be adequate as given for the applied loads.

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EXAMPLE II.C-2 TRUSS SUPPORT CONNECTION

Given:

Verify the truss support connections, as shown in Figure II.C-2-1, at the following joints:

A. Joint L_1
B. Joint U_1

Use 70-ksi electrodes, ASTM A36 plate, ASTM A992 bottom and top chords, and ASTM A36 double angles.

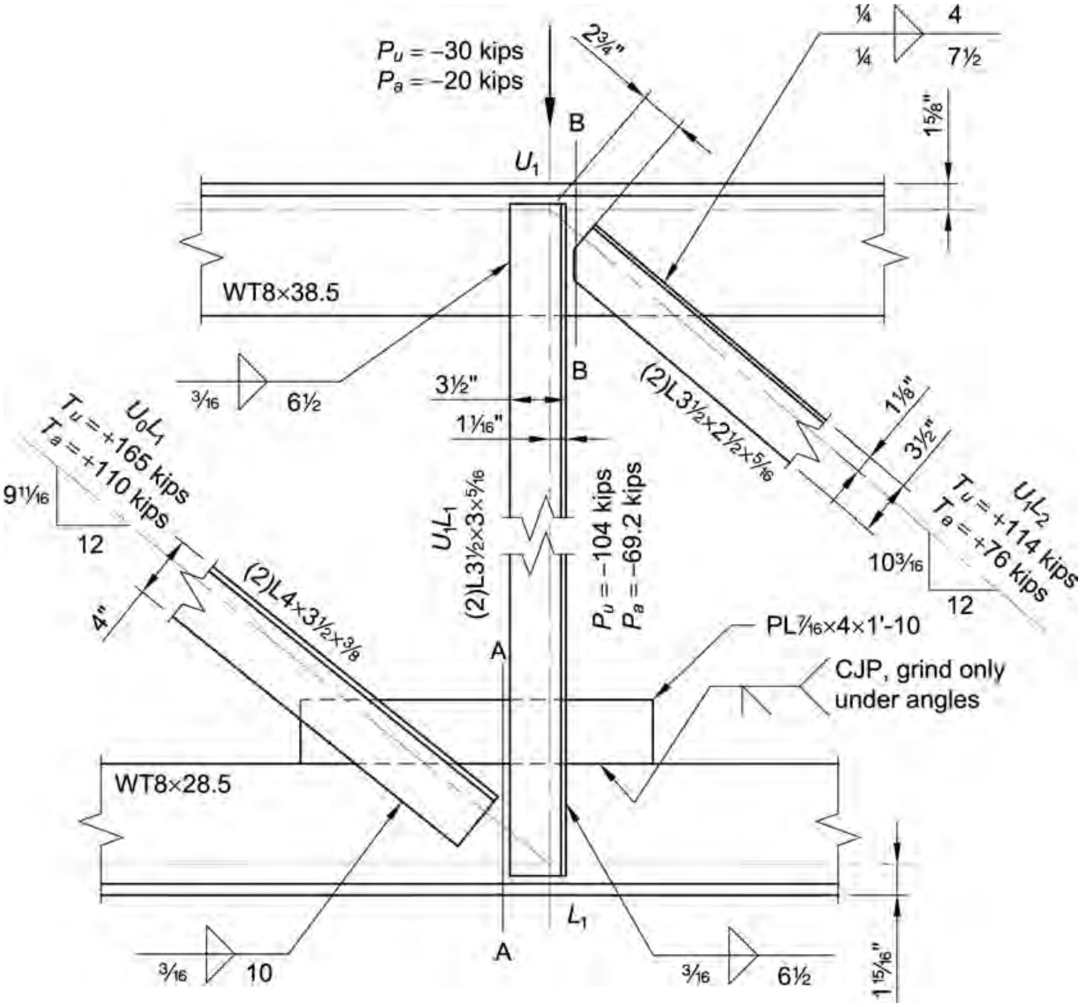

Fig. II.C-2-1. Connection geometry for Example II.C-2.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Top and bottom chord
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Web member, diagonal members and plate
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Tables 1-7, 1-8 and 1-15, the geometric properties are as follows:

Top Chord
WT8×38.5
t_w = 0.455 in.
d = 8.26 in.

Bottom Chord
WT8×28.5
t_w = 0.430 in.
d = 8.22 in.

Diagonal U_0L_1
2L4×3½×⅜
A = 5.36 in.^{2}
$\bar{x}$ = 0.947 in. (for single angle)

Web U_1L_1
2L3½×3×5⁄16
A = 3.90 in.^{2}

Diagonal U_1L_2
2L3½×2½×5⁄16
A = 3.58 in.^{2}
$\bar{x}$ = 0.632 in. (for single angle)

As shown in Figure II.C-2-1, the required forces are:

LRFD	ASD
Web U_1L_1 load: $P_u = -104$ kips Diagonal U_0L_1 load: $T_u = +165$ kips Diagonal U_1L_2 load: $T_u = +114$ kips	Web U_1L_1 load: $P_a = -69.2$ kips Diagonal U_0L_1 load: $T_a = +110$ kips Diagonal U_1L_2 load: $T_a = +76$ kips

Solution A:

Shear Yielding of Bottom Chord Stem

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the bottom chord at Section A-A (see Figure II.C-2-1) is determined as follows:

$$\begin{aligned}A_{gv} &= dt_w\\ &= (8.22\text{ in.})(0.430\text{ in.})\\ &= 3.53\text{ in.}^2\end{aligned}$$

$$\begin{aligned}R_n &= 0.60F_yA_{gv}\\ &= 0.60(50\text{ ksi})(3.53\text{ in.}^2)\\ &= 106\text{ kips}\end{aligned}$$ (*Spec*. Eq. J4-3)

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(106\text{ kips})$ $= 106\text{ kips} > 104\text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{106\text{ kips}}{1.50}$ $= 70.7\text{ kips} > 69.2\text{ kips}$ **o.k.**

Welds for Member U$_1$L$_1$

Note: AISC *Specification* Section J1.7 requiring that the center of gravity of the weld group coincide with the center of gravity of the member does not apply to end connections of statically loaded single angle, double angle and similar members.

From AISC *Specification* Table J2.4, the minimum weld size for a 5⁄16-in.-thick angle is:

$w_{min} = \text{3⁄16 in.}$

From AISC *Specification* Section J2.2b(b)(2), the maximum weld size is:

$$\begin{aligned}w_{max} &= t - \text{1⁄16 in.}\\ &= \text{5⁄16} - \text{1⁄16 in.}\\ &= \text{¼ in.}\end{aligned}$$

Try a 3⁄16 in. fillet weld.

The minimum weld length is determined using AISC *Manual* Equation 8-2a or 8-2b:

LRFD	ASD
$l_{min} = \dfrac{R_u}{(2\text{ sides})(2\text{ welds})(1.392\text{ kip/in.})D}$ $= \dfrac{104\text{ kips}}{(2\text{ sides})(2\text{ welds})(1.392\text{ kip/in.})(3)}$ $= 6.23\text{ in.}$ Use a 6½-in.-long weld at the heel and toe of the angles.	$l_{min} = \dfrac{R_a}{(2\text{ sides})(2\text{ welds})(0.928\text{ kip/in.})D}$ $= \dfrac{69.2\text{ kips}}{(2\text{ sides})(2\text{ welds})(0.928\text{ kip/in.})(3)}$ $= 6.21\text{ in.}$ Use a 6½-in.-long weld at the heel and toe of the angles.

Shear Rupture Strength of Angles at Welds

The minimum angle thickness to match the required shear rupture strength of the welds is determined as follows:

$$t_{min} = \frac{3.09D}{F_u} \quad \text{(\textit{Manual} Eq. 9-2)}$$
$$= \frac{3.09(3)}{58\text{ ksi}}$$
$$= 0.160\text{ in.} < \tfrac{5}{16}\text{ in.} \quad \textbf{o.k.}$$

Shear Rupture Strength of Tee-Stem at Welds

The minimum tee-stem thickness to match the required shear rupture strength of the welds is determined as follows:

$$t_{min} = \frac{6.19D}{F_u} \quad \text{(\textit{Manual} Eq. 9-3)}$$
$$= \frac{6.19(3)}{65\text{ ksi}}$$
$$= 0.286\text{ in.} < 0.430\text{ in.} \quad \textbf{o.k.}$$

Note, both the top and bottom chords are acceptable for 3⁄16-in. fillet welds.

Welds for Member U_0L_1

From AISC *Specification* Table J2.4, the minimum weld size for a 3⁄8-in.-thick angle is:

$$w_{min} = \tfrac{3}{16}\text{ in.}$$

From AISC *Specification* Section J2.2b(b)(2), the maximum weld size is:

$$w_{max} = t - \tfrac{1}{16}\text{ in.}$$
$$= \tfrac{3}{8} - \tfrac{1}{16}\text{ in.}$$
$$= \tfrac{5}{16}\text{ in.}$$

Try a 3⁄16 in. fillet weld.

The minimum weld length is determined using AISC *Manual* Equation 8-2a or 8-2b:

LRFD	ASD
$l_{min} = \frac{R_u}{(2\text{ sides})(2\text{ welds})(1.392\text{ kip/in.})D}$ $= \frac{165\text{ kips}}{(2\text{ sides})(2\text{ welds})(1.392\text{ kip/in.})(3)}$ $= 9.88\text{ in.}$ Use a 10-in.-long weld at the heel and toe of the angles.	$l_{min} = \frac{R_a}{(2\text{ sides})(2\text{ welds})(0.928\text{ kip/in.})D}$ $= \frac{110\text{ kips}}{(2\text{ sides})(2\text{ welds})(0.928\text{ kip/in.})(3)}$ $= 9.88\text{ in.}$ Use a 10-in.-long weld at the heel and toe of the angles.

Note: A plate will be welded to the stem of the WT to provide room for the connection. Based on the preceding calculations for the minimum angle and stem thicknesses, by inspection the angles, stems, and stem plate extension have adequate strength.

Tensile Strength of Diagonal U_0L_1

From AISC *Specification* Section D2, the available tensile yielding strength of the angles is determined as follows:

$$
\begin{aligned}
P_n &= F_y A_g \qquad \text{(}\textit{Spec}\text{. Eq. D2-1)}\\
&= (36\text{ ksi})(5.36\text{ in.}^2)\\
&= 193\text{ kips}
\end{aligned}
$$

LRFD	ASD
$\phi_t = 0.90$ $\phi_t P_n = 0.90(193\text{ kips})$ $= 174\text{ kips} > 165\text{ kips}$ **o.k.**	$\Omega_t = 1.67$ $\dfrac{P_n}{\Omega_t} = \dfrac{193\text{ kips}}{1.67}$ $= 116\text{ kips} > 110\text{ kips}$ **o.k.**

From AISC *Specification* Section D2, the available tensile rupture strength of the angles is determined as follows. The shear lag factor, U, is determined using AISC *Specification* Table D3.1, Case 4.

$$
\begin{aligned}
U &= \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right)\\
&= \frac{3(10\text{ in.})^2}{3(10\text{ in.})^2 + (4\text{ in.})^2}\left(1 - \frac{0.947\text{ in.}}{10\text{ in.}}\right)\\
&= 0.859
\end{aligned}
$$

$$
\begin{aligned}
P_n &= F_u A_e \qquad \text{(}\textit{Spec}\text{. Eq. D2-2)}\\
&= (58\text{ ksi})(5.36\text{ in.}^2)(0.859)\\
&= 267\text{ kips}
\end{aligned}
$$

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(267\text{ kips})$ $= 200\text{ kips} > 165\text{ kips}$ **o.k.**	$\Omega_t = 2.00$ $\dfrac{P_n}{\Omega_t} = \dfrac{267\text{ kips}}{2.00}$ $= 134\text{ kips} > 110\text{ kips}$ **o.k.**

Block Shear Rupture of Bottom Chord

The available strength for the limit state of block shear rupture of the chord is determined as follows.

$$
R_n = 0.60F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.60F_y A_{gv} + U_{bs} F_u A_{nt} \qquad \text{(}\textit{Spec}\text{. Eq. J4-5)}
$$

where

$$
\begin{aligned}
A_{gv} &= A_{nv}\\
&= (2\text{ lines})\,l t_w\\
&= (2\text{ lines})(10\text{ in.})(0.430\text{ in.})\\
&= 8.60\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
A_{nt} &= (angle\ leg)t \\
&= (4\text{ in.})(0.430\text{ in.}) \\
&= 1.72\text{ in.}^2
\end{aligned}
$$

$$U_{bs} = 1.0$$

Note, because ASTM A36 is used for the stem extension plate, F_y = 36 ksi and F_u = 58 ksi are used for the shear components of AISC *Specification* Equation J4-5.

$$
\begin{aligned}
R_n &= 0.60(58\text{ ksi})(8.60\text{ in.}^2)+1.0(65\text{ ksi})(1.72\text{ in.}^2) \le 0.60(36\text{ ksi})(8.60\text{ in.}^2)+1.0(65\text{ ksi})(1.72\text{ in.}^2) \\
&= 411\text{ kips} > 298\text{ kips}
\end{aligned}
$$

Therefore:

$$R_n = 298\text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(298\text{ kips})$ $= 224\text{ kips} > 165\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{298\text{ kips}}{2.00}$ $= 149\text{ kips} > 110\text{ kips}$ **o.k.**

Solution B:

Shear Yielding of Top Chord Stem

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the top chord at Section B-B (see Figure II.C-2-1) is determined as follows:

$$
\begin{aligned}
A_{gv} &= dt_w \\
&= (8.26\text{ in.})(0.455\text{ in.}) \\
&= 3.76\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
R_n &= 0.60F_yA_{gv} \\
&= 0.60(50\text{ ksi})(3.76\text{ in.}^2) \\
&= 113\text{ kips}
\end{aligned}
$$

(*Spec*. Eq. J4-3)

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(113\text{ kips})$ $= 113\text{ kips} > 74.0\text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{113\text{ kips}}{1.50}$ $= 75.3\text{ kips} > 49.2\text{ kips}$ **o.k.**

Welds for Member U_1L_1

As calculated previously in Solution A, use 6½-in.-long 3⁄16-in. fillet welds at the heel and toe of both angles.

Welds for Member U_1L_2

As determined in previous calculations, the minimum and maximum weld sizes for a 5⁄16-in.-thick angle are:

$$w_{min} = \tfrac{3}{16}\text{ in.}$$
$$w_{max} = \tfrac{1}{4}\text{ in.}$$

Try a ¼ in. fillet weld.

To avoid having to use a stem extension plate unequal length welds are provided at the heel and toe of the angle. The minimum weld length for each angle is determined using AISC *Manual* Equation 8-2a or 8-2b:

LRFD	ASD
$l_{min} = \dfrac{R_u}{(2\text{ sides})(1.392\text{ kip/in.})D}$ $= \dfrac{114\text{ kips}}{(2\text{ sides})(1.392\text{ kip/in.})(4)}$ $= 10.2\text{ in.}$	$l_{min} = \dfrac{R_a}{(2\text{ sides})(0.928\text{ kip/in.})D}$ $= \dfrac{76\text{ kips}}{(2\text{ sides})(0.928\text{ kip/in.})(4)}$ $= 10.2\text{ in.}$

Try 7½ in. of ¼-in. fillet weld at the heel and 4 in. of ¼-in. fillet weld at the toe of each angle.

$$\begin{aligned} l &= 7\tfrac{1}{2}\text{ in.} + 4\text{ in.} \\ &= 11.5\text{ in.} > 10.2\text{ in.} \quad \textbf{o.k.} \end{aligned}$$

Shear Rupture Strength of Angles at Welds

The minimum angle thickness to match the required shear rupture strength of the welds is determined as follows:

$$\begin{aligned} t_{min} &= \frac{3.09D}{F_u} \qquad (\textit{Manual}\text{ Eq. 9-2}) \\ &= \frac{3.09(4)}{58\text{ ksi}} \\ &= 0.213\text{ in.} < \tfrac{5}{16}\text{ in.} \quad \textbf{o.k.} \end{aligned}$$

Shear Rupture Strength of Tee-Stem at Welds

The minimum tee-stem thickness to match the required shear rupture strength of the welds is determined as follows:

$$\begin{aligned} t_{min} &= \frac{6.19D}{F_u} \qquad (\textit{Manual}\text{ Eq. 9-3}) \\ &= \frac{6.19(4)}{65\text{ ksi}} \\ &= 0.381\text{ in.} < 0.455\text{ in.} \quad \textbf{o.k.} \end{aligned}$$

Tensile Strength of Diagonal U_1L_2

From AISC *Specification* Section J4.1(a), the available tensile yielding strength of the angles are determined as follows:

$$R_n = F_y A_g \qquad \text{(Spec. Eq. J4-1)}$$
$$= (36 \text{ ksi})(3.58 \text{ in.}^2)$$
$$= 129 \text{ kips}$$

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(129 \text{ kips})$ $= 116 \text{ kips} > 114 \text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{129 \text{ kips}}{1.67}$ $= 77.2 \text{ kips} > 76 \text{ kips}$ **o.k.**

From AISC *Specification* Section J4.1(b), the available tensile rupture strength of the angles are determined as follows. The shear lag factor, U, is determined using AISC *Specification* Table D3.1, Case 4.

$$l = \frac{l_1 + l_2}{2}$$
$$= \frac{7\tfrac{1}{2} \text{ in.} + 4 \text{ in.}}{2}$$
$$= 5.75 \text{ in.}$$

$$U = \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right)$$
$$= \frac{3(5.75 \text{ in.})^2}{3(5.75 \text{ in.})^2 + (3\tfrac{1}{2} \text{ in.})^2}\left(1 - \frac{0.632 \text{ in.}}{5.75 \text{ in.}}\right)$$
$$= 0.792$$

$$R_n = F_u A_e \qquad \text{(Spec. Eq. J4-2)}$$
$$= (58 \text{ ksi})(3.58 \text{ in.}^2)(0.792)$$
$$= 164 \text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(164 \text{ kips})$ $= 123 \text{ kips} > 114 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{164 \text{ kips}}{2.00}$ $= 82.0 \text{ kips} > 76 \text{ kips}$ **o.k.**

Conclusion

Joints L_1 and U_1 are found to be adequate as given for the applied loads.

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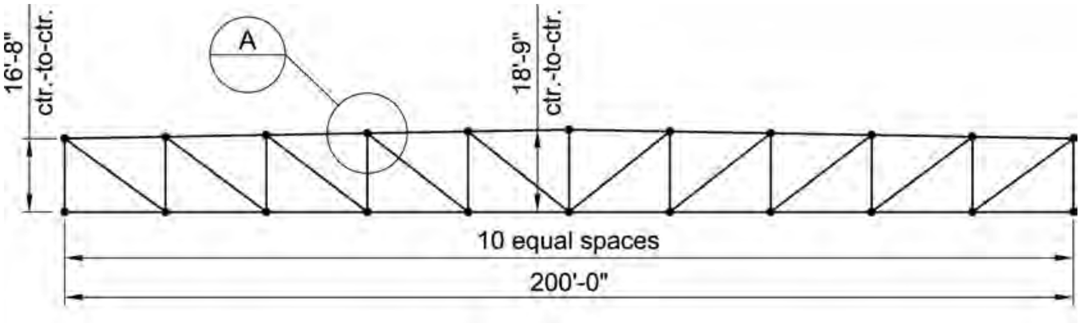
EXAMPLE II.C-3 HEAVY WIDE-FLANGE COMPRESSION CONNECTION (FLANGES ON THE OUTSIDE)

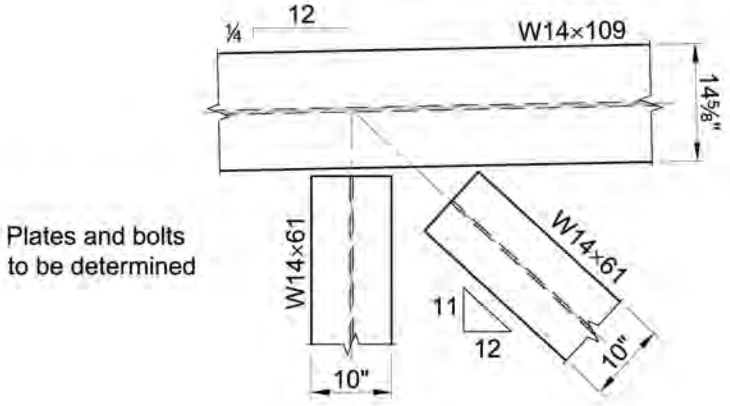
Given:

The truss shown in Figure II.C-3-1 has been designed with ASTM A992 W14 shapes with flanges to the outside of the truss. Beams framing into the top chord and lateral bracing are not shown but can be assumed to be adequate.

Based on multiple load cases, the critical dead and live load forces for this connection are shown in Figure II.C-3-2. A typical top chord connection is shown in Figure II.C-3-1, Detail A. Design this typical connection using 1-in.-diameter Group A slip-critical bolts in standard holes with threads not excluded from the shear plane (thread condition N) with Class A faying surfaces and ASTM A36 gusset plates.

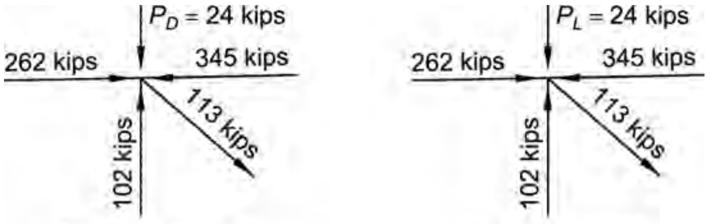


Elevation



Detail A

Fig II.C-3-1. Truss layout for Example II.C-3.



Dead Load Force

Live Load Force

Fig. II.C-3-2. Forces at Detail A.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

W-shapes
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Gusset plates
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Top chord
W14×109
d = 14.3 in.
b_f = 14.6 in.
t_f = 0.860 in.

Web members
W14×61
d = 13.9 in.
b_f = 10.0 in.
t_f = 0.645 in.

From AISC *Specification* Table J3.3, for 1-in.-diameter bolts with standard holes:

$d_h = 1\frac{1}{8}$ in.

From ASCE/SEI 7, Chapter 2, the required strengths are determined as follows and summarized in Figure II.C-3-2.

LRFD	ASD
Left top chord: $P_u = 1.2(262\text{ kips}) + 1.6(262\text{ kips})$ $= 734\text{ kips}$ Right top chord: $P_u = 1.2(345\text{ kips}) + 1.6(345\text{ kips})$ $= 966\text{ kips}$ Vertical Web: $P_u = 1.2(102\text{ kips}) + 1.6(102\text{ kips})$ $= 286\text{ kips}$	Left top chord: $P_a = 262\text{ kips} + 262\text{ kips}$ $= 524\text{ kips}$ Right top chord: $P_a = 345\text{ kips} + 345\text{ kips}$ $= 690\text{ kips}$ Vertical Web: $P_a = 102\text{ kips} + 102\text{ kips}$ $= 204\text{ kips}$

LRFD	ASD
Diagonal Web: $P_u = 1.2(113\text{ kips}) + 1.6(113\text{ kips})$ $= 316\text{ kips}$	Diagonal Web: $P_a = 113\text{ kips} + 113\text{ kips}$ $= 226\text{ kips}$

Note: In checking equilibrium of vertical forces, $\Sigma F_y \neq 0$, due to the external (loading) forces not included. Refer to Figure II.C-3-2 for the magnitude of external load forces. In most truss designs, member forces only are provided and force equilibrium of the internal truss forces will not sum to zero.

Bolt Slip Resistance Strength

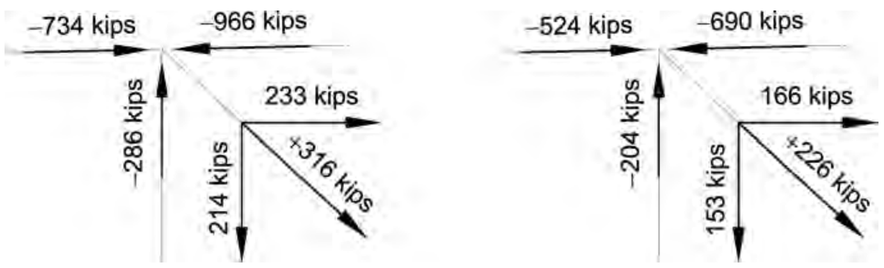
From AISC *Specification* Section J3.8(a), the available slip resistance for the limit state of slip for standard size holes is determined as follows:

$\mu = 0.30$ for Class A surface
$D_u = 1.13$
$h_f = 1.0,$ no filler is provided
$T_b = 51$ kips, from AISC *Specification* Table J3.1, Group A
$n_s = 1,$ number of slip planes

$$\begin{aligned} r_n &= \mu D_u h_f T_b n_s \\ &= (0.30)(1.13)(1.0)(51\text{ kips})(1) \\ &= 17.3\text{ kips/bolt} \end{aligned}$$ (*Spec*. Eq. J3-4)

LRFD	ASD
$\phi = 1.00$ $\phi r_n = 1.00(17.3\text{ kips/bolt})$ $= 17.3\text{ kips/bolt}$	$\Omega = 1.50$ $\dfrac{r_n}{\Omega} = \dfrac{17.3\text{ kips/bolt}}{1.50}$ $= 11.5\text{ kips/bolt}$

Note: Standard holes are used in both plies for this example. Other hole sizes may be used and should be considered based on the preferences of the fabricator or erector on a case-by-case basis.



(a) LRFD (b) ASD

Fig. II.C-3-2. Required forces at Detail A.

Diagonal Connection

The required number of bolts is determined as follows:

LRFD	ASD
$P_u = 316 \text{ kips}$ $n_{req} = \dfrac{P_u}{\phi r_n} = \dfrac{316 \text{ kips}}{17.3 \text{ kips/bolt}} = 18.3 \text{ bolts}$ For two lines of bolts on both sides, the required number of rows is: $\dfrac{18.3 \text{ bolts}}{(2 \text{ sides})(2 \text{ lines})} = 4.58$ Therefore, use five rows at min. 3-in. spacing.	$P_a = 226 \text{ kips}$ $n_{req} = \dfrac{\Omega P_a}{r_n} = \dfrac{226 \text{ kips}}{11.5 \text{ kips/bolt}} = 19.7 \text{ bolts}$ For two lines of bolts on both sides, the required number of rows is: $\dfrac{19.7 \text{ bolts}}{(2 \text{ sides})(2 \text{ lines})} = 4.93$ Therefore, use five rows at min. 3-in. spacing.

Whitmore section in gusset plate

The width of the Whitmore section, l_w, is determined as shown in AISC *Manual* Figure 9-1.

$$\begin{aligned} l_w &= gage + 2l\tan 30° \\ &= 5\tfrac{1}{2} \text{ in.} + 2(12 \text{ in.})(\tan 30°) \\ &= 19.4 \text{ in.} \end{aligned}$$

Try a ⅜-in.-thick plate.

$$\begin{aligned} A_g &= (2 \text{ plates})l_w t \\ &= (2 \text{ plates})(19.4 \text{ in.})(\tfrac{3}{8} \text{ in.}) \\ &= 14.6 \text{ in.}^2 \end{aligned}$$

From AISC *Specification* Section J4.1(a), the available tensile yielding strength of the gusset plate is determined as follows:

$$\begin{aligned} R_n &= F_y A_g \\ &= (36 \text{ ksi})(14.6 \text{ in.}^2) \\ &= 526 \text{ kips} \end{aligned}$$ (*Spec*. Eq. J4-1)

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(526 \text{ kips}) = 473 \text{ kips} > 316 \text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{526 \text{ kips}}{1.67} = 315 \text{ kips} > 226 \text{ kips}$ **o.k.**

Block shear rupture of gusset plate

The available strength for the limit state of block shear rupture of the gusset plates is determined as follows.

$$R_n = 0.60F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.60 F_y A_{gv} + U_{bs} F_u A_{nt} \qquad (\textit{Spec.}\ \text{Eq. J4-5})$$

where

$$\begin{aligned} A_{gv} &= (2\text{ plates})(2\text{ lines})\left[l_{ev} + (n-1)s\right]t \\ &= (2\text{ plates})(2\text{ lines})\left[2\text{ in.} + (5-1)(3\text{ in.})\right](\tfrac{3}{8}\text{ in.}) \\ &= 21.0\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - (2\text{ plates})(2\text{ lines})(5-0.5)(d_h + \tfrac{1}{16}\text{ in.})t \\ &= 21.0\text{ in.}^2 - (2\text{ plates})(2\text{ lines})(5-0.5)(1\tfrac{1}{8}\text{ in.} + \tfrac{1}{16}\text{ in.})(\tfrac{3}{8}\text{ in.}) \\ &= 13.0\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= (2\text{ plates})\left[gage - (d_h + \tfrac{1}{16}\text{ in.})\right]t \\ &= (2\text{ plates})\left[5\tfrac{1}{2}\text{ in.} - (1\tfrac{1}{8}\text{ in.} + \tfrac{1}{16}\text{ in.})\right](\tfrac{3}{8}\text{ in.}) \\ &= 3.23\text{ in.}^2 \end{aligned}$$

$$U_{bs} = 1.0$$

and

$$\begin{aligned} R_n &= 0.60(58\text{ ksi})(13.0\text{ in.}^2) + 1.0(58\text{ ksi})(3.23\text{ in.}^2) \leq 0.60(36\text{ ksi})(21.0\text{ in.}^2) + 1.0(58\text{ ksi})(3.23\text{ in.}^2) \\ &= 640\text{ kips} < 641\text{ kips} \end{aligned}$$

Therefore:

$$R_n = 640\text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(640\text{ kips})$ $= 480\text{ kips} > 316\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{640\text{ kips}}{2.00}$ $= 320\text{ kips} > 226\text{ kips}$ **o.k.**

Block shear rupture of diagonal flange

By inspection, block shear rupture on the diagonal flange will not control.

Strength of bolted connection—gusset plate

Slip-critical connections must also be designed for the limit states of bearing-type connections. From the Commentary to AISC *Specification* Section J3.6, the strength of the bolt group is taken as the sum of the individual strengths of the individual fasteners, which may be taken as the lesser of the fastener shear strength per AISC *Specification* Section J3.6, the bearing strength at the bolt hole per AISC *Specification* Section J3.10, or the tearout strength at the bolt hole per AISC *Specification* Section J3.10.

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From AISC *Manual* Table 7-1, the available shear strength per bolt for 1-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) is:

LRFD	ASD
$\phi r_n = 31.8\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 21.2\text{ kips/bolt}$

The available bearing and tearout strength of the gusset plate at the edge bolts is determined using AISC *Manual* Table 7-5, using l_e = 2 in.

LRFD	ASD
$\phi r_n = \left(75.0\text{ kip/in.}\right)\left(\tfrac{3}{8}\text{ in.}\right)$ $= 28.1\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = \left(50.0\text{ kip/in.}\right)\left(\tfrac{3}{8}\text{ in.}\right)$ $= 18.8\text{ kips/bolt}$

Therefore, the bearing or tearout strength controls over bolt shear at the edge bolts.

The available bearing and tearout strength of the gusset plate at the other bolts is determined using AISC *Manual* Table 7-4 with s = 3 in.

LRFD	ASD
$\phi r_n = \left(97.9\text{ kip/in.}\right)\left(\tfrac{3}{8}\text{ in.}\right)$ $= 36.7\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = \left(65.3\text{ kip/in.}\right)\left(\tfrac{3}{8}\text{ in.}\right)$ $= 24.5\text{ kips/bolt}$

Therefore, bolt shear controls over bearing or tearout at the other bolts.

The strength of the bolt group in the gusset plate is determined by summing the strength of the individual fasteners as follows:

LRFD	ASD
$\phi R_n = \left(2\text{ sides}\right)\left(2\text{ lines}\right)\begin{bmatrix}\left(1\text{ bolt}\right)\left(28.1\text{ kips/bolt}\right)\\ +\left(4\text{ bolts}\right)\left(31.8\text{ kips/bolt}\right)\end{bmatrix}$ $= 621\text{ kips} > 316\text{ kips}\quad \textbf{o.k.}$	$\dfrac{R_n}{\Omega} = \left(2\text{ sides}\right)\left(2\text{ lines}\right)\begin{bmatrix}\left(1\text{ bolt}\right)\left(18.8\text{ kips/bolt}\right)\\ +\left(4\text{ bolts}\right)\left(21.2\text{ kips/bolt}\right)\end{bmatrix}$ $= 414\text{ kips} > 226\text{ kips}\quad \textbf{o.k.}$

Strength of bolted connection—diagonal flange

By inspection the strength of the bolted connection at the gusset plate will control.

Horizontal Connection

The required strength of the gusset plate to horizontal member is determined as follows:

LRFD	ASD
$P_u = 966\text{ kips} - 734\text{ kips}$ $= 232\text{ kips}$	$P_a = 690\text{ kips} - 524\text{ kips}$ $= 166\text{ kips}$

Using the bolt slip resistance strength determined previously, the required number of rows of bolts is determined as follows:

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LRFD	ASD
$n_{req} = \dfrac{P_u}{\phi r_n}$ $= \dfrac{232 \text{ kips}}{17.3 \text{ kips/bolt}}$ $= 13.4 \text{ bolts}$ For two lines of bolts on both sides the required number of rows is: $\dfrac{13.4 \text{ bolts}}{(2 \text{ sides})(2 \text{ lines})} = 3.35$	$n_{req} = \dfrac{\Omega P_u}{r_n}$ $= \dfrac{166 \text{ kips}}{11.5 \text{ kips/bolt}}$ $= 14.4 \text{ bolts}$ For two lines of bolts on both sides the required number of rows is: $\dfrac{14.4 \text{ bolts}}{(2 \text{ sides})(2 \text{ lines})} = 3.60$

For members not subject to corrosion the maximum bolt spacing is determined using AISC *Specification* Section J3.5(a):

$$\begin{aligned} 24t &= 24(\tfrac{3}{8} \text{ in.}) \\ &= 9.00 \text{ in.} \end{aligned}$$

Due to the geometry of the gusset plate, the use of 4 rows of bolts in the horizontal connection will exceed the maximum bolt spacing; instead use 5 rows of bolts in two lines.

Shear strength of the gusset plate

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the gusset plates is determined as follows:

$$\begin{aligned} A_{gv} &= (2 \text{ plates})lt \\ &= (2 \text{ plates})(32.0 \text{ in.})(\tfrac{3}{8} \text{ in.}) \\ &= 24.0 \text{ in.}^2 \end{aligned}$$

$$R_n = 0.60 F_y A_{gv} \qquad (\textit{Spec.}\text{ Eq. J4-3})$$

$$\begin{aligned} &= 0.60(36 \text{ ksi})(24.0 \text{ in.}^2) \\ &= 518 \text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(518 \text{ kips})$ $= 518 \text{ kips} > 232 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{518 \text{ kips}}{1.50}$ $= 345 \text{ kips} > 166 \text{ kips}$ **o.k.**

From AISC *Specification* Section J4.2(b), the available shear rupture strength of gusset plates is determined as follows:

$$\begin{aligned} A_{nv} &= (2 \text{ plates})\left[l - n\left(d_h + \tfrac{1}{16} \text{ in.}\right)\right]t \\ &= (2 \text{ plates})\left[32.0 \text{ in.} - 5\left(1\tfrac{1}{8} \text{ in.} + \tfrac{1}{16} \text{ in.}\right)\right](\tfrac{3}{8} \text{ in.}) \\ &= 19.5 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60F_uA_{nv} \\ &= 0.60(58\text{ ksi})(19.5\text{ in.}^2) \\ &= 679\text{ kips} \end{aligned}$$

(*Spec*. Eq. J4-4)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(679\text{ kips})$ $= 509\text{ kips} > 232\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{679\text{ kips}}{1.50}$ $= 453\text{ kips} > 166\text{ kips}$ **o.k.**

Strength of bolted connection

By comparison to the preceding calculations for the diagonal connection, bolt bearing or tearout does not control.

Vertical Connection

Using the bolt slip resistance strength determined previously, the required number of bolts is determined as follows:

LRFD	ASD
$P_u = 286\text{ kips}$ $n_{req} = \dfrac{P_u}{\phi r_n}$ $= \dfrac{286\text{ kips}}{17.3\text{ kips/bolt}}$ $= 16.5\text{ bolts}$ For two lines of bolts on both sides, the required number of rows is: $\dfrac{16.5\text{ bolts}}{(2\text{ sides})(2\text{ lines})} = 4.12$ Therefore, use 5 rows at min. 3-in. spacing.	$P_u = 204\text{ kips}$ $n_{req} = \dfrac{\Omega P_u}{r_n}$ $= \dfrac{204\text{ kips}}{11.5\text{ kips/bolt}}$ $= 17.7\text{ bolts}$ For two lines of bolts on both sides, the required number of rows is: $\dfrac{17.7\text{ bolts}}{(2\text{ sides})(2\text{ lines})} = 4.43$ Therefore, use 5 rows at min. 3-in. spacing.

Shear strength of the gusset plate

From AISC *Specification* Section J4.2(a), the available shear yielding strength of gusset plates is determined as follows:

$$\begin{aligned} A_{gv} &= (2\text{ plates})lt \\ &= (2\text{ plates})(31\tfrac{3}{4}\text{ in.})(\tfrac{3}{8}\text{ in.}) \\ &= 23.8\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= 0.60F_yA_{gv} \\ &= 0.60(36\text{ ksi})(23.8\text{ in.}^2) \\ &= 514\text{ kips} \end{aligned}$$

(*Spec*. Eq. J4-3)

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(514 \text{ kips})$ $= 514 \text{ kips} > 286 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{514 \text{ kips}}{1.50}$ $= 343 \text{ kips} > 204 \text{ kips}$ **o.k.**

From AISC *Specification* Section J4.2(b), the available shear rupture strength of gusset plates is determined as follows:

$$\begin{aligned} A_{nv} &= (2 \text{ plates})\left[l - n\left(d_h + \tfrac{1}{16} \text{ in.}\right)\right]t \\ &= (2 \text{ plates})\left[31\tfrac{3}{4} \text{ in.} - 7\left(1\tfrac{1}{8} \text{ in.} + \tfrac{1}{16} \text{ in.}\right)\right]\left(\tfrac{3}{8} \text{ in.}\right) \\ &= 17.6 \text{ in.}^2 \end{aligned}$$

$$R_n = 0.60 F_u A_{nv}$$ (*Spec*. Eq. J4-4)

$$\begin{aligned} &= 0.60(58 \text{ ksi})\left(17.6 \text{ in.}^2\right) \\ &= 612 \text{ kips} \end{aligned}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(612 \text{ kips})$ $= 459 \text{ kips} > 286 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{612 \text{ kips}}{2.00}$ $= 306 \text{ kips} > 204 \text{ kips}$ **o.k.**

Strength of bolted connection

By comparison to the preceding calculations for the diagonal connection, bolt bearing does not control.

Note that because of the difference in depths between the top chord and the vertical and diagonal members, $\tfrac{3}{16}$-in. loose shims are required on each side of the shallower members.

The final connection design is shown in Figure II.C-3-4.

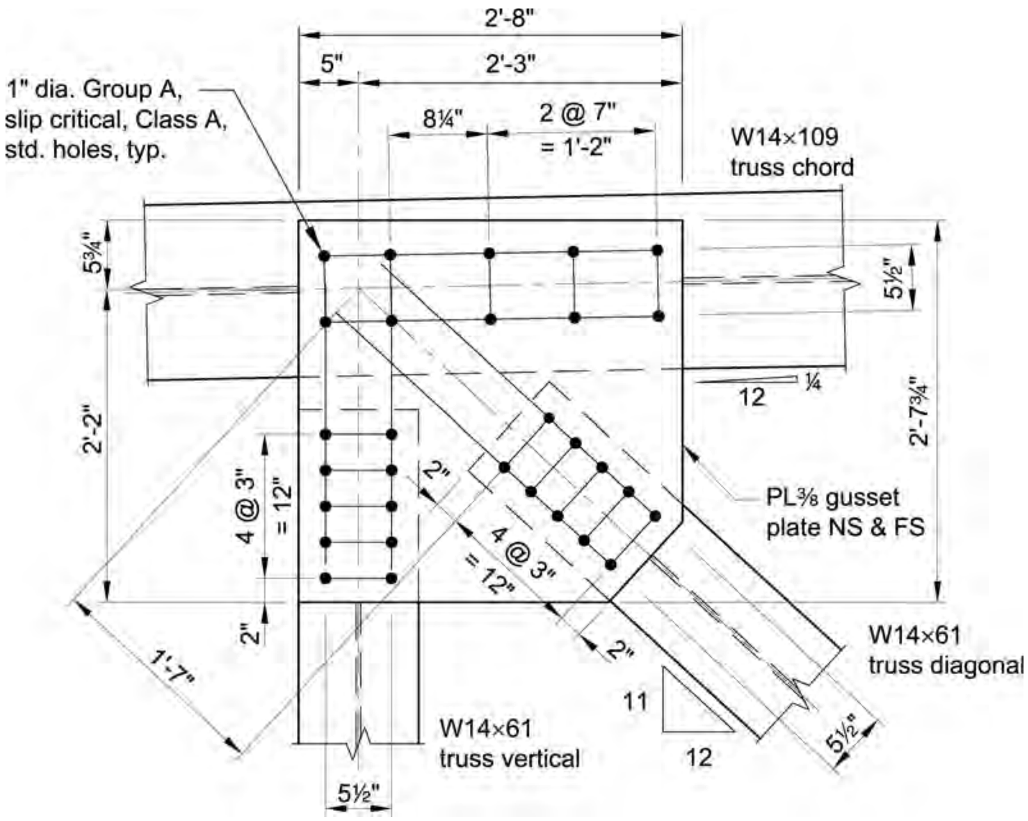

Fig. II.C-3-4. Connection layout for Example II.C-3.

Chapter IID
Miscellaneous Connections

This section contains design examples on connections in the AISC *Steel Construction Manual* that are not covered in other sections of the AISC *Design Examples*.

EXAMPLE II.D-1 WT HANGER CONNECTION

Given:

Design an ASTM A992 WT hanger connection between an ASTM A36 2L3×3×5⁄16 tension member and an ASTM A992 W24×94 beam to support the following loads:

P_D = 13.5 kips
P_L = 40 kips

Use 70-ksi electrodes.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam and WT hanger
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Angles
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Tables 1-1, 1-7 and 1-15, the geometric properties are as follows:

Beam
W24×94
d = 24.3 in.
t_w = 0.515 in.
b_f = 9.07 in.
t_f = 0.875 in.

Angles
2L3×3×5⁄16
A = 3.56 in.^{2}
$\bar{x}$ = 0.860 in. (for single angle)

From AISC *Specification* Table J3.3, the hole diameter for ¾-in.-diameter bolts with standard holes is:

$d_h = 13/16$ in.

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$P_u = 1.2(13.5\text{ kips}) + 1.6(40\text{ kips})$ $= 80.2\text{ kips}$	$P_a = 13.5\text{ kips} + 40\text{ kips}$ $= 53.5\text{ kips}$

Weld Design

Note: AISC *Specification* Section J1.7 requiring that the center of gravity of the weld group coincide with the center of gravity of the member does not apply to end connections of statically loaded single-angle, double-angle and similar members.

From AISC *Specification* Table J2.4, the minimum weld size for a 5⁄16-in.-thick angle is:

$$w_{min} = \tfrac{3}{16} \text{ in.}$$

From AISC *Specification* Section J2.2b(b)(2), the maximum weld size is:

$$\begin{aligned} w_{max} &= t - \tfrac{1}{16} \text{ in.} \\ &= \tfrac{5}{16} - \tfrac{1}{16} \text{ in.} \\ &= \tfrac{1}{4} \text{ in.} \end{aligned}$$

Try ¼-in. fillet welds.

The minimum weld length is determined using AISC *Manual* Equations 8-2a or 8-2b, as follows:

LRFD	ASD
$l_{min} = \dfrac{R_u}{(2 \text{ sides})(2 \text{ welds})(1.392 \text{ kip/in.})D}$ $= \dfrac{80.2 \text{ kips}}{(2 \text{ sides})(2 \text{ welds})(1.392 \text{ kip/in.})(4)}$ $= 3.60 \text{ in.}$ Use a 4-in.-long weld at the heel and toe of the angles.	$l_{min} = \dfrac{R_a}{(2 \text{ sides})(2 \text{ welds})(0.928 \text{ kip/in.})D}$ $= \dfrac{53.5 \text{ kips}}{(2 \text{ sides})(2 \text{ welds})(0.928 \text{ kip/in.})(4)}$ $= 3.60 \text{ in.}$ Use a 4-in.-long weld at the heel and toe of the angles.

Tensile Strength of Angles

From AISC *Specification* Section D2, the available tensile yielding strength of the angles is determined as follows:

$$\begin{aligned} P_n &= F_y A_g \\ &= (36 \text{ ksi})(3.56 \text{ in.}^2) \\ &= 128 \text{ kips} \end{aligned}$$ (*Spec*. Eq. D2-1)

LRFD	ASD
$\phi_t = 0.90$ $\phi_t P_n = 0.90(128 \text{ kips})$ $= 115 \text{ kips} > 80.2 \text{ kips}$ **o.k.**	$\Omega_t = 1.67$ $\dfrac{P_n}{\Omega_t} = \dfrac{128 \text{ kips}}{1.67}$ $= 76.6 \text{ kips} > 53.5 \text{ kips}$ **o.k.**

From AISC *Specification* Section D2, the available tensile rupture strength of the brace is determined as follows:

$$\begin{aligned} A_n &= A_g \\ &= 3.56 \text{ in.}^2 \end{aligned}$$

The shear lag factor, U, is determined from AISC *Specification* Table D3.1, Case 4:

$$U = \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right)$$
$$= \frac{3(4\text{ in.})^2}{3(4\text{ in.})^2 + (3\text{ in.})^2}\left(1 - \frac{0.860\text{ in.}}{4\text{ in.}}\right)$$
$$= 0.661$$

$$A_e = A_n U \qquad (Spec.\ \text{Eq. D3-1})$$
$$= (3.56\text{ in.}^2)(0.661)$$
$$= 2.35\text{ in.}^2$$

$$P_n = F_u A_e \qquad (Spec.\ \text{Eq. D2-2})$$
$$= (58\text{ ksi})(2.35\text{ in.}^2)$$
$$= 136\text{ kips}$$

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(136\text{ kips})$ $= 102\text{ kips} > 80.2\text{ kips}$ **o.k.**	$\Omega_t = 2.00$ $\dfrac{P_n}{\Omega_t} = \dfrac{136\text{ kips}}{2.00}$ $= 68.0\text{ kips} > 53.5\text{ kips}$ **o.k.**

Preliminary WT Selection Using Beam Gage

Try four ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N), with a 4-in. gage.

LRFD	ASD
$T_r = r_{ut}$ $= \dfrac{P_u}{n}$ $= \dfrac{80.2\text{ kips}}{4\text{ bolts}}$ $= 20.1\text{ kips/bolt}$ From AISC *Manual* Table 7-2: $B_c = \phi r_n$ $= 29.8\text{ kips/bolt} > 20.1\text{ kips/bolt}$ **o.k.**	$T_r = r_{at}$ $= \dfrac{P_a}{n}$ $= \dfrac{53.5\text{ kips}}{4\text{ bolts}}$ $= 13.4\text{ kips/bolt}$ From AISC *Manual* Table 7-2: $B_c = \dfrac{r_n}{\Omega}$ $= 19.9\text{ kips/bolt} > 13.4\text{ kips/bolt}$ **o.k.**

Determine tributary length per pair of bolts, *p*, using AISC *Manual* Figure 9-4.

$$p = \frac{4\tfrac{1}{2}\text{ in.}}{2} + \frac{8.00\text{ in.} - 4\tfrac{1}{2}\text{ in.}}{2}$$
$$= 4.00\text{ in.}$$

Check:

$$p \le s$$
$$4.00\text{ in.} < 4\tfrac{1}{2}\text{ in.} \quad \textbf{o.k.}$$

Verify that the tributary length on each side of the bolt conforms to dimensional limits assuming a ½-in. tee stem thickness:

$$\begin{aligned} b &= \frac{\left(4.00\text{ in.} - \tfrac{1}{2}\text{ in.}\right)}{2} \\ &= 1.75\text{ in.} \end{aligned}$$

$$\frac{4\tfrac{1}{2}\text{ in.}}{2} \le 1.75b$$
$$2.25\text{ in.} < 3.06\text{ in.} \quad \textbf{o.k.}$$

$$\frac{8.00\text{ in.} - 4\tfrac{1}{2}\text{ in.}}{2} \le 1.75b$$
$$1.75\text{ in.} < 3.06\text{ in.} \quad \textbf{o.k.}$$

A preliminary hanger connection is determined using AISC *Manual* Table 15-2b.

LRFD	ASD
$$\begin{aligned} 2R_{ut} &= \frac{(rows)B_c}{p} \\ &= \frac{(2)(20.1\text{ kips/bolt})}{4.00\text{ in.}} \\ &= 10.1\text{ kip/in.} \end{aligned}$$	$$\begin{aligned} 2R_{at} &= \frac{(rows)B_c}{p} \\ &= \frac{(2\text{ bolts})(13.4\text{ kips/bolt})}{4.00\text{ in.}} \\ &= 6.70\text{ kip/in.} \end{aligned}$$

From AISC *Manual* Table 15-2b, with an assumed b = (4.00 in. – ½ in.)/2 = 1.75 in., the flange thickness, $t = t_f$, of the WT hanger should be approximately ⅝ in.

The minimum depth WT that can be used is equal to the sum of the weld length plus the weld size plus the k-dimension for the selected section. From AISC *Manual* Table 1-8 with an assumed b = 1.75 in., $t_f \approx$ ⅝ in., and d_{min} = 4 in. + ¼ in. + $k \approx$ 6 in., appropriate selections include:

WT6×25
WT7×26.5
WT8×25
WT9×27.5

Try a WT6×25.

From AISC *Manual* Table 1-8, the geometric properties are as follows:

b_f = 8.08 in.
t_f = 0.640 in.
t_w = 0.370 in.

Prying Action

From AISC *Manual* Part 9, the available tensile strength of the bolts taking prying action into account is determined as follows. The beam flange is thicker than the WT flange; therefore, prying in the tee flange will control over prying in the beam flange.

$$\begin{aligned} a &= \frac{b_f - gage}{2} \\ &= \frac{8.08\text{ in.} - 4\text{ in.}}{2} \\ &= 2.04\text{ in.} \end{aligned}$$

$$\begin{aligned} b &= \frac{gage - t_w}{2} \\ &= \frac{4\text{ in.} - 0.370\text{ in.}}{2} \\ &= 1.82\text{ in.} \end{aligned}$$

$$\begin{aligned} b' &= b - \frac{d_b}{2} \\ &= 1.82\text{ in.} - \left(\frac{\tfrac{3}{4}\text{ in.}}{2}\right) \\ &= 1.45\text{ in.} \end{aligned}$$ (*Manual* Eq. 9-18)

$$\begin{aligned} a' &= \left(a + \frac{d_b}{2}\right) \le \left(1.25b + \frac{d_b}{2}\right) \\ &= 2.04\text{ in.} + \frac{\tfrac{3}{4}\text{ in.}}{2} \le 1.25(1.82\text{ in.}) + \frac{\tfrac{3}{4}\text{ in.}}{2} \\ &= 2.42\text{ in.} < 2.65\text{ in.} \end{aligned}$$ (*Manual* Eq. 9-23)

$$\begin{aligned} \rho &= \frac{b'}{a'} \\ &= \frac{1.45\text{ in.}}{2.42\text{ in.}} \\ &= 0.599 \end{aligned}$$ (*Manual* Eq. 9-22)

From AISC *Manual* Equation 9-21:

LRFD	ASD
$\beta = \frac{1}{\rho}\left(\frac{B_c}{T_r} - 1\right)$ $= \frac{1}{0.599}\left(\frac{29.8\text{ kips/bolt}}{20.1\text{ kips/bolt}} - 1\right)$ $= 0.806$	$\beta = \frac{1}{\rho}\left(\frac{B_c}{T_r} - 1\right)$ $= \frac{1}{0.599}\left(\frac{19.9\text{ kips/bolt}}{13.4\text{ kips/bolt}} - 1\right)$ $= 0.810$

$$\begin{aligned} d' &= d_h \\ &= \tfrac{13}{16}\text{ in.} \end{aligned}$$

$$\delta = 1 - \frac{d'}{p} \qquad \text{(}\textit{Manual}\text{ Eq. 9-20)}$$

$$= 1 - \frac{{}^{13}\!/_{16}\text{ in.}}{4.00\text{ in.}}$$

$$= 0.797$$

Because $\beta < 1.0$:

LRFD	ASD
$\alpha' = \frac{1}{\delta}\left(\frac{\beta}{1-\beta}\right) \le 1.0$ $= \frac{1}{0.797}\left(\frac{0.806}{1-0.806}\right) > 1.0$ $= 5.21 > 1.0$ Therefore, $\alpha' = 1.0$. $\phi = 0.90$ $t_{min} = \sqrt{\frac{4T_u b'}{\phi p F_u (1+\delta\alpha')}}$ (*Manual* Eq. 9-19a) $= \sqrt{\frac{4(20.1\text{ kips/bolt})(1.45\text{ in.})}{0.90(4.00\text{ in.})(65\text{ ksi})[1+(0.797)(1.0)]}}$ $= 0.527\text{ in.} < t_f = 0.640\text{ in.}$ **o.k.**	$\alpha' = \frac{1}{\delta}\left(\frac{\beta}{1-\beta}\right) \le 1.0$ $= \frac{1}{0.797}\left(\frac{0.810}{1-0.810}\right) > 1.0$ $= 5.35 > 1.0$ Therefore, $\alpha' = 1.0$. $\Omega = 1.67$ $t_{min} = \sqrt{\frac{\Omega 4T_a b'}{p F_u (1+\delta\alpha')}}$ (*Manual* Eq. 9-19b) $= \sqrt{\frac{1.67(4)(13.4\text{ kips/bolt})(1.45\text{ in.})}{(4.00\text{ in.})(65\text{ ksi})[1+(0.797)(1.0)]}}$ $= 0.527\text{ in.} < t_f = 0.640\text{ in.}$ **o.k.**

Note: As an alternative to the preceding calculations, the designer can use a simplified procedure to select a WT hanger with a flange thick enough to eliminate prying action. Assuming *b*′ = 1.45 in., the required thickness to eliminate prying action is determined from AISC *Manual* Equation 9-17a or 9-17b, as follows:

LRFD	ASD
$\phi = 0.90$ $t_{np} = \sqrt{\frac{4T_u b'}{\phi p F_u}}$ $= \sqrt{\frac{4(20.1\text{ kips/bolt})(1.45\text{ in.})}{0.90(4.00\text{ in.})(65\text{ ksi})}}$ $= 0.706\text{ in.}$	$\Omega = 1.67$ $t_{np} = \sqrt{\frac{\Omega 4T_a b'}{p F_u}}$ $= \sqrt{\frac{1.67(4)(13.4\text{ kips/bolt})(1.45\text{ in.})}{(4.00\text{ in.})(65\text{ ksi})}}$ $= 0.707\text{ in.}$

The WT6×25 that was selected does not have a sufficient flange thickness to reduce the effect of prying action to an insignificant amount. In this case, the simplified approach requires a WT section with a thicker flange.

Tensile Yielding of the WT Stem on the Whitmore Section

As shown in AISC *Manual* Figure 9-1, the Whitmore section defines the effective width of the WT stem. Note that the Whitmore section cannot exceed the actual 8 in. width of the WT.

$$\begin{aligned} l_w &= 3.00\text{ in.} + 2\left(4.00\text{ in.}\right)\left(\tan 30^\circ\right) \le 8.00\text{ in.} \\ &= 7.62\text{ in.} < 8.00\text{ in.} \end{aligned}$$

Therefore:

$$l_w = 7.62\text{ in.}$$

From AISC *Specification* Section J4.1(a), the available tensile yielding strength of the WT stem is determined as follows:

$$\begin{aligned} A_g &= l_w t_w \\ &= \left(7.62\text{ in.}\right)\left(0.370\text{ in.}\right) \\ &= 2.82\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= F_y A_g \\ &= \left(50\text{ ksi}\right)\left(2.82\text{ in.}^2\right) \\ &= 141\text{ kips} \end{aligned}$$ (*Spec*. Eq. J4-1)

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90\left(141\text{ kips}\right)$ $= 127\text{ kips} > 80.2\text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{141\text{ kips}}{1.67}$ $= 84.4\text{ kips} > 53.5\text{ kips}$ **o.k.**

Shear Rupture of the WT Stem Base Metal

From AISC *Specification* Section J4.2(b), the available shear rupture strength of the WT stem at the welds is determined as follows:

$$\begin{aligned} R_n &= \left(2\text{ welds}\right)\left(2\text{ planes}\right)0.60F_u l_w t_w \\ &= \left(2\text{ welds}\right)\left(2\text{ planes}\right)\left(0.60\right)\left(65\text{ ksi}\right)\left(4\text{ in.}\right)\left(0.370\text{ in.}\right) \\ &= 231\text{ kips} \end{aligned}$$ (from *Spec*. Eq. J4-4)

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75\left(231\text{ kips}\right)$ $= 173\text{ kips} > 80.2\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{231\text{ kips}}{2.00}$ $= 116\text{ kips} > 53.5\text{ kips}$ **o.k.**

Block Shear Rupture of the WT Stem

The available strength for the limit state of block shear rupture of the stem is determined as follows.

$$R_n = 0.60F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.60F_y A_{gv} + U_{bs} F_u A_{nt}$$ (*Spec*. Eq. J4-5)

where

$$\begin{aligned} A_{gv} &= A_{nv} \\ &= (2\text{ lines})lt_w \\ &= (2\text{ lines})(4\text{ in.})(0.370\text{ in.}) \\ &= 2.96\text{ in.}^2 \\ A_{nt} &= (leg)t_w \\ &= (3\text{ in.})(0.370\text{ in.}) \\ &= 1.11\text{ in.}^2 \\ U_{bs} &= 1.0 \end{aligned}$$

and

$$\begin{aligned} R_n &= 0.60(65\text{ ksi})\left(2.96\text{ in.}^2\right) + 1.0(65\text{ ksi})\left(1.11\text{ in.}^2\right) \le 0.60(50\text{ ksi})\left(2.96\text{ in.}^2\right) + 1.0(65\text{ ksi})\left(1.11\text{ in.}^2\right) \\ &= 188\text{ kips} > 161\text{ kips} \end{aligned}$$

Therefore:

$$R_n = 161\text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(161\text{ kips})$ $= 121\text{ kips} > 80.2\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{161\text{ kips}}{2.00}$ $= 80.5\text{ kips} > 53.5\text{ kips}$ **o.k.**

The final connection design is shown in Figure II.D-1-1.

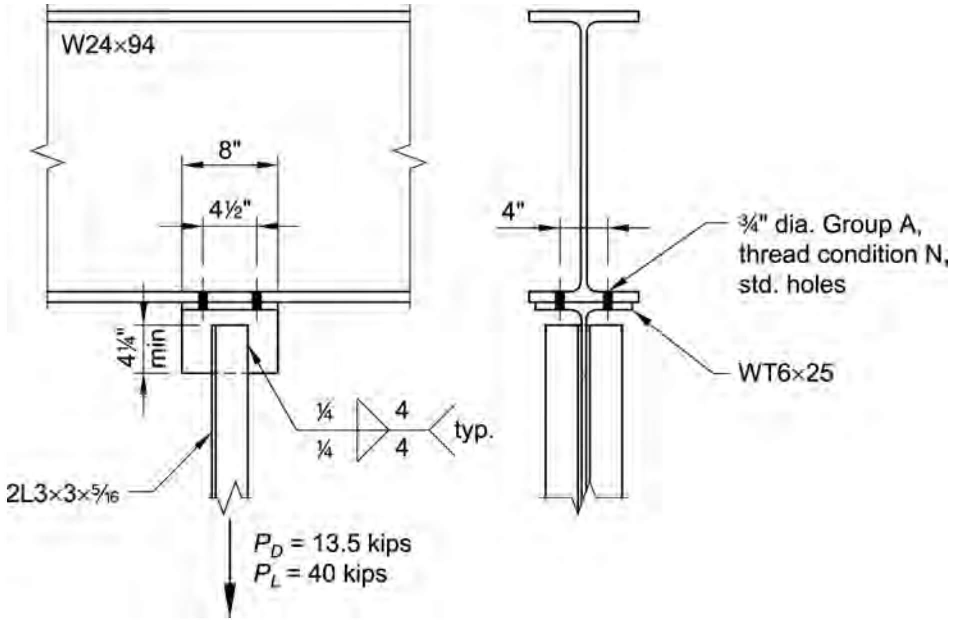

Fig. II.D-1-1. Final hanger design for Example II.D-1

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EXAMPLE II.D-2 BEAM BEARING PLATE

Given:

An ASTM A992 W18×50 beam supported by a 10-in.-thick concrete wall, as shown in Figure II.D-2-1, has the following end reactions:

R_D = 15 kips
R_L = 45 kips

Verify the following:

A. If a bearing plate is required when the beam is supported by the full wall thickness ($l_b = h$ = 10 in)
B. The bearing plate required if $l_b = h$ = 10 in. (the full wall thickness)
C. The bearing plate required if l_b = 6½ in. and the bearing plate is centered on the thickness of the wall

The concrete has f_c' = 3 ksi and the bearing plate is ASTM A36 material.

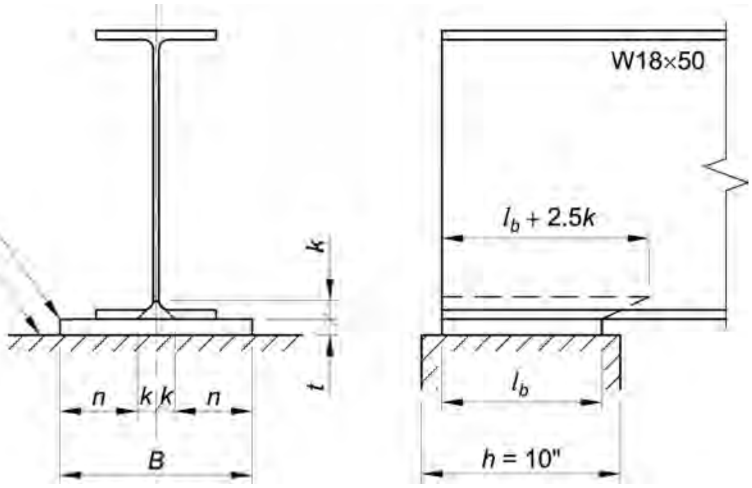

Fig. II.D-2-1. Connection geometry for Example II.D-2.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Beam
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Bearing plate
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

Concrete wall
f_c' = 3 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W18×50
d = 18.0 in.
t_w = 0.355 in.
b_f = 7.50 in.
t_f = 0.570 in.
k_{des} = 0.972 in.
k_1 = 13⁄16 in.

From ASCE/SEI, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(15\text{ kips}) + 1.6(45\text{ kips})$ $= 90.0\text{ kips}$	$R_a = 15\text{ kips} + 45\text{ kips}$ $= 60.0\text{ kips}$

Solution A:

Required Bearing Length

The required bearing length for the limit state of web local yielding is determined using AISC *Manual* Table 9-4 and AISC *Manual* Equation 9-46a or 9-46b, as follows:

LRFD	ASD
$\phi R_1 = 43.1\text{ kips}$ $\phi R_2 = 17.8\text{ kip/in.}$ $l_{b\,min} = \dfrac{R_u - \phi R_1}{\phi R_2} \geq k_{des}$ $= \dfrac{90.0\text{ kips} - 43.1\text{ kips}}{17.8\text{ kip/in.}} > 0.972\text{ in.}$ $= 2.63\text{ in.} > 0.972\text{ in.}$ Therefore: $l_{b\,min} = 2.63\text{ in.} < 10.0\text{ in.}$ **o.k.**	$R_1/\Omega = 28.8\text{ kips}$ $R_2/\Omega = 11.8\text{ kip/in.}$ $l_{b\,min} = \dfrac{R_a - R_1/\Omega}{R_2/\Omega} \geq k_{des}$ $= \dfrac{60.0\text{ kips} - 28.8\text{ kips}}{11.8\text{ kip/in.}} > 0.972\text{ in.}$ $= 2.64\text{ in.} > 0.972\text{ in.}$ Therefore: $l_{b\,min} = 2.64\text{ in.} < 10.0\text{ in.}$ **o.k.**

The required bearing length for the limit state of web local crippling is determined using AISC *Manual* Table 9-4.

$$\frac{l_b}{d} = \frac{10.0\text{ in.}}{18.0\text{ in.}}$$
$$= 0.556$$

Because $\dfrac{l_b}{d} > 0.2$, use AISC *Manual* Table 9-4 and AISC *Manual* Equation 9-49a or 9-49b, as follows:

LRFD	ASD
$\phi R_5 = 52.0\text{ kips}$ $\phi R_6 = 6.30\text{ kip/in.}$	$R_5/\Omega = 34.7\text{ kips}$ $R_6/\Omega = 4.20\text{ kip/in.}$

LRFD	ASD
$l_{b\ min} = \dfrac{R_u - \phi R_5}{\phi R_6} \geq k_{des}$ $= \dfrac{90.0\text{ kips} - 52.0\text{ kips}}{6.30\text{ kip/in.}} > 0.972\text{ in.}$ $= 6.03\text{ in.} > 0.972\text{ in.}$ Therefore: $l_{b\ min} = 6.03\text{ in.} < 10.0\text{ in.}$ **o.k.** Verify $\dfrac{l_b}{d} > 0.2$: $\dfrac{l_b}{d} = \dfrac{6.03\text{ in.}}{18.0\text{ in.}}$ $= 0.335 > 0.2$ **o.k.**	$l_{b\ min} = \dfrac{R_a - R_5/\Omega}{R_6/\Omega} \geq k_{des}$ $= \dfrac{60.0\text{ kips} - 34.7\text{ kips}}{4.20\text{ kip/in.}} > 0.972\text{ in.}$ $= 6.02\text{ in.} > 0.972\text{ in.}$ Therefore: $l_{b\ min} = 6.02\text{ in.} < 10.0\text{ in.}$ **o.k.** Verify $\dfrac{l_b}{d} > 0.2$: $\dfrac{l_b}{d} = \dfrac{6.02\text{ in.}}{18.0\text{ in.}}$ $= 0.334 > 0.2$ **o.k.**

The bearing strength of the concrete is determined from AISC *Specification* Section J8. Note that AISC *Specification* Equation J8-1 is used because A_2 is not larger than A_1 in this case.

$$
\begin{aligned}
A_1 &= b_f l_b \\
&= (7.50\text{ in.})(10.0\text{ in.}) \\
&= 75.0\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
P_p &= 0.85 f_c' A_1 \\
&= 0.85(3\text{ ksi})\left(75.0\text{ in.}^2\right) \\
&= 191\text{ kips}
\end{aligned}
$$

(*Spec*. Eq. J8-1)

LRFD	ASD
$\phi_c = 0.65$ $\phi_c P_p = 0.65(191\text{ kips})$ $= 124\text{ kips} > 90.0\text{ kips}$ **o.k.**	$\Omega_c = 2.31$ $\dfrac{P_p}{\Omega_c} = \dfrac{191\text{ kips}}{2.31}$ $= 82.7\text{ kips} > 60.0\text{ kips}$ **o.k.**

Beam Flange Thickness

Using the cantilever length from AISC *Manual* Part 14, determine the minimum beam flange thickness required if no bearing plate is provided. The beam flanges along the length, n, are assumed to be fixed end cantilevers with a minimum thickness determined using the limit state of flexural yielding.

$$
\begin{aligned}
n &= \frac{b_f}{2} - k_{des} \\
&= \frac{7.50\text{ in.}}{2} - 0.972\text{ in.} \\
&= 2.78\text{ in.}
\end{aligned}
$$

(from *Manual* Eq. 14-1)

LRFD	ASD
The bearing pressure is determined as follows: $f_p = \dfrac{R_u}{A_1}$	The bearing pressure is determined as follows: $f_p = \dfrac{R_a}{A_1}$
The required flexural strength of the flange is: $M_u = \dfrac{f_p n^2}{2}$ $= \dfrac{R_u n^2}{2A_1}$	The required flexural strength of the flange is: $M_a = \dfrac{f_p n^2}{2}$ $= \dfrac{R_a n^2}{2A_1}$
The available flexural strength of the flange is: $\phi = 0.90$ $\phi M_n = \phi F_y Z$ $= \phi F_y \left(\dfrac{t_f{}^2}{4}\right)$	The available flexural strength of the flange is: $\Omega = 1.67$ $\dfrac{M_n}{\Omega} = \dfrac{F_y Z}{\Omega}$ $= \dfrac{F_y}{\Omega}\left(\dfrac{t_f{}^2}{4}\right)$
For $\phi R_n = R_u$ and solving for t_f, the minimum flange thickness is determined as follows: $t_{f\ min} = \sqrt{\dfrac{2R_u n^2}{\phi A_1 F_y}}$ $= \sqrt{\dfrac{2(90.0\text{ kips})(2.78\text{ in.})^2}{0.90(75.0\text{ in.}^2)(50\text{ ksi})}}$ $= 0.642\text{ in.} > t_f = 0.570\text{ in.}$ **n.g.**	For $R_n/\Omega = R_a$ and solving for t_f, the minimum flange thickness is determined as follows: $t_{f\ min} = \sqrt{\dfrac{\Omega 2R_a n^2}{A_1 F_y}}$ $= \sqrt{\dfrac{1.67(2)(60.0\text{ kips})(2.78\text{ in.})^2}{(75.0\text{ in.}^2)(50\text{ ksi})}}$ $= 0.643\text{ in.} > t_f = 0.570\text{ in.}$ **n.g.**
Therefore, a bearing plate is required.	Therefore, a bearing plate is required.

Note: The designer may assume a bearing width narrower than the beam flange to justify a thinner flange. In this case, the bearing width is constrained by the lower bound concrete bearing strength and the upper bound 0.570-in. flange thickness.

$$5.43\text{ in.} \le \text{bearing width} \le 6.56\text{ in.}$$

Solution B:

Bearing Length

From Solution A, with l_b = 10 in., the web local yielding and web local crippling strengths for the beam are adequate.

Bearing Plate Design

The required bearing plate width is determined using AISC *Specification* Equation J8-1 as follows:

LRFD	ASD
$\phi_c = 0.65$	$\Omega_c = 2.31$
$A_{1\,req} = \dfrac{R_u}{\phi_c 0.85 f_c'}$ $= \dfrac{90.0\text{ kips}}{0.65(0.85)(3\text{ ksi})}$ $= 54.3\text{ in.}^2$	$A_{1\,req} = \dfrac{R_a \Omega_c}{0.85 f_c'}$ $= \dfrac{(60.0\text{ kips})(2.31)}{0.85(3\text{ ksi})}$ $= 54.4\text{ in.}^2$
$B_{req} = \dfrac{A_{1\,req}}{l_b}$ $= \dfrac{54.3\text{ in.}^2}{10.0\text{ in.}}$ $= 5.43\text{ in.}$	$B_{req} = \dfrac{A_{1\,req}}{l_b}$ $= \dfrac{54.4\text{ in.}^2}{10.0\text{ in.}}$ $= 5.44\text{ in.}$
Use B = 8 in. (selected as the least whole-inch dimension that exceeds b_f).	Use B = 8 in. (selected as the least whole-inch dimension that exceeds b_f).

From AISC *Manual* Part 14, the bearing plate cantilever dimension is determined as follows:

$$n = \frac{B}{2} - k_{des} \qquad \text{(\textit{Manual} Eq. 14-1)}$$

$$= \frac{8\text{ in.}}{2} - 0.972\text{ in.}$$

$$= 3.03\text{ in.}$$

The required thickness of the base plate is determined using the available flexural strength equation previously derived for the required beam flange thickness.

LRFD	ASD
$t_{min} = \sqrt{\dfrac{2R_u n^2}{\phi F_y B l_b}}$ $= \sqrt{\dfrac{2(90.0\text{ kips})(3.03\text{ in.})^2}{0.90(36\text{ ksi})(8\text{ in.})(10\text{ in.})}}$ $= 0.798\text{ in.}$	$t_{min} = \sqrt{\dfrac{\Omega 2 R_a n^2}{F_y B l_b}}$ $= \sqrt{\dfrac{1.67(2)(60.0\text{ kips})(3.03\text{ in.})^2}{(36\text{ ksi})(8\text{ in.})(10\text{ in.})}}$ $= 0.799\text{ in.}$
Use PL⅞ in.×10 in.×0 ft 8 in.	Use PL⅞ in.×10 in.×0 ft 8 in.

Note: The calculations for t_{min} are conservative. Taking the strength of the beam flange into consideration results in a thinner required bearing plate or no bearing plate at all.

Solution C:

From Solution A, with l_b = 6½ in., the web local yielding and web local crippling strengths for the beam are adequate.

Bearing Plate Design

Try B = 8 in.

$$\begin{aligned}A_1 &= Bl_b\\ &= (8\text{ in.})(6\tfrac{1}{2}\text{ in.})\\ &= 52.0\text{ in.}^2\end{aligned}$$

AISC *Specification* Section J8 requires that the area, A_2, be geometrically similar to A_1.

$$\begin{aligned}N_1 &= 6\tfrac{1}{2}\text{ in.} + 2(1.75\text{ in.})\\ &= 10.0\text{ in.}\end{aligned}$$

$$\begin{aligned}B_1 &= 8\text{ in.} + 2(1.75\text{ in.})\\ &= 11.5\text{ in.}\end{aligned}$$

$$\begin{aligned}A_2 &= B_1N_1\\ &= (11.5\text{ in.})(10.0\text{ in.})\\ &= 115\text{ in.}^2\end{aligned}$$

The bearing strength of the concrete is determined from AISC *Specification* Section J8. Note that AISC *Specification* Equation J8-2 is used because A_2 is larger than A_1 in this case.

$$\begin{aligned}P_p &= 0.85 f_c{}' A_1\sqrt{A_2/A_1} \le 1.7 f_c{}' A_1 \qquad \text{(\textit{Spec}. Eq. J8-2)}\\ &= 0.85(3\text{ ksi})\left(52.0\text{ in.}^2\right)\sqrt{115\text{ in.}^2/52.0\text{ in.}^2} \le 1.7(3\text{ ksi})\left(52.0\text{ in.}^2\right)\\ &= 197\text{ kips} < 265\text{ kips}\end{aligned}$$

Therefore:

$$P_p = 197\text{ kips}$$

LRFD	ASD
$\phi_c = 0.65$ $\begin{aligned}\phi_c P_p &= 0.65(197\text{ kips})\\ &= 128\text{ kips} > 90.0\text{ kips} \quad \textbf{o.k.}\end{aligned}$	$\Omega_c = 2.31$ $\begin{aligned}\frac{P_p}{\Omega_c} &= \frac{197\text{ kips}}{2.31}\\ &= 85.3\text{ kips} > 60.0\text{ kips} \quad \textbf{o.k.}\end{aligned}$

From AISC *Manual* Part 14, the bearing plate cantilever dimension is determined as follows:

$$\begin{aligned}n &= \frac{B}{2} - k_{des} \qquad \text{(\textit{Manual} Eq. 14-1)}\\ &= \frac{8\text{ in.}}{2} - 0.972\text{ in.}\\ &= 3.03\text{ in.}\end{aligned}$$

The required thickness of the base plate is determined using the available flexural strength equation previously derived for the required beam flange thickness.

LRFD	ASD
$t_{min} = \sqrt{\dfrac{2R_u n^2}{\phi F_y B l_b}}$ $= \sqrt{\dfrac{2(90.0\text{ kips})(3.03\text{ in.})^2}{0.90(36\text{ ksi})(8\text{ in.})(6\tfrac{1}{2}\text{ in.})}}$ $= 0.990$ in. Use PL1 in.×6½ in.×0 ft 8 in.	$t_{min} = \sqrt{\dfrac{\Omega 2R_a n^2}{F_y B l_b}}$ $= \sqrt{\dfrac{1.67(2)(60.0\text{ kips})(3.03\text{ in.})^2}{(36\text{ ksi})(8\text{ in.})(6\tfrac{1}{2}\text{ in.})}}$ $= 0.991$ in. Use PL1 in.×6½ in.×0 ft 8 in

Note: The calculations for t_{min} are conservative. Taking the strength of the beam flange into consideration results in a thinner required bearing plate or no bearing plate at all.

EXAMPLE II.D-3 SLIP-CRITICAL CONNECTION WITH OVERSIZED HOLES

Given:

Verify the connection of an ASTM A36 2L3×3×5⁄16 tension member to an ASTM A36 plate welded to an ASTM A992 beam, as shown in Figure II.D-3-1, for the following loads:

P_D = 15 kips
P_L = 45 kips

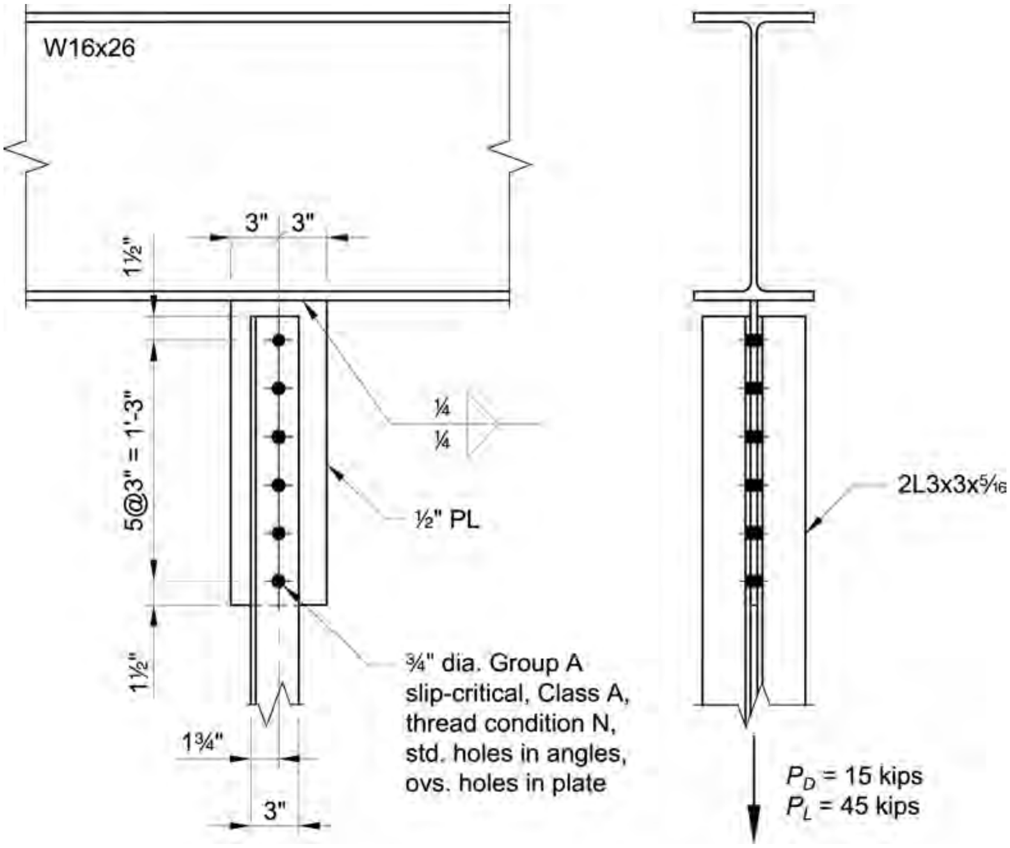

Fig. II.D-3-1. Connection configuration for Example II.D-3.

Solution:

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

Beam
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Hanger and plate
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Tables 1-1, 1-7 and 1-15, the geometric properties are as follows:

Beam
W16×26
t_f = 0.345 in.
t_w = 0.250 in.
k_{des} = 0.747 in.

Hanger
2L3×3×5⁄16
A = 3.56 in.^{2}
$\bar{x} = 0.860$ in. for single angle

From AISC *Specification* Table J3.3, the hole diameter for ¾-in.-diameter bolts with standard and oversized holes is:

d_h = 13⁄16 in. (standard hole)
d_h = 15⁄16 in. (oversized hole)

From ASCE/SEI 7, Chapter 2, the required strength is:

LRFD	ASD
$R_u = 1.2(15\text{ kips}) + 1.6(45\text{ kips})$ $= 90.0\text{ kips}$	$R_a = 15\text{ kips} + 45\text{ kips}$ $= 60.0\text{ kips}$

Bolt Slip Resistance Strength

From AISC *Manual* Table 7-3, with ¾-in.-diameter Group A slip-critical bolts with Class A faying surfaces in oversized holes and double shear, the available slip resistance strength is:

LRFD	ASD
$\phi r_n = 16.1\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 10.8\text{ kips/bolt}$

The required number of bolts is determined as follows:

LRFD	ASD
$n = \dfrac{R_u}{\phi r_n}$ $= \dfrac{90.0\text{ kips}}{16.1\text{ kips/bolt}}$ $= 5.59$ Therefore, use 6 bolts.	$n = \dfrac{R_a}{(r_n / \Omega)}$ $= \dfrac{60.0\text{ kips}}{10.8\text{ kips/bolt}}$ $= 5.56$ Therefore, use 6 bolts.

Strength of Bolted Connection—Angles

Slip-critical connections must also be designed for the limit states of bearing-type connections. From the Commentary to AISC *Specification* Section J3.6, the strength of the bolt group is taken as the sum of the individual strengths of the individual fasteners, which may be taken as the lesser of the fastener shear strength per AISC *Specification* Section J3.6, the bearing strength at the bolt hole per AISC *Specification* Section J3.10, or the tearout strength at the bolt hole per AISC *Specification* Section J3.10.

From AISC *Manual* Table 7-1, the available shear strength per bolt for ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) in double shear is:

LRFD	ASD
$\phi r_n = 35.8 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 23.9 \text{ kips/bolt}$

The available bearing and tearout strength of the angles using standard holes at the edge bolt is determined using AISC *Manual* Table 7-5, conservatively using l_e = 1¼ in.

LRFD	ASD
$\phi r_n = (2 \text{ angles})(44.0 \text{ kip/in.})(\tfrac{5}{16} \text{ in.})$ $= 27.5 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (2 \text{ angles})(29.4 \text{ kip/in.})(\tfrac{5}{16} \text{ in.})$ $= 18.4 \text{ kips/bolt}$

Therefore, the bearing or tearout strength controls over bolt shear at the edge bolts.

The available bearing and tearout strength of the angles using standard holes at the other bolts is determined using AISC *Manual* Table 7-4 with s = 3 in.

LRFD	ASD
$\phi r_n = (2 \text{ angles})(78.3 \text{ kip/in.})(\tfrac{5}{16} \text{ in.})$ $= 48.9 \text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (2 \text{ angles})(52.2 \text{ kip/in.})(\tfrac{5}{16} \text{ in.})$ $= 32.6 \text{ kips/bolt}$

Therefore, bolt shear controls over bearing or tearout at the other bolts.

The strength of the bolt group in the angles is determined by summing the strength of the individual fasteners as follows:

LRFD	ASD
$\phi R_n = (1 \text{ bolt})(27.5 \text{ kips/bolt})$ $+ (5 \text{ bolts})(35.8 \text{ kips/bolt})$ $= 207 \text{ kips} > 90.0 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = (1 \text{ bolt})(18.4 \text{ kips/bolt})$ $+ (5 \text{ bolts})(23.9 \text{ kips/bolt})$ $= 138 \text{ kips} > 60.0 \text{ kips}$ **o.k.**

Tensile Strength of the Angles

From AISC *Specification* Section J4.1(a), the available tensile yielding strength of the angles is determined as follows:

$$
\begin{aligned}
P_n &= F_y A_g \qquad \text{(}\textit{Spec}\text{. Eq. J4-1)}\\
&= (36 \text{ ksi})(3.56 \text{ in.}^2)\\
&= 128 \text{ kips}
\end{aligned}
$$

LRFD	ASD
$\phi = 0.90$ $\phi P_n = 0.90(128 \text{ kips})$ $= 115 \text{ kips} > 90.0 \text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{P_n}{\Omega} = \dfrac{128 \text{ kips}}{1.67}$ $= 76.6 \text{ kips} > 60.0 \text{ kips}$ **o.k.**

From AISC *Specification* Section J4.1(b), the available tensile rupture strength of the angles is determined as follows. The shear lag factor, *U*, is determined using AISC *Specification* Table D3.1, Case 2.

$$\begin{aligned} U &= 1 - \frac{\bar{x}}{l} \\ &= 1 - \frac{0.860 \text{ in.}}{15.0 \text{ in.}} \\ &= 0.943 \end{aligned}$$

$$\begin{aligned} A_e &= A_n U \\ &= \left[A_g - (2 \text{ angles})\left(d_h + {}^{1\!}/_{16} \text{ in.}\right)t\right]U \\ &= \left[3.56 \text{ in.}^2 - (2 \text{ angles})\left({}^{13\!}/_{16} \text{ in.} + {}^{1\!}/_{16} \text{ in.}\right)\left({}^{5\!}/_{16} \text{ in.}\right)\right](0.943) \\ &= 2.84 \text{ in.}^2 \end{aligned} \qquad (Spec.\ \text{Eq. D3-1})$$

$$\begin{aligned} P_n &= F_u A_e \\ &= (58 \text{ ksi})\left(2.84 \text{ in.}^2\right) \\ &= 165 \text{ kips} \end{aligned} \qquad (Spec.\ \text{Eq. J4-2})$$

LRFD	ASD
$\phi = 0.75$ $\phi P_n = 0.75(165 \text{ kips})$ $= 124 \text{ kips} > 90.0 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{P_n}{\Omega} = \dfrac{165 \text{ kips}}{2.00}$ $= 82.5 \text{ kips} > 60.0 \text{ kips}$ **o.k.**

Block Shear Rupture Strength of the Angles

The available strength for the limit state of block shear rupture of the angles is determined as follows:

$$R_n = 0.60 F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.60 F_y A_{gv} + U_{bs} F_u A_{nt} \qquad (Spec.\ \text{Eq. J4-5})$$

where

$$\begin{aligned} A_{gv} &= (2 \text{ angles})\left[l_{ev} + (n-1)s\right]t \\ &= (2 \text{ angles})\left[1\tfrac{1}{2} \text{ in.} + (6-1)(3 \text{ in.})\right]\left({}^{5\!}/_{16} \text{ in.}\right) \\ &= 10.3 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nv} &= A_{gv} - (2 \text{ angles})(n - 0.5)\left(d_h + {}^{1\!}/_{16} \text{ in.}\right)t \\ &= 10.3 \text{ in.}^2 - (2 \text{ angles})(6 - 0.5)\left({}^{13\!}/_{16} \text{ in.} + {}^{1\!}/_{16} \text{ in.}\right)\left({}^{5\!}/_{16} \text{ in.}\right) \\ &= 7.29 \text{ in.}^2 \end{aligned}$$

$$\begin{aligned} A_{nt} &= (2\text{ angles})\left[l_{eh} - 0.5\left(d_h + {}^{1}\!/_{16}\text{ in.}\right)\right]t \\ &= (2\text{ angles})\left[1{}^{1}\!/_{4}\text{ in.} - 0.5\left({}^{13}\!/_{16}\text{ in.} + {}^{1}\!/_{16}\text{ in.}\right)\right]\left({}^{5}\!/_{16}\text{ in.}\right) \\ &= 0.508\text{ in.}^2 \end{aligned}$$

$$U_{bs} = 1.0$$

and

$$\begin{aligned} R_n &= 0.60(58\text{ ksi})\left(7.29\text{ in.}^2\right) + 1.0(58\text{ ksi})\left(0.508\text{ in.}^2\right) \le 0.60(36\text{ ksi})\left(10.3\text{ in.}^2\right) + 1.0(58\text{ ksi})\left(0.508\text{ in.}^2\right) \\ &= 283\text{ kips} > 252\text{ kips} \end{aligned}$$

Therefore:

$$R_n = 252\text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(252\text{ kips})$ $= 189\text{ kips} > 90.0\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{252\text{ kips}}{2.00}$ $= 126\text{ kips} > 60.0\text{ kips}$ **o.k.**

Strength of Bolted Connection—Plate

From AISC *Manual* Table 7-1, the available shear strength per bolt for ¾-in.-diameter Group A bolts with threads not excluded from the shear plane (thread condition N) in double shear is:

LRFD	ASD
$\phi r_n = 35.8\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = 23.9\text{ kips/bolt}$

The available bearing and tearout strength of the plate using oversized holes at the edge bolt is determined using AISC *Manual* Table 7-5, conservatively using l_e = 1¼ in.

LRFD	ASD
$\phi r_n = (40.8\text{ kip/in.})\left({}^{1}\!/_{2}\text{ in.}\right)$ $= 20.4\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (27.2\text{ kip/in.})\left({}^{1}\!/_{2}\text{ in.}\right)$ $= 13.6\text{ kips/bolt}$

Therefore, the bearing or tearout strength controls over bolt shear at the edge bolts.

The available bearing and tearout strength of the plate using oversized holes at the other bolts is determined using AISC *Manual* Table 7-4 with s = 3 in.

LRFD	ASD
$\phi r_n = (78.3\text{ kip/in.})\left({}^{1}\!/_{2}\text{ in.}\right)$ $= 39.2\text{ kips/bolt}$	$\dfrac{r_n}{\Omega} = (52.2\text{ kip/in.})\left({}^{1}\!/_{2}\text{ in.}\right)$ $= 26.1\text{ kips/bolt}$

Therefore, bolt shear controls over bearing or tearout at the other bolts.

The strength of the bolt group in the plate is determined by summing the strength of the individual fasteners as follows:

LRFD	ASD
$\phi R_n = (1\text{ bolt})(20.4\text{ kips/bolt})$ $\quad + (5\text{ bolts})(35.8\text{ kips/bolt})$ $= 199\text{ kips} > 90.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = (1\text{ bolt})(13.6\text{ kips/bolt})$ $\quad + (5\text{ bolts})(23.9\text{ kips/bolt})$ $= 133\text{ kips} > 60.0\text{ kips}$ **o.k.**

Tensile Strength of the Plate

From AISC *Specification* Section J4.1(a), the available tensile yielding strength of the plate is determined as follows. By inspection, the Whitmore section, as defined in AISC *Manual* Figure 9-1, includes the entire width of the ½-in. plate.

$$\begin{aligned} A_g &= bt \\ &= (6\text{ in.})(\tfrac{1}{2}\text{ in.}) \\ &= 3.00\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} R_n &= F_y A_g \\ &= (36\text{ ksi})(3.00\text{ in.}^2) \\ &= 108\text{ kips} \end{aligned}$$

(*Spec*. Eq. J4-1)

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(108\text{ kips})$ $= 97.2\text{ kips} > 90.0\text{ kips}$ **o.k.**	$\Omega = 1.67$ $\dfrac{P_n}{\Omega} = \dfrac{108\text{ kips}}{1.67}$ $= 64.7\text{ kips} > 60.0\text{ kips}$ **o.k.**

From AISC *Specification* Section J4.1(b), the available tensile rupture strength of the plate is determined as follows:

$$\begin{aligned} A_n &= A_g - (d_h + \tfrac{1}{16}\text{ in.})t \\ &= 3.00\text{ in.}^2 - (\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})(\tfrac{1}{2}\text{ in.}) \\ &= 2.50\text{ in.}^2 \end{aligned}$$

AISC *Specification* Table D3.1, Case 1, applies in this case because tension load is transmitted directly to the cross-sectional element by fasteners; therefore, $U = 1.0$.

$$\begin{aligned} A_e &= A_n U \\ &= (2.50\text{ in.}^2)(1.0) \\ &= 2.50\text{ in.}^2 \end{aligned}$$

(*Spec*. Eq. D3-1)

$$
\begin{aligned}
R_n &= F_u A_e \qquad \text{(}\textit{Spec}\text{. Eq. J4-2)}\\
&= (58\text{ ksi})(2.50\text{ in.}^2)\\
&= 145\text{ kips}
\end{aligned}
$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(145\text{ kips})$ $= 109\text{ kips} > 90.0\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{P_n}{\Omega} = \dfrac{145\text{ kips}}{2.00}$ $= 72.5\text{ kips} > 60.0\text{ kips}$ **o.k.**

Block Shear Rupture Strength of the Plate

The available strength for the limit state of block shear rupture of the plate is determined as follows.

$$R_n = 0.60F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.60F_y A_{gv} + U_{bs} F_u A_{nt} \qquad \text{(}\textit{Spec}\text{. Eq. J4-5)}$$

where

$$
\begin{aligned}
A_{gv} &= \left[l_{ev} + (n-1)s\right]t\\
&= \left[1\tfrac{1}{2}\text{ in.} + (6-1)(3\text{ in.})\right](\tfrac{1}{2}\text{ in.})\\
&= 8.25\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
A_{nv} &= A_{gv} - (n-0.5)(d_h + \tfrac{1}{16}\text{ in.})t\\
&= 8.25\text{ in.}^2 - (6-0.5)(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})(\tfrac{1}{2}\text{ in.})\\
&= 5.50\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
A_{nt} &= \left[l_{eh} - 0.5(d_h + \tfrac{1}{16}\text{ in.})\right]t\\
&= \left[3\text{ in.} - 0.5(\tfrac{15}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})\right](\tfrac{1}{2}\text{ in.})\\
&= 1.25\text{ in.}^2
\end{aligned}
$$

$$U_{bs} = 1.0$$

and

$$
\begin{aligned}
R_n &= 0.60(58\text{ ksi})(5.50\text{ in.}^2) + 1.0(58\text{ ksi})(1.25\text{ in.}^2) \le 0.60(36\text{ ksi})(8.25\text{ in.}^2) + 1.0(58\text{ ksi})(1.25\text{ in.}^2)\\
&= 264\text{ kips} > 251\text{ kips}
\end{aligned}
$$

Therefore:

$$R_n = 251\text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(251\text{ kips})$ $= 188\text{ kips} > 90.0\text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{251\text{ kips}}{2.00}$ $= 126\text{ kips} > 60.0\text{ kips}$ **o.k.**

Plate-to-Beam Weld

The applied load is perpendicular to the weld length ($\theta = 90°$), therefore the directional strength factor is determined from AISC *Specification* Equation J2-5. This increase factor due to directional strength is incorporated into the weld strength calculation.

$$\begin{aligned}1.0 + 0.50\sin^{1.5}\theta &= 1.0 + 0.50\sin^{1.5}(90°)\\ &= 1.50\end{aligned}$$

The required fillet weld size is determined using AISC *Manual* Equation 8-2a or 8-2b, as follows:

LRFD	ASD
$D_{req} = \dfrac{P_u}{(2\text{ welds})(1.50)(1.392\text{ kip/in.})l}$ $= \dfrac{90.0\text{ kips}}{(2\text{ welds})(1.50)(1.392\text{ kip/in.})(6\text{ in.})}$ $= 3.59$ Use ¼-in. fillet welds on each side of the plate.	$D_{req} = \dfrac{P_a}{(2\text{ welds})(1.50)(0.928\text{ kip/in.})l}$ $= \dfrac{60.0\text{ kips}}{(2\text{ welds})(1.50)(0.928\text{ kip/in.})(6\text{ in.})}$ $= 3.59$ Use ¼-in. fillet welds on each side of the plate.

From AISC *Manual* Table J2.4, the minimum fillet weld size is:

$w_{min} = \tfrac{3}{16}\text{ in.} < \tfrac{1}{4}\text{ in.}$ **o.k.**

Beam Flange Base Metal Check

The minimum flange thickness to match the required shear rupture strength of the welds is determined as follows:

$$\begin{aligned}t_{min} &= \frac{3.09D}{F_u}\\ &= \frac{3.09(3.59)}{65\text{ ksi}}\\ &= 0.171\text{ in.} < 0.345\text{ in.} \quad \textbf{o.k.}\end{aligned}$$

(*Manual* Eq. 9-2)

Beam Concentrated Forces Check

From AISC *Specification* Section J10.2, the beam web is checked for the limit state of web local yielding assuming the connection is at a distance from the member end greater than the depth of the member, *d*.

$$\begin{aligned}R_n &= F_{yw}t_w(5k_{des} + l_b)\\ &= (50\text{ ksi})(0.250\text{ in.})[5(0.747\text{ in.}) + 6\text{ in.}]\\ &= 122\text{ kips}\end{aligned}$$

(*Spec*. Eq. J10-2)

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(122\text{ kips})$ $= 122\text{ kips} > 90.0\text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{122\text{ kips}}{1.50}$ $= 81.3\text{ kips} > 60.0\text{ kips}$ **o.k.**

Conclusion

The connection is found to be adequate as given for the applied loads.

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Part III
System Design Examples

EXAMPLE III-1 DESIGN OF SELECTED MEMBERS AND LATERAL ANALYSIS OF A FOUR-STORY BUILDING

INTRODUCTION

This section illustrates the load determination and selection of representative members that are part of the gravity and lateral frame of a typical four-story building. The design is completed in accordance with the AISC *Specification* and AISC *Manual*. Loading criteria are based on ASCE/SEI 7.

This section includes:

- Analysis and design of a typical steel frame for gravity loads
- Analysis and design of a typical steel frame for lateral loads
- Examples illustrating three methods for satisfying the stability provisions of AISC *Specification* Chapter C

The building being analyzed in this design example is located in a Midwestern city with moderate wind and seismic loads. The loads are given in the description of the design example. All members are ASTM A992 material.

CONVENTIONS

The following conventions are used throughout this example:

1. Beams or columns that have similar, but not necessarily identical, loads are grouped together. This is done because such grouping is generally a more economical practice for design, fabrication and erection.

2. Certain calculations, such as design loads for snow drift, which might typically be determined using a spreadsheet or structural analysis program, are summarized and then incorporated into the analysis. This simplifying feature allows the design example to illustrate concepts relevant to the member selection process.

3. Two commonly used deflection calculations, for uniform loads, have been rearranged so that the conventional units in the problem can be directly inserted into the equation for design. They are as follows:

   Simple beam:

$$\Delta = \frac{5\left(w \text{ kip/in.}\right)\left(L \text{ in.}\right)^4}{384\left(29{,}000 \text{ ksi}\right)\left(I \text{ in.}^4\right)}$$
$$= \frac{\left(w \text{ kip/ft}\right)\left(L \text{ ft}\right)^4}{1{,}290\left(I \text{ in.}^4\right)}$$

   Beam fixed at both ends:

$$\Delta = \frac{\left(w \text{ kip/in.}\right)\left(L \text{ in.}\right)^4}{384\left(29{,}000 \text{ ksi}\right)\left(I \text{ in.}^4\right)}$$
$$= \frac{\left(w \text{ kip/ft}\right)\left(L \text{ ft}\right)^4}{6{,}440\left(I \text{ in.}^4\right)}$$

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DESIGN SEQUENCE

The design sequence is presented as follows:

1. General description of the building including geometry, gravity loads and lateral loads
2. Roof member design and selection
3. Floor member design and selection
4. Column design and selection for gravity loads
5. Wind load determination
6. Seismic load determination
7. Horizontal force distribution to the lateral frames
8. Preliminary column selection for the moment frames and braced frames
9. Seismic load application to lateral systems
10. Stability (P-Δ) analysis

GENERAL DESCRIPTION OF THE BUILDING

Geometry

The design example is a four-story building, consisting of seven bays at 30 ft in the east-west (numbered grids) direction and bays of 45 ft, 30 ft and 45 ft in the north-south (lettered grids) direction, as shown in Figure III-1. The floor-to-floor height for the four floors is 13 ft 6 in. and the height from the fourth floor to the roof (at the edge of the building) is 14 ft 6 in. Based on discussions with fabricators, the same column size will be used for the whole height of the building.

The plans of these floors and the roof are shown on Sheets S2.1 thru S2.3, found at the end of this Chapter. The exterior of the building is a ribbon window system with brick spandrels supported and back-braced with steel and infilled with metal studs. The spandrel wall extends 2 ft above the elevation of the edge of the roof. The window and spandrel system is shown on design drawing Sheet S4.1.

The roof system is 1½-in. metal deck on open web steel joists. The open web steel joists are supported on steel beams as shown on Sheet S2.3. The roof slopes to interior drains. The middle three bays have a 6-ft-tall screen wall around them and house the mechanical equipment and the elevator over run. This area has steel beams, in place of open web steel joists, to support the mechanical equipment.

The three elevated floors have 3 in. of normal weight concrete over 3-in. composite deck for a total slab thickness of 6 in. The supporting beams are spaced at 10 ft on center. These beams are carried by composite girders in the east-west direction to the columns. There is a 30 ft by 29 ft opening in the second floor, to create a two-story atrium at the entrance. These floor layouts are shown on Sheets S2.1 and S2.2. The first floor is a slab on grade and the foundation consists of conventional spread footings.

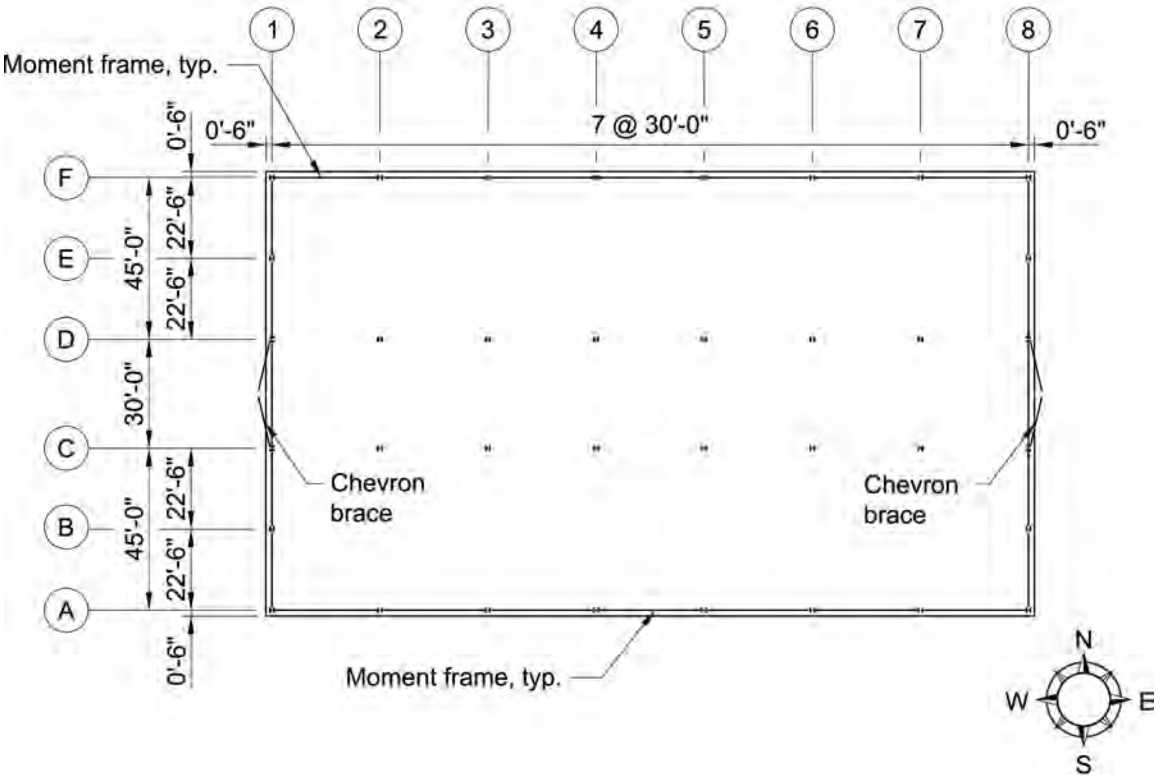

Fig. III-1. Basic building layout.

The building includes both moment frames and braced frames for lateral resistance. The lateral system in the north-south direction consists of chevron braces at the end of the building located adjacent to the stairways. In the east-west direction there are no locations in which chevron braces can be concealed; consequently, the lateral system in the east-west direction is composed of moment frames at the north and south faces of the building.

This building is sprinklered and has large open spaces around it, and consequently does not require fireproofing for the floors.

Wind Forces

The Basic Wind Speed is 107 miles per hour (3-second gust). Because it is sited in an open, rural area, it will be analyzed as Wind Exposure Category C. Because it is an ordinary office occupancy, the building is Risk Category II.

Seismic Forces

The sub-soil has been evaluated and the site class has been determined to be Site Class D. The area has a short period $S_s = 0.121g$ and a one-second period $S_1 = 0.060g$. The Seismic Importance Factor is 1.0, that of an ordinary office occupancy (Risk Category II).

Roof and Floor Loads

Roof Loads

The ground snow load, p_g, is 20 psf. The slope of the roof is ¼ in./ft or more at all locations, but not exceeding ½ in./ft; consequently, 5 psf rain-on-snow surcharge is to be considered, but ponding instability design calculations are beyond the scope of this example. This roof can be designed as a fully exposed roof, but, per ASCE/SEI 7, Section 7.3, cannot be designed for less than $p_f = (I)p_g = 20$ psf uniform snow load. Snow drift will be applied at the edges of the roof and at the screen wall around the mechanical area. The roof live load for this building is 20 psf, but may be reduced per ASCE/SEI 7, Section 4.8, where applicable.

Floor Loads

The basic live load for the floor is 50 psf. An additional partition live load of 20 psf is specified, which exceeds the minimum partition load required by ASCE/SEI 7, Section 4.3.2. Because the locations of partitions and, consequently, corridors are not known, and will be subject to change, the entire floor will be designed for a live load of 80 psf. This live load will be reduced based on type of member and area per the ASCE/SEI 7 provisions for live-load reduction.

Wall Loads

A wall load of 55 psf will be used for the brick spandrels, supporting steel, and metal stud back-up. A wall load of 15 psf will be used for the ribbon window glazing system.

ROOF MEMBER DESIGN AND SELECTION

Calculate dead load and snow load.

Dead load:

Roofing	= 5 psf
Insulation	= 2 psf
Deck	= 2 psf
Beams	= 3 psf
Joists	= 3 psf
Misc.	= 5 psf
Total	= 20 psf

Snow load from ASCE/SEI 7, Sections 7.3 and 7.10:

Snow	= 20 psf
Rain on snow	= 5 psf
Total	= 25 psf

Note: In this design, the rain and snow load is greater than the roof live load.

The deck is 1½ in., wide rib, 22 gage, painted roof deck, placed in a pattern of three continuous spans minimum. The typical joist spacing is 6 ft on center. At 6 ft on center, this deck has an allowable total load capacity of 87 psf (from the manufacturer's catalog). The roof diaphragm and roof loads extend 6 in. past the centerline of grid as shown on Sheet S4.1.

From ASCE/SEI 7, Section 7.7, the following drift loads are calculated:

Flat roof snow load: $p_g = 20$ psf
Density: $\gamma = 16.6$ lb/ft^3
$h_b = 1.20$ ft

Summary of Drifts

The snow drift at the penthouse was calculated for the maximum effect, using the east-west wind and an upwind fetch from the parapet to the centerline of the columns at the penthouse. This same drift is conservatively used for wind in the north-south direction. The precise location of the drift will depend upon the details of the penthouse construction, but will not affect the final design in this case. A summary of the drift load is given in Table III-1.

Table III-1
Summary of Drifts

	Upwind Roof Length, l_u, ft	Projection Height, ft	Max. Drift Load, psf	Max. Drift Width, W, ft
Side parapet	121	2	13.2	6.36
End parapet	211	2	13.2	6.36
Screen wall	60.5	6	30.5	7.35

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SELECT ROOF JOISTS

Layout loads and size joists.

The 45-ft side joist with the heaviest loads is shown in Figure III-2 with end reactions and maximum moment.

Note: Joists may be specified using ASD or LRFD but are most commonly specified by ASD as shown here.

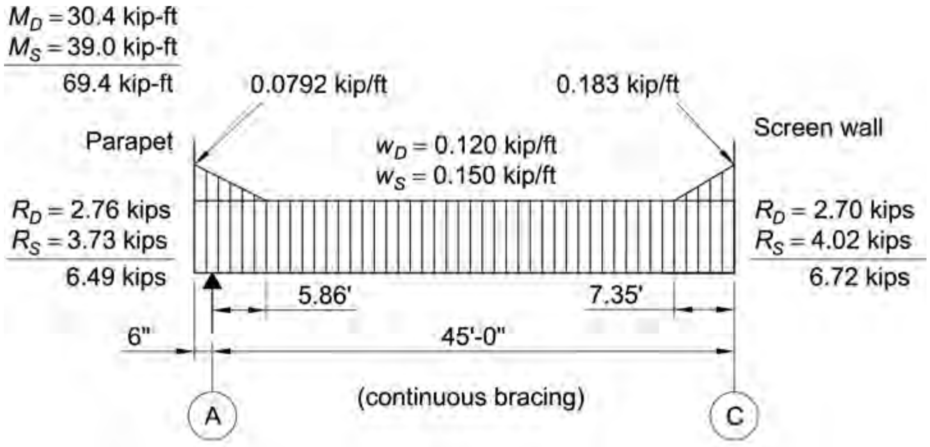


Fig III-2. Joist loading and bracing diagram—ASD.

Because the load is not uniform, select a 24KCS4 joist from the Steel Joist Institute (SJI) *Load Tables and Weight Tables for Steel Joists and Joist Girders* (SJI, 2015). This joist has an allowable moment of 92.3 kip-ft, an allowable shear of 8.40 kips, a gross moment of inertia of 453 in.^{4} and weighs 16.5 plf.

The first joist away from the end of the building is loaded with snow drift along the length of the member. Based on analysis, a 24KCS4 joist is also acceptable for this uniform load case.

As an alternative to directly specifying the joist sizes on the design document, as done in this example, loading diagrams can be included on the design documents to allow the joist manufacturer to economically design the joists.

The typical 30-ft-long joist in the middle bay will have a uniform load of:

$$\begin{aligned} w &= (6\text{ ft})(20\text{ psf} + 25\text{ psf}) \\ &= 270\text{ plf} \end{aligned}$$

$$\begin{aligned} w_S &= (6\text{ ft})(25\text{ psf}) \\ &= 150\text{ plf} \end{aligned}$$

From the SJI load tables, select an 18K5 joist that weighs approximately 7.7 plf and satisfies both strength and deflection requirements.

Note: the first joist away from the screen wall and the first joist away from the end of the building carry snow drift. Based on analysis, an 18K7 joist will be used in these locations.

SELECT ROOF BEAMS

Calculate loads and select beams in the mechanical area.

For the beams in the mechanical area, the mechanical units could weigh as much as 60 psf. Use 40 psf additional dead load, which will account for the mechanical units and the screen wall around the mechanical area. Use 15 psf additional snow load, which will account for any snow drift that could occur in the mechanical area. The beams in the mechanical area are spaced at 6 ft on center. Loading is calculated as follows and shown in Figure III-3.

$$\begin{aligned} w_D &= (6\text{ ft})\left(0.020\text{ kip/ft}^2 + 0.040\text{ kip/ft}^2\right) \\ &= 0.360\text{ kip/ft} \end{aligned}$$

$$\begin{aligned} w_S &= (6\text{ ft})\left(0.025\text{ kip/ft}^2 + 0.015\text{ kip/ft}^2\right) \\ &= 0.240\text{ kip/ft} \end{aligned}$$

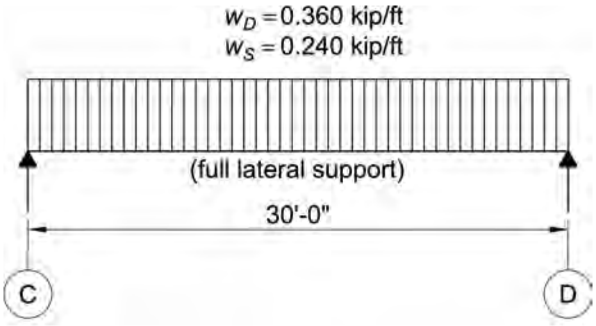


Fig. III-3. Loading and bracing diagram for roof beams in mechanical area.

From ASCE/SEI 7, Chapter 2, calculate the required strength of the beams in the mechanical area.

LRFD	ASD
$w_u = 1.2(0.360\text{ kip/ft}) + 1.6(0.240\text{ kip/ft})$ $= 0.816\text{ kip/ft}$	$w_a = 0.360\text{ kip/ft} + 0.240\text{ kip/ft}$ $= 0.600\text{ kip/ft}$
$R_u = (0.816\text{ kip/ft})\left(\frac{30\text{ ft}}{2}\right)$ $= 12.2\text{ kips}$	$R_a = (0.600\text{ kip/ft})\left(\frac{30\text{ ft}}{2}\right)$ $= 9.00\text{ kips}$
$M_u = \frac{(0.816\text{ kip/ft})(30\text{ ft})^2}{8}$ $= 91.8\text{ kip-ft}$	$M_a = \frac{(0.600\text{ kip/ft})(30\text{ ft})^2}{8}$ $= 67.5\text{ kip-ft}$

As discussed in AISC Design Guide 3, *Serviceability Design Considerations for Steel Buildings* (West and Fisher, 2003), limit deflection to $L/360$ because a plaster ceiling will be used in the lobby area.

$$\begin{aligned} \frac{L}{360} &= \frac{(30\text{ ft})(12\text{ in./ft})}{360} \\ &= 1.00\text{ in.} \end{aligned}$$

Using the equation for deflection derived previously, the required moment of inertia, $I_{x\ req}$, can be determined as follows. Use 40 psf as an estimate of the snow load, including some drifting that could occur in this area, for deflection calculations.

$$I_{x\ req} = \frac{(0.240\text{ kip/ft})(30\text{ ft})^4}{1{,}290(1.00\text{ in.})}$$

$$= 151\text{ in.}^4$$

From AISC *Manual* Table 3-3, select a beam size with an adequate moment of inertia. Try a W14×22:

$$I_x = 199\text{ in.}^4 > 151\text{ in.}^4 \quad \textbf{o.k.}$$

From AISC *Manual* Table 6-2, the available flexural strength and shear strength for a W14×22 is determined as follows. Assume the beam has full lateral support; therefore, $L_b = 0$.

LRFD	ASD
$\phi_b M_{nx} = 125\text{ kip-ft} > 91.8\text{ kip-ft}$ **o.k.**	$\dfrac{M_{nx}}{\Omega_b} = 82.8\text{ kip-ft} > 67.5\text{ kip-ft}$ **o.k.**
$\phi_v V_n = 94.5\text{ kips} > 12.2\text{ kips}$ **o.k.**	$\dfrac{V_n}{\Omega_v} = 63.0\text{ kips} > 9.00\text{ kips}$ **o.k.**

Note: The beams and supporting girders in this area should be rechecked when the final weights and locations for the mechanical units have been determined.

SELECT ROOF BEAMS AT THE END (EAST & WEST) OF THE BUILDING

The beams at the ends of the building carry the brick spandrel panel and a small portion of roof load. For these beams, the cladding weight exceeds 25% of the total dead load on the beam. Therefore, per AISC Design Guide 3, limit the vertical deflection due to cladding and initial dead load to *L*/600 or ⅜ in. maximum. In addition, because these beams are supporting brick above and there is continuous glass below, limit the superimposed dead and live load deflection to *L*/600 or 0.3 in. maximum to accommodate the brick and *L*/360 or ¼ in. maximum to accommodate the glass. Therefore, combining the two limitations, limit the superimposed dead and live load deflection to *L*/600 or ¼ in. The superimposed dead load includes all of the dead load that is applied after the cladding has been installed. In calculating the wall loads, the spandrel panel weight is taken as 55 psf. Beam loading is calculated as follows and shown in Figure III-4. Note, the beams are laterally supported by the deck as shown in Detail 4 on Sheet S4.1.

The dead load from the spandrel is:

$$\begin{aligned} w_D &= (7.50\text{ ft})\left(0.055\text{ kip/ft}^2\right) \\ &= 0.413\text{ kip/ft} \end{aligned}$$

The dead load from the roof is equal to:

$$\begin{aligned} w_D &= (3.50\text{ ft})\left(0.020\text{ kip/ft}^2\right) \\ &= 0.070\text{ kip/ft} \end{aligned}$$

Use 8 psf for the initial dead load, which includes the deck, beams and joists:

$$\begin{aligned} w_{D(initial)} &= (3.50\text{ ft})\left(0.008\text{ kip/ft}^2\right) \\ &= 0.028\text{ kip/ft} \end{aligned}$$

Use 12 psf for the superimposed dead load:

$$\begin{aligned} w_{D(super)} &= (3.50\text{ ft})\left(0.012\text{ kip/ft}^2\right) \\ &= 0.042\text{ kip/ft} \end{aligned}$$

The snow load from the roof conservatively uses the maximum snow drift as a uniform load, considering both side and end parapet drift pressures:

$$\begin{aligned} w_S &= (3.50\text{ ft})\left(0.025\text{ kip/ft}^2 + 0.0132\text{ kip/ft}^2\right) \\ &= 0.134\text{ kip/ft} \end{aligned}$$

From ASCE/SEI 7, Chapter 2, calculate the required strength of the beams at the east and west ends of the roof.

LRFD	ASD
$w_u = 1.2(0.483\text{ kip/ft}) + 1.6(0.134\text{ kip/ft})$ $= 0.794\text{ kip/ft}$	$w_a = 0.483\text{ kip/ft} + 0.134\text{ kip/ft}$ $= 0.617\text{ kip/ft}$

LRFD	ASD
$R_u = (0.794\text{ kip/ft})\left(\dfrac{22.5\text{ ft}}{2}\right)$ $= 8.93\text{ kips}$ $M_u = \dfrac{(0.794\text{ kip/ft})(22.5\text{ ft})^2}{8}$ $= 50.2\text{ kip-ft}$	$R_a = (0.617\text{ kip/ft})\left(\dfrac{22.5\text{ ft}}{2}\right)$ $= 6.94\text{ kips}$ $M_a = \dfrac{(0.617\text{ kip/ft})(22.5\text{ ft})^2}{8}$ $= 39.0\text{ kip-ft}$

Assume the beams are simple spans of 22.5 ft. Calculate the minimum moment of inertia to limit the superimposed dead and live load deflection after cladding is installed to *L*/600 or ¼ in.

$$\frac{L}{600} = \frac{(22.5\text{ ft})(12\text{ in./ft})}{600} \leq \tfrac{1}{4}\text{ in.}$$
$$= 0.450\text{ in.} > \tfrac{1}{4}\text{ in.}$$

Therefore, limit deflection to ¼ in. Using the equation for deflection derived previously, the required moment of inertia, $I_{x\,req}$, can be determined as follows:

$$I_{x\,req} = \frac{(0.042\text{ kip/ft} + 0.134\text{ kip/ft})(22.5\text{ ft})^4}{1{,}290(\tfrac{1}{4}\text{ in.})}$$
$$= 140\text{ in.}^4$$

Calculate minimum moment of inertia to limit the cladding and initial dead load deflection to *L*/600 or ⅜ in.

$$\frac{L}{600} = \frac{(22.5\text{ ft})(12\text{ in./ft})}{600} \leq \tfrac{3}{8}\text{ in.}$$
$$= 0.450\text{ in.} > \tfrac{3}{8}\text{ in.}$$

Therefore, limit deflection to ⅜ in. Using the equation for deflection derived previously, the required moment of inertia, $I_{x\,req}$, can be determined as follows:

$$I_{x\,req} = \frac{(0.413\text{ kip/ft} + 0.028\text{ kip/ft})(22.5\text{ ft})^4}{1{,}290(\tfrac{3}{8}\text{ in.})}$$
$$= 234\text{ in.}^4 \quad \textbf{controls}$$

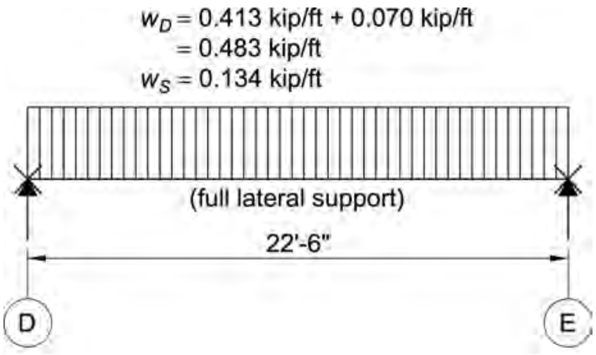


Fig. III-4. Beam loading and bracing diagram for roof beams at east and west ends of building.

From AISC *Manual* Table 3-3, select a beam size with an adequate moment of inertia. Try a W16×26:

$$I_x = 301\text{ in.}^4 > 234\text{ in.}^4 \quad \textbf{o.k.}$$

From AISC *Manual* Table 6-2, the available flexural strength and shear strength for a W16×26 is determined as follows. The beam has full lateral support; therefore, $L_b = 0$.

LRFD	ASD
$\phi_b M_{nx} = 166\text{ kip-ft} > 50.2\text{ kip-ft} \quad \textbf{o.k.}$	$\dfrac{M_{nx}}{\Omega_b} = 110\text{ kip-ft} > 39.0\text{ kip-ft} \quad \textbf{o.k.}$
$\phi_v V_n = 106\text{ kips} > 8.93\text{ kips} \quad \textbf{o.k.}$	$\dfrac{V_n}{\Omega_v} = 70.5\text{ kips} > 6.94\text{ kips} \quad \textbf{o.k.}$

SELECT ROOF BEAMS ALONG THE SIDE (NORTH & SOUTH) OF THE BUILDING

The beams along the side of the building carry the spandrel panel and a substantial roof dead load and live load. For these beams, the cladding weight exceeds 25% of the total dead load on the beam. From AISC Design Guide 3, limit the vertical deflection due to cladding and initial dead load to *L*/600 or ⅜ in. maximum. In addition, because these beams are supporting brick above and there is continuous glass below, limit the superimposed dead and live load deflection to *L*/600 or 0.3 in. maximum to accommodate the brick and *L*/360 or ¼ in. maximum to accommodate the glass. Therefore, combining the two limitations, limit the superimposed dead and live load deflection to *L*/600 or ¼ in. The superimposed dead load includes all of the dead load that is applied after the cladding has been installed. These beams will be part of the moment frames on the side of the building and therefore will be designed as fixed at both ends. The roof dead load and snow load on this edge beam is equal to the joist end dead load and snow load reaction. Treat this as a uniform load and divide by the joist spacing. (Note: treating this as a uniform load is a convenient and reasonable approximation in this case, resulting in a difference in maximum moment of approximately 4% as compared to the moment calculated using concentrated loading from each of the roof joists acting on the beam). Beam loading is calculated as follows, and shown in Figure III-5.

The dead load from the joist end reaction is:

$$
\begin{aligned}
w_D &= \frac{2.76\text{ kips}}{6.00\text{ ft}} \\
&= 0.460\text{ kip/ft}
\end{aligned}
$$

From previous calculations, the dead load from the spandrel is:

$$w_D = 0.413\text{ kip/ft}$$

The snow load from the joist end reaction is:

$$
\begin{aligned}
w_S &= \frac{3.73\text{ kips}}{6.00\text{ ft}} \\
&= 0.622\text{ kip/ft}
\end{aligned}
$$

Use 8 psf for initial dead load and 12 psf for superimposed dead load.

$$
\begin{aligned}
w_{D(initial)} &= (22.5\text{ ft} + 0.5\text{ ft})(0.008\text{ kip/ft}^2) \\
&= 0.184\text{ kip/ft}
\end{aligned}
$$

$$
\begin{aligned}
w_{D(super)} &= (22.5\text{ ft} + 0.5\text{ ft})(0.012\text{ kip/ft}^2) \\
&= 0.276\text{ kip/ft}
\end{aligned}
$$

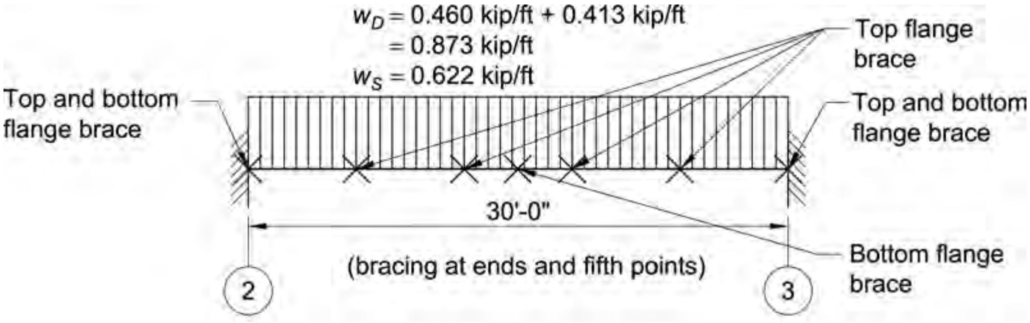


Fig. III-5. Loading and bracing diagram for roof beams at north and south ends of building.

From ASCE/SEI 7, Chapter 2, calculate the required strength of the beams at the roof sides.

LRFD	ASD
$w_u = 1.2\left(0.873\text{ kip/ft}\right) + 1.6\left(0.622\text{ kip/ft}\right)$ $= 2.04\text{ kip/ft}$ $R_u = \left(2.04\text{ kip/ft}\right)\left(\dfrac{30\text{ ft}}{2}\right)$ $= 30.6\text{ kips}$	$w_a = 0.873\text{ kip/ft} + 0.622\text{ kip/ft}$ $= 1.50\text{ kip/ft}$ $R_a = \left(1.50\text{ kip/ft}\right)\left(\dfrac{30\text{ ft}}{2}\right)$ $= 22.5\text{ kips}$

Using the equation for deflection derived previously, the required moment of inertia, $I_{x\,req}$, is determined as follows.

To limit the superimposed dead and live load deflection to ¼ in.:

$$I_{x\,req} = \frac{\left(0.622\text{ kip/ft} + 0.276\text{ kip/ft}\right)\left(30\text{ ft}\right)^4}{6{,}440\left(\tfrac{1}{4}\text{ in.}\right)}$$

$$= 452\text{ in.}^4 \quad \textbf{controls}$$

To limit the cladding and initial dead load deflection to ⅜ in.:

$$I_{req} = \frac{\left(0.597\text{ kip/ft}\right)\left(30.0\text{ ft}\right)^4}{6{,}440\left(\tfrac{3}{8}\text{ in.}\right)}$$

$$= 200\text{ in.}^4$$

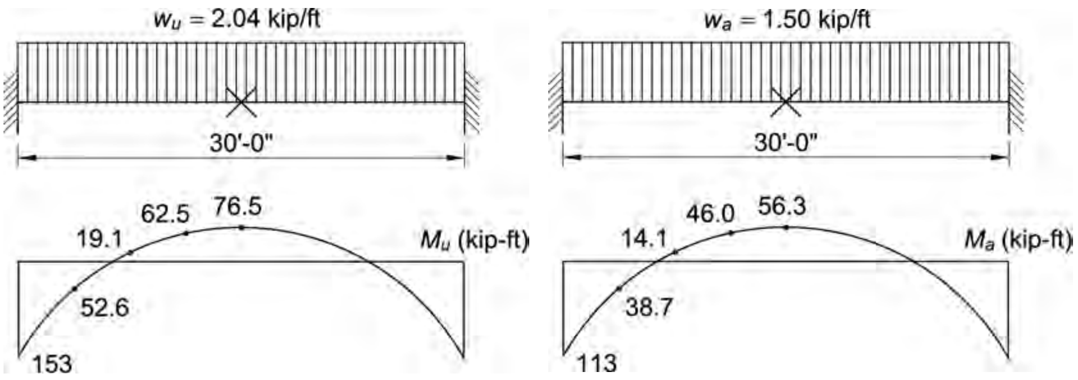
From AISC *Manual* Table 3-3, select a beam size with an adequate moment of inertia. Try a W18×35:

$$I_x = 510\text{ in.}^4 > 452\text{ in.}^4 \quad \textbf{o.k.}$$

Calculate C_b for compression in the bottom flange braced at the midpoint and supports using AISC *Specification* Equation F1-1. Moments along the span are summarized in Figure III-6.

LRFD	ASD
From AISC *Manual* Table 3-23, Case 15: $M_{u\,max} = \dfrac{\left(2.04\text{ kip/ft}\right)\left(30\text{ ft}\right)^2}{12}$ $= 153\text{ kip-ft (at supports)}$ At midpoint: $M_u = \dfrac{\left(2.04\text{ kip/ft}\right)\left(30\text{ ft}\right)^2}{24}$ $= 76.5\text{ kip-ft}$	From AISC *Manual* Table 3-23, Case 15: $M_{a\,max} = \dfrac{\left(1.50\text{ kip/ft}\right)\left(30\text{ ft}\right)^2}{12}$ $= 113\text{ kip-ft (at supports)}$ At midpoint: $M_a = \dfrac{\left(1.50\text{ kip/ft}\right)\left(30\text{ ft}\right)^2}{24}$ $= 56.3\text{ kip-ft}$

LRFD	ASD
At quarter-point of unbraced length: $M_{uA} = \frac{2.04\text{ kip/ft}}{12}\left[\begin{array}{l}6(30\text{ ft})(3.75\text{ ft})-(30\text{ ft})^2\\-6(3.75\text{ ft})^2\end{array}\right]$ $= 52.6\text{ kip-ft}$	At quarter-point of unbraced length: $M_{aA} = \frac{1.50\text{ kip/ft}}{12}\left[\begin{array}{l}6(30\text{ ft})(3.75\text{ ft})-(30\text{ ft})^2\\-6(3.75\text{ ft})^2\end{array}\right]$ $= 38.7\text{ kip-ft}$
At midpoint of unbraced length: $M_{uB} = \frac{2.04\text{ kip/ft}}{12}\left[\begin{array}{l}6(30\text{ ft})(7.50\text{ ft})-(30\text{ ft})^2\\-6(7.50\text{ ft})^2\end{array}\right]$ $= 19.1\text{ kip-ft}$	At midpoint of unbraced length: $M_{aB} = \frac{1.50\text{ kip/ft}}{12}\left[\begin{array}{l}6(30\text{ ft})(7.50\text{ ft})-(30\text{ ft})^2\\-6(7.50\text{ ft})^2\end{array}\right]$ $= 14.1\text{ kip-ft}$
At three-quarter point of unbraced length: $M_{uC} = \frac{2.04\text{ kip/ft}}{12}\left[\begin{array}{l}6(30\text{ ft})(11.3\text{ ft})-(30\text{ ft})^2\\-6(11.3\text{ ft})^2\end{array}\right]$ $= 62.5\text{ kip-ft}$	At three-quarter point of unbraced length: $M_{aC} = \frac{1.50\text{ kip/ft}}{12}\left[\begin{array}{l}6(30\text{ ft})(11.3\text{ ft})-(30\text{ ft})^2\\-6(11.3\text{ ft})^2\end{array}\right]$ $= 46.0\text{ kip-ft}$
Using AISC *Specification* Equation F1-1: $C_b = \frac{12.5M_{max}}{2.5M_{max}+3M_A+4M_B+3M_C}$ $= \frac{12.5(153\text{ kip-ft})}{\left[\begin{array}{l}2.5(153\text{ kip-ft})+3(52.6\text{ kip-ft})\\+4(19.1\text{ kip-ft})+3(62.5\text{ kip-ft})\end{array}\right]}$ $= 2.38$	Using AISC *Specification* Equation F1-1: $C_b = \frac{12.5M_{max}}{2.5M_{max}+3M_A+4M_B+3M_C}$ $= \frac{12.5(113\text{ kip-ft})}{\left[\begin{array}{l}2.5(113\text{ kip-ft})+3(38.7\text{ kip-ft})\\+4(14.1\text{ kip-ft})+3(46.0\text{ kip-ft})\end{array}\right]}$ $= 2.38$



(a) LRFD *(b) ASD*

Fig. III-6. Beam moment diagram.

From AISC *Manual* Table 6-2, with L_b = 6 ft and C_b = 1.0 the available flexural strength is determined as follows:

LRFD	ASD
$\phi_b M_n = 229 \text{ kip-ft} > 76.5 \text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = 152 \text{ kip-ft} > 56.3 \text{ kip-ft}$ **o.k.**

From AISC *Manual* Table 6-2, with L_b = 15 ft and C_b = 2.38, the available flexural strength is determined as follows:

LRFD	ASD
$\phi_b M_n C_b \le \phi_b M_p$ $(109 \text{ kip-ft})(2.38) > 249 \text{ kip-ft}$ $259 \text{ kip-ft} > 249 \text{ kip-ft}$ Therefore: $\phi_b M_n = 249 \text{ kip-ft} > 153 \text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} C_b \le \dfrac{M_p}{\Omega_b}$ $(72.4 \text{ kip-ft})(2.38) > 166 \text{ kip-ft}$ $172 \text{ kip-ft} > 166 \text{ kip-ft}$ Therefore: $\dfrac{M_n}{\Omega_b} = 166 \text{ kip-ft} > 113 \text{ kip-ft}$ **o.k.**

From AISC *Manual* Table 6-2, the available shear strength is determined as follows:

LRFD	ASD
$\phi_v V_n = 159 \text{ kips} > 30.6 \text{ kips}$ **o.k.**	$\dfrac{V_n}{\Omega_v} = 106 \text{ kips} > 22.5 \text{ kips}$ **o.k.**

Therefore, the W18×35 is acceptable.

Note: This roof beam may need to be upsized during the lateral load analysis to increase the stiffness and strength of the member and improve lateral frame drift performance.

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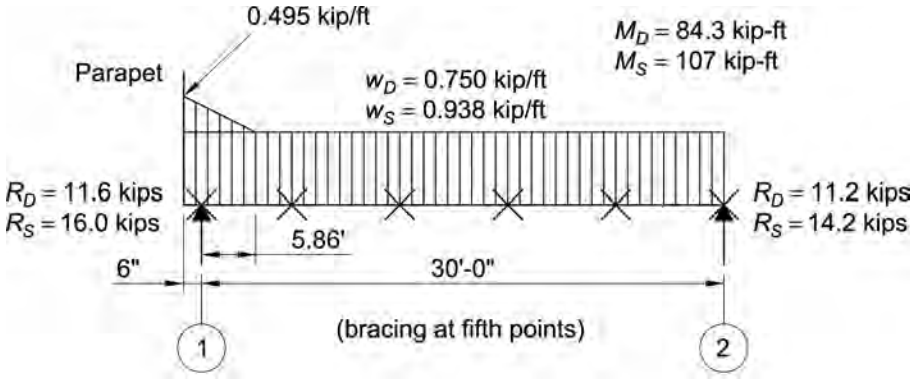
SELECT THE ROOF BEAMS ALONG THE INTERIOR LINES OF THE BUILDING

There are three individual beam loadings that occur along grids C and D. The beams from 1 to 2 and 7 to 8 have a uniform snow load except for the snow drift at the end at the parapet. The snow drift from the far ends of the 45-ft joists is negligible. The beams from 2 to 3 and 6 to 7 are the same as the first group, except they have snow drift at the screen wall. The live load deflection is limited to *L*/240 (or 1.50 in.). Joist reactions are divided by the joist spacing and treated as a uniform load, just as they were for the side beams.

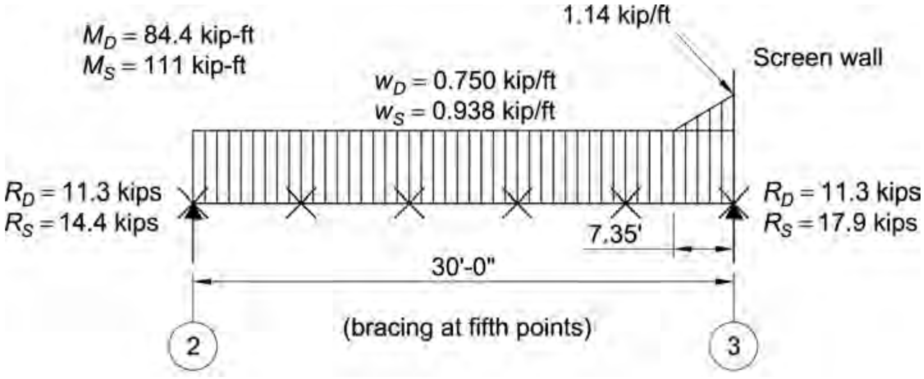
$$w_D = \left(0.020 \text{ kip/ft}^2\right)\left(\frac{45 \text{ ft} + 30 \text{ ft}}{2}\right)$$
$$= 0.750 \text{ kip/ft}$$

$$w_S = \left(0.025 \text{ kip/ft}^2\right)\left(\frac{45 \text{ ft} + 30 \text{ ft}}{2}\right)$$
$$= 0.938 \text{ kip/ft}$$

The loading diagrams with moments and end reactions are shown in Figure III-7.



(a) Grids 1 to 2 and 7 to 8



(b) Grids 2 to 3 and 6 to 7

Fig. III-7. Roof beam loading and bracing diagram.

From ASCE/SEI 7, Chapter 2, the required strength for the beams from grids 1 to 2 and 7 to 8 (opposite hand) is determined as follows:

LRFD	ASD
$R_u\,(\text{left end}) = 1.2(11.6\text{ kips}) + 1.6(16.0\text{ kips})$ $= 39.5\text{ kips}$	$R_a\,(\text{left end}) = 11.6\text{ kips} + 16.0\text{ kips}$ $= 27.6\text{ kips}$
$R_u\,(\text{right end}) = 1.2(11.2\text{ kips}) + 1.6(14.2\text{ kips})$ $= 36.2\text{ kips}$	$R_a\,(\text{right end}) = 11.2\text{ kips} + 14.2\text{ kips}$ $= 25.4\text{ kips}$
$M_u = 1.2(84.3\text{ kip-ft}) + 1.6(107\text{ kip-ft})$ $= 272\text{ kip-ft}$	$M_a = 84.3\text{ kip-ft} + 107\text{ kip-ft}$ $= 191\text{ kip-ft}$

Using the equation for deflection derived previously, the minimum moment of inertia, $I_{x\,req}$, to limit the live load deflection to 1.50 in., considering a 30-ft simply supported beam and neglecting the modest snow drift is:

$$
\begin{aligned}
I_{x\,req} &= \frac{(0.938\text{ kip/ft})(30\text{ ft})^4}{1{,}290(1.50\text{ in.})} \\
&= 393\text{ in.}^4
\end{aligned}
$$

From AISC *Manual* Table 3-3, select a beam size with an adequate moment of inertia. Try a W21×44:

$$I_x = 843\text{ in.}^4 > 393\text{ in.}^4 \quad \textbf{o.k.}$$

From AISC *Manual* Table 6-2, for a W21×44 with L_b = 6 ft and C_b = 1.0, the available flexural strength and shear strength is determined as follows:

LRFD	ASD
$\phi_b M_n = 332\text{ kip-ft} > 272\text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = 221\text{ kip-ft} > 191\text{ kip-ft}$ **o.k.**
$\phi_v V_n = 217\text{ kips} > 39.5\text{ kips}$ **o.k.**	$\dfrac{V_n}{\Omega_v} = 145\text{ kips} > 27.6\text{ kips}$ **o.k.**

From ASCE/SEI 7, Chapter 2, the required strength for the beams from grids 2 to 3 and 6 to 7 (opposite hand) is determined as follows:

LRFD	ASD
$R_u\,(\text{left end}) = 1.2(11.3\text{ kips}) + 1.6(14.4\text{ kips})$ $= 36.6\text{ kips}$	$R_a\,(\text{left end}) = 11.3\text{ kips} + 14.4\text{ kips}$ $= 25.7\text{ kips}$
$R_u\,(\text{right end}) = 1.2(11.3\text{ kips}) + 1.6(17.9\text{ kips})$ $= 42.2\text{ kips}$	$R_a\,(\text{right end}) = 11.3\text{ kips} + 17.9\text{ kips}$ $= 29.2\text{ kips}$
$M_u = 1.2(84.4\text{ kip-ft}) + 1.6(111\text{ kip-ft})$ $= 279\text{ kip-ft}$	$M_a = 84.4\text{ kip-ft} + 111\text{ kip-ft}$ $= 195\text{ kip-ft}$

From AISC *Manual* Table 6-2, for a W21×44 with L_b = 6 ft and C_b = 1.0, the available flexural strength and shear strength is determined as follows:

LRFD	ASD
$\phi_b M_n = 332 \text{ kip-ft} > 279 \text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = 221 \text{ kip-ft} > 195 \text{ kip-ft}$ **o.k.**
$\phi_v V_n = 217 \text{ kips} > 42.2 \text{ kips}$ **o.k.**	$\dfrac{V_n}{\Omega_v} = 145 \text{ kips} > 29.2 \text{ kips}$ **o.k.**

The third individual beam loading occurs at the beams from 3 to 4, 4 to 5, and 5 to 6. For these beams there is a uniform snow load outside the screen walled area, except for the snow drift at the parapet ends and the screen wall ends of the 45-ft-long joists. Inside the screen walled area the beams support the mechanical equipment. The loading diagram is shown in Figure III-8.

$$\begin{aligned} w_D &= \left(\frac{2.70 \text{ kips}}{6 \text{ ft}}\right) + \left(0.360 \text{ kip/ft}^2\right)\left(\frac{15 \text{ ft}}{6 \text{ ft}}\right) \\ &= 1.35 \text{ kip/ft} \end{aligned}$$

$$\begin{aligned} w_S &= \left(\frac{4.02 \text{ kips}}{6 \text{ ft}}\right) + \left(0.240 \text{ kip/ft}^2\right)\left(\frac{15 \text{ ft}}{6 \text{ ft}}\right) \\ &= 1.27 \text{ kip/ft} \end{aligned}$$

From ASCE/SEI 7, Chapter 2, the required strength for the beams from grids 3 to 4, 4 to 5, and 5 to 6 is determined as follows:

LRFD	ASD
$w_u = 1.2(1.35 \text{ kip/ft}) + 1.6(1.27 \text{ kip/ft})$ $= 3.65 \text{ kip/ft}$	$w_a = 1.35 \text{ kip/ft} + 1.27 \text{ kip/ft}$ $= 2.62 \text{ kip/ft}$
$M_u = \dfrac{(3.65 \text{ kip/ft})(30 \text{ ft})^2}{8}$ $= 411 \text{ kip-ft}$	$M_a = \dfrac{(2.62 \text{ kip/ft})(30 \text{ ft})^2}{8}$ $= 295 \text{ kip-ft}$

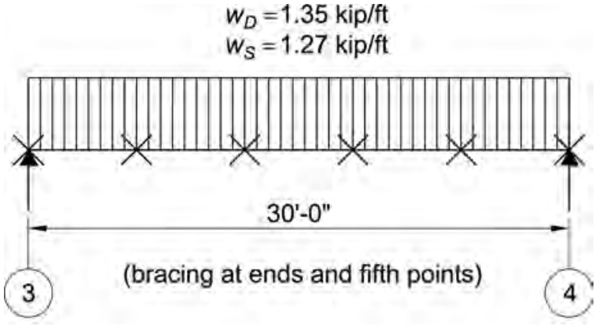


Fig. III-8. Loading and bracing diagram for roof beams from grid 3 to 4, 4 to 5, and 5 to 6.

LRFD	ASD
$R_u = (3.65 \text{ kip/ft})\left(\dfrac{30 \text{ ft}}{2}\right)$ $= 54.8 \text{ kips}$	$R_a = (2.62 \text{ kip/ft})\left(\dfrac{30 \text{ ft}}{2}\right)$ $= 39.3 \text{ kips}$

Using the equation for deflection derived previously, the minimum moment of inertia, $I_{x\ req}$, to limit the live load deflection to 1.50 in. is:

$$I_{x\ req} = \frac{(1.27 \text{ kip/ft})(30 \text{ ft})^4}{1{,}290(1.50 \text{ in.})}$$
$$= 532 \text{ in.}^4$$

From AISC *Manual* Table 3-3, select a beam size with an adequate moment of inertia. Try a W21×55:

$$I_x = 1{,}140 \text{ in.}^4 > 532 \text{ in.}^4 \quad \textbf{o.k.}$$

From AISC *Manual* Table 6-2, for a W21×55 with L_b = 6 ft and C_b = 1.0, the available flexural strength and shear strength is determined as follows:

LRFD	ASD
$\phi_b M_n = 473 \text{ kip-ft} > 411 \text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = 314 \text{ kip-ft} > 295 \text{ kip-ft}$ **o.k.**
$\phi_v V_n = 234 \text{ kips} > 54.8 \text{ kips}$ **o.k.**	$\dfrac{V_n}{\Omega_v} = 156 \text{ kips} > 39.3 \text{ kips}$ **o.k.**

FLOOR MEMBER DESIGN AND SELECTION

Calculate dead load and live load.

Dead load:

Slab and deck	= 57 psf
Beams (est.)	= 8 psf
Misc. (ceiling, mechanical, etc.)	= 10 psf
Total	= 75 psf

Note: The weight of the floor slab and deck was obtained from the manufacturer’s literature.

Live load:

Total (can be reduced for area per ASCE/SEI 7) = 80 psf

The floor and deck will be 3 in. of normal weight concrete, f'_c = 4 ksi, on 3-in., 20 gage, galvanized, composite deck, laid in a pattern of three or more continuous spans. The total depth of the slab is 6 in. From the Steel Deck Institute *Floor Deck Design Manual* (SDI, 2014), the maximum unshored span for construction with this deck and a three-span condition is 10 ft 6 in. The general layout for the floor beams is 10 ft on center; therefore, the deck does not need to be shored during construction. At 10 ft on center, this deck has an allowable superimposed live load capacity of 143 psf. In addition, it can be shown that this deck can carry a 2,000 pound load over an area of 2.5 ft by 2.5 ft as required by ASCE/SEI 7, Section 4.4. The floor diaphragm and the floor loads extend 6 in. past the centerline of grid as shown on Sheet S4.1.

SELECT FLOOR BEAMS (COMPOSITE AND NONCOMPOSITE)

Note: There are two early and important checks in the design of composite beams. First, select a beam that either does not require camber, or establish a target camber and moment of inertia at the start of the design process. A reasonable approximation of the camber is between *L*/300 minimum and *L*/180 maximum (or a maximum of 1½ to 2 in.).

Second, check that the beam is strong enough to safely carry the wet concrete and a 20 psf construction live load [per *Design Loads on Structures During Construction*, ASCE 37-14 (ASCE, 2014)] when designed by the ASCE/SEI 7 load combinations and the provisions of AISC *Specification* Chapter F.

SELECT TYPICAL 45-FT-LONG INTERIOR COMPOSITE BEAM (10 FT ON CENTER)

Find a target moment of inertia for an unshored beam.

$$\begin{aligned} w_D &= (10\text{ ft})\left(0.057\text{ kip/ft}^2 + 0.008\text{ kip/ft}^2\right) \\ &= 0.650\text{ kip/ft} \end{aligned}$$

Hold deflection to 2 in. maximum to facilitate concrete placement. Using the equation for deflection derived previously, the required moment of inertia is determined as follows:

$$\begin{aligned} I_{req} &\approx \frac{(0.650\text{ kip/ft})(45\text{ ft})^4}{1{,}290(2\text{ in.})} \\ &= 1{,}030\text{ in.}^4 \end{aligned}$$

The construction live load is determined as follows:

$$\begin{aligned} w_L &= (10\text{ ft})\left(0.020\text{ kip/ft}^2\right) \\ &= 0.200\text{ kip/ft} \end{aligned}$$

From ASCE/SEI 7, the required flexural strength due to wet concrete only is determined as follows:

LRFD	ASD
$w_u = 1.4(0.650\text{ kip/ft}) = 0.910\text{ kip/ft}$	$w_a = 0.650\text{ kip/ft}$
$M_u = \dfrac{(0.910\text{ kip/ft})(45\text{ ft})^2}{8} = 230\text{ kip-ft}$	$M_a = \dfrac{(0.650\text{ kip/ft})(45\text{ ft})^2}{8} = 165\text{ kip-ft}$

From ASCE/SEI 7, the required flexural strength due to wet concrete and construction live load is determined as follows:

LRFD	ASD
$w_u = 1.2(0.650\text{ kip/ft}) + 1.6(0.200\text{ kip/ft}) = 1.10\text{ kip/ft}$	$w_a = 0.650\text{ kip/ft} + 0.200\text{ kip/ft} = 0.850\text{ kip/ft}$

LRFD	ASD
$M_u = \dfrac{(1.10 \text{ kip/ft})(45 \text{ ft})^2}{8}$ $= 278 \text{ kip-ft}$ **controls**	$M_a = \dfrac{(0.850 \text{ kip/ft})(45 \text{ ft})^2}{8}$ $= 215 \text{ kip-ft}$ **controls**

Use AISC *Manual* Table 3-2 to select a beam with $I_x \geq 1{,}030 \text{ in.}^4$ Select W21×50, with $I_x = 984 \text{ in.}^4$, close to the target value.

From AISC *Manual* Table 6-2, the available flexural strength for a fully braced, $L_b = 0$ ft, W21×50 is determined as follows:

LRFD	ASD
$\phi_b M_n = 413 \text{ kip-ft} > 278 \text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = 274 \text{ kip-ft} > 215 \text{ kip-ft}$ **o.k.**

Check for possible live load reduction due to area in accordance with ASCE/SEI 7, Section 4.7.2.

From ASCE/SEI 7, Table 4.7-1, for interior beams:

$$K_{LL} = 2$$

The beams are at 10 ft on center, therefore the tributary area is:

$$\begin{aligned} A_T &= (45 \text{ ft})(10 \text{ ft}) \\ &= 450 \text{ ft}^2 \end{aligned}$$

$$\begin{aligned} K_{LL} A_T &= 2\left(450 \text{ ft}^2\right) \\ &= 900 \text{ ft}^2 \end{aligned}$$

Because $K_{LL}A_T \geq 400 \text{ ft}^2$, a reduced live load can be used.

From ASCE/SEI 7, Equation 4.7-1:

$$\begin{aligned} L &= L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right) \geq 0.50 L_o \\ &= (80 \text{ psf})\left(0.25 + \frac{15}{\sqrt{900 \text{ ft}^2}}\right) > 0.50(80 \text{ psf}) \\ &= 60.0 \text{ psf} > 40.0 \text{ psf} \end{aligned}$$

Therefore, use $L = 60.0$ psf.

The beams are at 10 ft on center, therefore the loading is as shown in Figure III-9. Note, the beam is continuously braced by the deck.

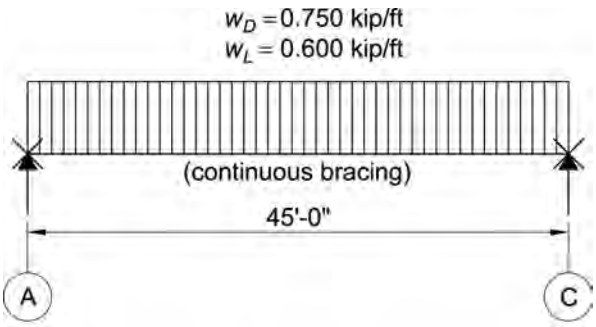


Fig. III-9. Loading and bracing diagram for typical interior composite floor beams.

From ASCE/SEI 7, Chapter 2, the required strengths are determined as follows:

LRFD	ASD
$w_u = 1.2(0.750\text{ kip/ft}) + 1.6(0.600\text{ kip/ft})$ $= 1.86\text{ kip/ft}$	$w_a = 0.750\text{ kip/ft} + 0.600\text{ kip/ft}$ $= 1.35\text{ kip/ft}$
$R_u = (1.86\text{ kip/ft})\left(\dfrac{45\text{ ft}}{2}\right)$ $= 41.9\text{ kips}$	$R_a = (1.35\text{ kip/ft})\left(\dfrac{45\text{ ft}}{2}\right)$ $= 30.4\text{ kips}$
$M_u = \dfrac{(1.86\text{ kip/ft})(45\text{ ft})^2}{8}$ $= 471\text{ kip-ft}$	$M_a = \dfrac{(1.35\text{ kip/ft})(45\text{ ft})^2}{8}$ $= 342\text{ kip-ft}$

The available flexural strength for the composite beam is determined using AISC *Manual* Part 3. Assume initially a = 1 in.

$$Y2 = Y_{con} - \frac{a}{2} \qquad \text{(\textit{Manual} Eq. 3-6)}$$

$$= 6.00\text{ in.} - \frac{1\text{ in.}}{2}$$

$$= 5.50\text{ in.}$$

Enter AISC *Manual* Table 3-19 for a W21×50 with $Y2$ = 5.50 in. Selecting PNA location 7, with $\Sigma Q_n = 184$ kips, the available flexural strength is:

LRFD	ASD
$\phi_b M_n = 598\text{ kip-ft} > 471\text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = 398\text{ kip-ft} > 342\text{ kip-ft}$ **o.k.**

Determine effective width, b

The effective width of the concrete slab is the sum of the effective widths for each side of the beam centerline as determined by the minimum value of the three widths set forth in AISC *Specification* Section I3.1a:

1. one-eighth of the span of the beam, center-to-center of the supports

$$\left(\frac{45\text{ ft}}{8}\right)(2\text{ sides}) = 11.3\text{ ft}$$

2. one-half the distance to the centerline of the adjacent beam

$$\left(\frac{10\text{ ft}}{2}\right)(2\text{ sides}) = 10.0\text{ ft} \quad \textbf{controls}$$

3. distance to the edge of the slab

The latter is not applicable for an interior member.

Determine the height of the compression block, *a*.

$$a = \frac{\sum Q_n}{0.85 f_c{}'b} \qquad \text{(}\textit{Manual}\text{ Eq. 3-7)}$$

$$= \frac{184\text{ kips}}{0.85(4\text{ ksi})(10\text{ ft})(12\text{ in./ft})}$$

$$= 0.451\text{ in.} < 1.00\text{ in.} \quad \textbf{o.k.}$$

From AISC *Manual* Table 6-2, the available shear strength of the W21×50 bare steel beam is determined as follows:

LRFD	ASD
$\phi_v V_n = 237\text{ kips} > 41.9\text{ kips}$ **o.k.**	$\dfrac{V_n}{\Omega_v} = 158\text{ kips} > 30.4\text{ kips}$ **o.k.**

Check live load deflection

$$\frac{L}{360} = \frac{(45\text{ ft})(12\text{ in./ft})}{360}$$

$$= 1.50\text{ in.}$$

Entering AISC *Manual* Table 3-20 for a W21×50, with PNA location 7 and $Y2$ = 5.50 in., provides a lower bound moment of inertia of I_{LB} = 1,730 in.^{4} From the equation previously derived, the live load deflection is determined as follows:

$$\Delta_{LL} = \frac{w_L L^4}{1{,}290 I_{LB}}$$

$$= \frac{(0.600\text{ kip/ft})(45\text{ ft})^4}{1{,}290(1{,}730\text{ in.}^4)}$$

$$= 1.10\text{ in.} < 1.50\text{ in.} \quad \textbf{o.k.}$$

From AISC Design Guide 3 limit the live load deflection, using 50% of the (unreduced) design live load, to L/360 with a maximum absolute value of 1 in. across the bay. From the equation previously derived, the deflection is determined as follows:

$$\Delta_{LL} = \frac{0.5\left(0.800\text{ kip/ft}\right)\left(45\text{ ft}\right)^4}{1{,}290\left(1{,}730\text{ in.}^4\right)} \le 1\text{ in.}$$

$$= 0.735\text{ in.} < 1\text{ in.}$$

$$= 0.735\text{ in.}$$

$$1\text{ in.} - 0.735\text{ in.} = 0.265\text{ in.}$$

Note: Limit the supporting girders to 0.265 in. deflection under the same load case at the connection point of the beam.

Determine the required number of shear stud connectors

From AISC *Manual* Table 3-21, using perpendicular deck with one ¾-in.-diameter anchor per rib in normal weight concrete with $f'_c = 4\text{ ksi}$ in the weak position:

$$Q_n = 17.2\text{ kips/anchor}$$

$$n = \frac{\Sigma Q_n}{Q_n}$$

$$= \frac{184\text{ kips}}{17.2\text{ kips/anchor}}$$

$$= 10.7\text{ anchors (on each side of maximum moment)}$$

Therefore, 22 studs are required to satisfy strength requirements. However, per AISC *Specification* Commentary Section I3.2d.1, 44 studs are specified to provide sufficient deformation capacity by ensuring a degree of composite action of at least 50%.

From AISC Design Guide 3, limit the wet concrete deflection in a bay to *L*/360, not to exceed 1 in. From the equation previously derived, the wet concrete deflection is determined as follows:

$$\Delta_{DL(wet\ conc)} = \frac{\left(0.650\text{ kip/ft}\right)\left(45\text{ ft}\right)^4}{1{,}290\left(984\text{ in.}^4\right)}$$

$$= 2.10\text{ in.}$$

Camber the beam for 80% of the calculated wet deflection.

$$Camber = 0.80\left(2.10\text{ in.}\right)$$

$$= 1.68\text{ in.}$$

Round the calculated value down to the nearest ¼ in.; therefore, specify 1½ in. of camber.

$$2.10\text{ in.} - 1\tfrac{1}{2}\text{ in.} = 0.600\text{ in.}$$

$$1\text{ in.} - 0.600\text{ in.} = 0.400\text{ in.}$$

Note: Limit the supporting girders to 0.400 in. deflection under the same load combination at the connection point of the beam.

SELECT TYPICAL 30-FT INTERIOR COMPOSITE (OR NONCOMPOSITE) BEAM (10 FT ON CENTER)

Find a target moment of inertia for an unshored beam.

Determine the required strength to carry wet concrete and construction live load. The dead load from the slab and deck is:

$$\begin{aligned} w_D &= (10\text{ ft})\left(0.057\text{ kip/ft}^2 + 0.008\text{ kip/ft}^2\right) \\ &= 0.650\text{ kip/ft} \end{aligned}$$

Hold deflection to 1½ in. maximum to facilitate concrete placement. Using the equation for deflection derived previously, the required moment of inertia is determined as follows:

$$\begin{aligned} I_{req} &\approx \frac{(0.650\text{ kip/ft})(30\text{ ft})^4}{1{,}290(1\tfrac{1}{2}\text{ in.})} \\ &= 272\text{ in.}^4 \end{aligned}$$

The construction live load is:

$$\begin{aligned} w_L &= (10\text{ ft})\left(0.020\text{ kip/ft}^2\right) \\ &= 0.200\text{ kip/ft} \end{aligned}$$

From ASCE/SEI 7, Chapter 2, determine the required flexural strength due to wet concrete only.

LRFD	ASD
$w_u = 1.4(0.650\text{ kip/ft})$ $= 0.910\text{ kip/ft}$ $M_u = \dfrac{(0.910\text{ kip/ft})(30\text{ ft})^2}{8}$ $= 102\text{ kip-ft}$	$w_a = 0.650\text{ kip/ft}$ $M_a = \dfrac{(0.650\text{ kip/ft})(30\text{ ft})^2}{8}$ $= 73.1\text{ kip-ft}$

From ASCE/SEI 7, Chapter 2, determine the required flexural strength due to wet concrete and construction live load.

LRFD	ASD
$w_u = 1.2(0.650\text{ kip/ft}) + 1.6(0.200\text{ kip/ft})$ $= 1.10\text{ kip/ft}$ $M_u = \dfrac{(1.10\text{ kip/ft})(30\text{ ft})^2}{8}$ $= 124\text{ kip-ft}$ **controls**	$w_a = 0.650\text{ kip/ft} + 0.200\text{ kip/ft}$ $= 0.850\text{ kip/ft}$ $M_a = \dfrac{(0.850\text{ kip/ft})(30\text{ ft})^2}{8}$ $= 95.6\text{ kip-ft}$ **controls**

Use AISC *Manual* Table 3-2 to find a beam with an $I_x \geq 272$ in.^{4} Select W16×26, with $I_x = 301$ in.^{4}, which exceeds the target value.

From AISC *Manual* Table 6-2, the available flexural strength for a fully braced, L_b = 0 ft, W16×26 is determined as follows:

LRFD	ASD
$\phi_b M_n = 166 \text{ kip-ft} > 124 \text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = 110 \text{ kip-ft} > 95.6 \text{ kip-ft}$ **o.k.**

Check for possible live load reduction due to area in accordance with ASCE/SEI 7, Section 4.7.2.

From ASCE/SEI 7, Table 4.7-1, for interior beams:

$$K_{LL} = 2$$

The beams are at 10 ft on center, therefore the tributary area is:

$$\begin{aligned} A_T &= (30 \text{ ft})(10 \text{ ft}) \\ &= 300 \text{ ft}^2 \end{aligned}$$

$$\begin{aligned} K_{LL} A_T &= 2\left(300 \text{ ft}^2\right) \\ &= 600 \text{ ft}^2 \end{aligned}$$

Because $K_{LL}A_T \geq 400 \text{ ft}^2$, a reduced live load can be used.

From ASCE/SEI 7, Equation 4.7-1:

$$\begin{aligned} L &= L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right) \geq 0.50 L_o \\ &= (80 \text{ psf})\left(0.25 + \frac{15}{\sqrt{600 \text{ ft}^2}}\right) > 0.50(80 \text{ psf}) \\ &= 69.0 \text{ psf} > 40.0 \text{ psf} \end{aligned}$$

Therefore, use L = 69.0 psf.

The beams are at 10 ft on center, therefore the loading is as shown in Figure III-10.

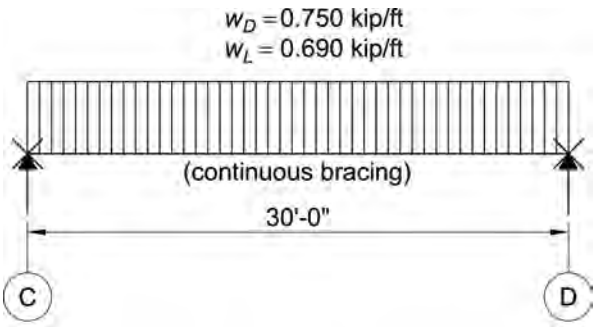


Fig. III-10. Loading and bracing diagram for typical 30-ft interior floor beams.

From ASCE/SEI 7, Chapter 2, calculate the required strength.

LRFD	ASD
$w_u = 1.2(0.750\text{ kip/ft}) + 1.6(0.690\text{ kip/ft})$ $= 2.00\text{ kip/ft}$ $R_u = (2.00\text{ kip/ft})\left(\dfrac{30\text{ ft}}{2}\right)$ $= 30.0\text{ kips}$ $M_u = \dfrac{(2.00\text{ kip/ft})(30\text{ ft})^2}{8}$ $= 225\text{ kip-ft}$	$w_a = 0.750\text{ kip/ft} + 0.690\text{ kip/ft}$ $= 1.44\text{ kip/ft}$ $R_a = (1.44\text{ kip/ft})\left(\dfrac{30\text{ ft}}{2}\right)$ $= 21.6\text{ kips}$ $M_a = \dfrac{(1.44\text{ kip/ft})(30\text{ ft})^2}{8}$ $= 162\text{ kip-ft}$

The available flexural strength for the composite beam is determined from AISC *Manual* Part 3 as follows. Assume initially that a = 1 in.

$$\begin{aligned} Y2 &= Y_{con} - \frac{a}{2} \\ &= 6.00\text{ in.} - \frac{1\text{ in.}}{2} \\ &= 5.50\text{ in.} \end{aligned}$$ (*Manual* Eq. 3-6)

Enter AISC *Manual* Table 3-19 for a W16×26 with $Y2$ = 5.50 in. Selecting PNA location 7, with ΣQ_n = 96.0 kips, the available flexural strength is:

LRFD	ASD
$\phi_b M_n = 248\text{ kip-ft} > 225\text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = 165\text{ kip-ft} > 162\text{ kip-ft}$ **o.k.**

Determine effective width, b

The effective width of the concrete slab is the sum of the effective widths for each side of the beam centerline as determined by the minimum value of the three widths set forth in AISC *Specification* Section I3.1a:

1. one-eighth of the span of the beam, center-to-center of the supports

$$\left(\frac{30\text{ ft}}{8}\right)(2\text{ sides}) = 7.50\text{ ft} \quad \textbf{controls}$$

2. one-half the distance to the centerline of the adjacent beam

$$\left(\frac{10\text{ ft}}{2}\right)(2\text{ sides}) = 10.0\text{ ft}$$

3. distance to the edge of the slab

The latter is not applicable for an interior member.

Determine the height of the compression block, a.

$$a = \frac{\sum Q_n}{0.85 f_c{}'b} \qquad (\textit{Manual}\text{ Eq. 3-7})$$

$$= \frac{96.0\text{ kips}}{0.85(4\text{ ksi})(7.50\text{ ft})(12\text{ in./ft})}$$

$$= 0.314\text{ in.} < 1.00\text{ in.} \quad \textbf{o.k.}$$

From AISC *Manual* Table 6-2, the available shear strength of the W16×26 bare steel beam is determined as follows:

LRFD	ASD
$\phi_v V_n = 106\text{ kips} > 30.0\text{ kips}$ **o.k.**	$\dfrac{V_n}{\Omega_v} = 70.5\text{ kips} > 21.6\text{ kips}$ **o.k.**

Check live load deflection

$$\frac{L}{360} = \frac{(30\text{ ft})(12\text{ in./ft})}{360}$$

$$= 1.00\text{ in.}$$

Entering AISC *Manual* Table 3-20 for a W16×26, with PNA location 7 and $Y2$ = 5.50 in., provides a lower bound moment of inertia of I_{LB} = 575 in.^{4} From the equation previously derived, the live load deflection is determined as follows:

$$\Delta_{LL} = \frac{w_L L^4}{1{,}290 I_{LB}}$$

$$= \frac{(0.690\text{ kip/ft})(30\text{ ft})^4}{1{,}290\left(575\text{ in.}^4\right)}$$

$$= 0.753\text{ in.} < 1.00\text{ in.} \quad \textbf{o.k.}$$

From AISC Design Guide 3, limit the live load deflection, using 50% of the (unreduced) design live load, to L/360 with a maximum absolute value of 1 in. across the bay. From the equation previously derived, the deflection is determined as follows:

$$\Delta_{LL} = \frac{0.5(0.800\text{ kip/ft})(30\text{ ft})^4}{1{,}290\left(575\text{ in.}^4\right)}$$

$$= 0.437\text{ in.} < 1\text{ in.} \quad \textbf{o.k.}$$

$$1\text{ in.} - 0.437\text{ in.} = 0.563\text{ in.}$$

Note: Limit the supporting girders to 0.563 in. deflection under the same load combination at the connection point of the beam.

Determine the required number of shear stud connectors

From AISC *Manual* Table 3-21, using perpendicular deck with one ¾-in.-diameter anchor per rib in normal weight concrete with $f_c' = 4$ ksi in the weak position:

$$Q_n = 17.2\text{ kips/anchor}$$

$$
\begin{aligned}
n &= \frac{\Sigma Q_n}{Q_n} \\
&= \frac{96.0 \text{ kips}}{17.2 \text{ kips/anchor}} \\
&= 5.58 \text{ anchors (on each side of maximum moment)}
\end{aligned}
$$

Note: Per AISC *Specification* Section I8.2d, there is a maximum spacing limit of 8(6 in.) = 48 in. (not to exceed 36 in.) between anchors.

Therefore use 12 anchors, uniformly spaced at no more than 36 in. on center. Per AISC *Specification* Commentary Section I3.2d.1, beams with spans not exceeding 30 ft are not susceptible to connector failure due to insufficient connector capacity.

Note: Although the studs may be placed up to 36 in. on center, the steel deck must still be anchored to the supporting member at a spacing not to exceed 18 in. per AISC *Specification* Section I3.2c.

From AISC Design Guide 3, limit the wet concrete deflection in a bay to *L*/360, not to exceed 1 in. From the equation previously derived, the wet concrete deflection is determined as follows:

$$
\begin{aligned}
\Delta_{DL(wet\ conc)} &= \frac{(0.650 \text{ kip/ft})(30 \text{ ft})^4}{1{,}290\left(301 \text{ in.}^4\right)} \\
&= 1.36 \text{ in.}
\end{aligned}
$$

Camber the beam for 80% of the calculated wet concrete dead load deflection.

$$
\begin{aligned}
Camber &= 0.80(1.36 \text{ in.}) \\
&= 1.09 \text{ in.}
\end{aligned}
$$

Round the calculated value down to the nearest ¼ in. Therefore, specify 1 in. of camber.

$$1.36 \text{ in.} - 1 \text{ in.} = 0.360 \text{ in.}$$

$$1.00 \text{ in.} - 0.360 \text{ in.} = 0.640 \text{ in.}$$

Note: Limit the supporting girders to 0.640 in. deflection under the same load combination at the connection point of the beam.

This beam could also be designed as a noncomposite beam.

Try a W18×35. From AISC *Manual* Table 6-2 the available flexural strength for a fully braced beam, L_b = 0 ft, and shear strength are determined as follows:

LRFD	ASD
$\phi_b M_n = 249 \text{ kip-in.} > 225 \text{ kip-ft}$ **o.k.**	$\frac{M_n}{\Omega_b} = 166 \text{ kip-in.} > 162 \text{ kip-ft}$ **o.k.**
$\phi_v V_n = 159 \text{ kips} > 30.0 \text{ kips}$ **o.k.**	$\frac{V_n}{\Omega_v} = 106 \text{ kips} > 21.6 \text{ kips}$ **o.k.**

Check beam deflections

Check live load deflection. From AISC *Manual* Table 3-2 for a W18×35:

$I_x = 510 \text{ in.}^4$

$$\begin{aligned}\Delta_{LL} &= \frac{(0.690 \text{ kip/ft})(30 \text{ ft})^4}{1{,}290(510 \text{ in.}^4)} \\ &= 0.850 \text{ in.} < 1 \text{ in.} \quad \textbf{o.k.}\end{aligned}$$

Based on AISC Design Guide 3, limit the live load deflection, using 50% of the (unreduced) design live load, to *L*/360 with a maximum absolute value of 1 in. across the bay. From the equation previously derived, the deflection is determined as follows:

$$\begin{aligned}\Delta_{LL} &= \frac{0.5(0.800 \text{ kip/ft})(30 \text{ ft})^4}{1{,}290(510 \text{ in.}^4)} \\ &= 0.492 \text{ in.} < 1 \text{ in.} \quad \textbf{o.k.}\end{aligned}$$

$$1 \text{ in.} - 0.492 \text{ in.} = 0.508 \text{ in.}$$

Note: Limit the supporting girders to 0.508 in. deflection under the same load combination at the connection point of the beam.

Note: Because this beam is stronger than the W16×26 composite beam, no wet concrete strength checks are required in this example.

From AISC Design Guide 3, limit the wet concrete deflection in a bay to *L*/360, not to exceed 1 in. From the equation previously derived, the wet concrete deflection is determined as follows:

$$\begin{aligned}\Delta_{DL(wet\ conc)} &= \frac{(0.650 \text{ kip/ft})(30 \text{ ft})^4}{1{,}290(510 \text{ in.}^4)} \\ &= 0.800 \text{ in.} < 1 \text{ in.} \quad \textbf{o.k.}\end{aligned}$$

Camber the beam for 80% of the calculated wet concrete deflection.

$$\begin{aligned}Camber &= 0.80(0.800 \text{ in.}) \\ &= 0.640 \text{ in.}\end{aligned}$$

A good break point to eliminate camber is ¾ in.; therefore, do not specify a camber for this beam.

$$1 \text{ in.} - 0.800 \text{ in.} = 0.200 \text{ in.}$$

Note: Limit the supporting girders to 0.200 in. deflection under the same load case at the connection point of the beam.

Therefore, selecting a W18×35 will eliminate both shear studs and cambering. The cost of the extra steel weight may be offset by the elimination of studs and cambering. Local labor and material costs should be checked to make this determination.

SELECT TYPICAL NORTH-SOUTH EDGE BEAM

The influence area, $K_{LL}A_T$, for these beams is less than 400 ft^2; therefore, no live load reduction can be taken per ASCE/SEI 7, Section 4.7.2.

These beams carry 5.5 ft of dead load and live load as well as a wall load.

The floor dead load is:

$$\begin{aligned} w &= (5.5\text{ ft})\left(0.075\text{ kip/ft}^2\right) \\ &= 0.413\text{ kip/ft} \end{aligned}$$

Use 65 psf for the initial dead load due to the wet concrete:

$$\begin{aligned} w_{D(initial)} &= (5.5\text{ ft})\left(0.065\text{ kip/ft}^2\right) \\ &= 0.358\text{ kip/ft} \end{aligned}$$

Use 10 psf for the superimposed dead load:

$$\begin{aligned} w_{D(super)} &= (5.5\text{ ft})\left(0.010\text{ kip/ft}^2\right) \\ &= 0.055\text{ kip/ft} \end{aligned}$$

The dead load of the wall system at the floor is:

$$\begin{aligned} w &= (7.50\text{ ft})\left(0.055\text{ kip/ft}^2\right) + (6.00\text{ ft})\left(0.015\text{ kip/ft}^2\right) \\ &= 0.413\text{ kip/ft} + 0.090\text{ kip/ft} \\ &= 0.503\text{ kip/ft} \end{aligned}$$

The total dead load is:

$$\begin{aligned} w_D &= 0.413\text{ kip/ft} + 0.503\text{ kip/ft} \\ &= 0.916\text{ kip/ft} \end{aligned}$$

The live load is:

$$\begin{aligned} w_L &= (5.5\text{ ft})\left(0.080\text{ kip/ft}^2\right) \\ &= 0.440\text{ kip/ft} \end{aligned}$$

Beam loading is shown in Figure III-11.

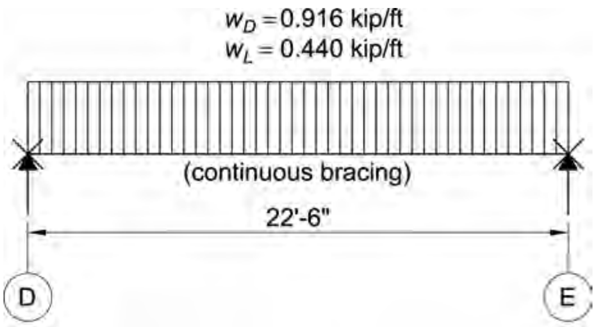


Fig. III-11. Loading and bracing diagram for typical north-south floor beams.

Calculate the required strengths from ASCE/SEI 7, Chapter 2:

LRFD	ASD
$w_u = 1.2(0.916\text{ kip/ft}) + 1.6(0.440\text{ kip/ft})$ $= 1.80\text{ kip/ft}$	$w_a = 0.916\text{ kip/ft} + 0.440\text{ kip/ft}$ $= 1.36\text{ kip/ft}$
$R_u = (1.80\text{ kip/ft})\left(\dfrac{22.5\text{ ft}}{2}\right)$ $= 20.3\text{ kips}$	$R_a = (1.36\text{ kip/ft})\left(\dfrac{22.5\text{ ft}}{2}\right)$ $= 15.3\text{ kips}$
$M_u = \dfrac{(1.80\text{ kip/ft})(22.5\text{ ft})^2}{8}$ $= 114\text{ kip-ft}$	$M_a = \dfrac{(1.36\text{ kip/ft})(22.5\text{ ft})^2}{8}$ $= 86.1\text{ kip-ft}$

Because these beams are less than 25 ft long, they will be most efficient as noncomposite beams. The beams at the edges of the building carry a brick spandrel panel. For these beams, the cladding weight exceeds 25% of the total dead load on the beam. From AISC Design Guide 3, limit the vertical deflection due to cladding and initial dead load to *L*/600 or ⅜ in. maximum. In addition, because these beams are supporting brick above and there is continuous glass below, limit the superimposed dead and live load deflection to *L*/600 or 0.3 in. maximum to accommodate the brick and *L*/360 or ¼ in. maximum to accommodate the glass. Therefore, combining the two limitations, limit the superimposed dead and live load deflection to *L*/600 or ¼ in. The superimposed dead load includes all of the dead load that is applied after the cladding has been installed. Note that it is typically not recommended to camber beams supporting spandrel panels.

Using the equation for deflection derived previously, the minimum moment of inertia, $I_{x\ req}$, to limit the superimposed dead and live load deflection to ¼ in.

$$I_{x\ req} = \frac{(0.055\text{ kip/ft} + 0.440\text{ kip/ft})(22.5\text{ ft})^4}{1{,}290(\tfrac{1}{4}\text{ in.})}$$
$$= 393\text{ in.}^4$$

Using the equation for deflection derived previously, the minimum moment of inertia, $I_{x\ req}$, to limit the cladding and initial dead load deflection to ⅜ in.

$$I_{x\ req} = \frac{(0.358\text{ kip/ft} + 0.503\text{ kip/ft})(22.5\text{ ft})^4}{1{,}290(\tfrac{3}{8}\text{ in.})}$$
$$= 456\text{ in.}^4 \quad \textbf{controls}$$

From AISC *Manual* Table 3-2, find a beam with $I_x \geq 456$ in.^{4} Select a W18×35 with $I_x = 510$ in.^{4}

From AISC *Manual* Table 6-2, the available flexural strength for a fully braced beam, $L_b = 0$ ft, and shear strength are determined as follows:

LRFD	ASD
$\phi_b M_n = 249 \text{ kip-in.} > 114 \text{ kip-ft}$ **o.k.** $\phi_v V_n = 159 \text{ kips} > 20.3 \text{ kips}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = 166 \text{ kip-in.} > 86.1 \text{ kip-ft}$ **o.k.** $\dfrac{V_n}{\Omega_v} = 106 \text{ kips} > 15.3 \text{ kips}$ **o.k.**

SELECT TYPICAL EAST-WEST EDGE GIRDER

The beams along the sides of the building carry the spandrel panel and glass, and dead load and live load from the intermediate floor beams. For these beams, the cladding weight exceeds 25% of the total dead load on the beam. Therefore, per AISC Design Guide 3, limit the vertical deflection due to cladding and initial dead load to $L/600$ or ⅜ in. maximum. In addition, because these beams are supporting brick above and there is continuous glass below, limit the superimposed dead and live load deflection to $L/600$ or 0.3 in. maximum to accommodate the brick and $L/360$ or ¼ in. maximum to accommodate the glass. Therefore, combining the two limitations, limit the superimposed dead and live load deflection to $L/600$ or ¼ in. The superimposed dead load includes all of the dead load that is applied after the cladding has been installed. These beams will be part of the moment frames on the north and south sides of the building and therefore will be designed as fixed at both ends.

Establish the loading.

The dead load reaction from the floor beams is:

$$\begin{aligned} P_D &= (0.750\text{ kip/ft})\left(\frac{45\text{ ft}}{2}\right) \\ &= 16.9\text{ kips} \end{aligned}$$

$$\begin{aligned} P_{D(initial)} &= (0.650\text{ kip/ft})\left(\frac{45\text{ ft}}{2}\right) \\ &= 14.6\text{ kips} \end{aligned}$$

$$\begin{aligned} P_{D(super)} &= (0.100\text{ kip/ft})\left(\frac{45\text{ ft}}{2}\right) \\ &= 2.25\text{ kips} \end{aligned}$$

The uniform dead load along the beam is:

$$\begin{aligned} w_D &= (0.5\text{ ft})\left(0.075\text{ kip/ft}^2\right) + 0.503\text{ kip/ft} \\ &= 0.541\text{ kip/ft} \end{aligned}$$

$$\begin{aligned} w_{D(initial)} &= (0.5\text{ ft})\left(0.065\text{ kip/ft}^2\right) \\ &= 0.033\text{ kip/ft} \end{aligned}$$

$$\begin{aligned} w_{D(super)} &= (0.5\text{ ft})\left(0.010\text{ kip/ft}^2\right) \\ &= 0.005\text{ kip/ft} \end{aligned}$$

Select typical 30-ft composite (or noncomposite) girders.

Check for possible live load reduction due to area in accordance with ASCE/SEI 7, Section 4.7.2.

From ASCE/SEI 7, Table 4.7-1, for edge beams with cantilevered slabs:

$$K_{LL} = 1$$

However, it is also permissible to calculate the value of K_{LL} based upon influence area. Because the cantilever dimension is small, K_{LL} will be closer to 2 than 1. The calculated value of K_{LL} based upon the influence area is:

$$K_{LL} = \frac{(45.5\text{ ft})(30\text{ ft})}{\left(\frac{45\text{ ft}}{2}+0.5\text{ ft}\right)(30\text{ ft})}$$
$$= 1.98$$

$$A_T = (30\text{ ft})(22.5\text{ ft}+0.5\text{ ft})$$
$$= 690\text{ ft}^2$$

From ASCE/SEI 7, Equation 4.7-1:

$$L = L_o\left(0.25+\frac{15}{\sqrt{K_{LL}A_T}}\right) \geq 0.50L_o$$
$$= (80\text{ psf})\left(0.25+\frac{15}{\sqrt{1.98\left(690\text{ ft}^2\right)}}\right) > 0.50(80\text{ psf})$$
$$= 52.5\text{ psf} > 40.0\text{ psf}$$

Therefore, use L = 52.5 psf.

The live load from the floor beams is:

$$P_L = (0.525\text{ kip/ft})\left(\frac{45\text{ ft}}{2}\right)$$
$$= 11.8\text{ kips}$$

The uniform live load along the beam is:

$$w_L = (0.5\text{ ft})\left(0.0526\text{ kip/ft}^2\right)$$
$$= 0.0263\text{ kip/ft}$$

The loading diagram is shown in Figure III-12.

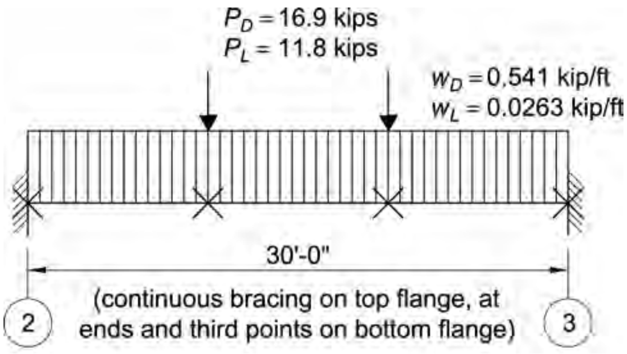


Fig. III-12. Loading and bracing diagram for typical east-west edge girders.

The required moment and end reactions at the floor side beams are determined from a structural analysis of a fixed-end beam and summarized as follows:

LRFD	ASD
Typical side beam:	Typical side beam:
$R_u = 49.5$ kips	$R_a = 37.2$ kips
$M_u\,(\text{at ends}) = 313$ kip-ft	$M_a\,(\text{at ends}) = 234$ kip-ft
$M_u\,(\text{at center}) = 156$ kip-ft	$M_a\,(\text{at center}) = 117$ kip-ft

The maximum moment occurs at the support with compression in the bottom flange. The bottom flange is laterally braced at 10 ft on center by the intermediate beams.

Note: During concrete placement, because the deck is parallel to the beam, the beam will not have continuous lateral support. It will be braced at 10 ft on center by the intermediate beams. By inspection, this condition will not control because the maximum moment under full loading causes compression in the bottom flange, which is braced at 10 ft on center.

LRFD	ASD
Calculate C_b for compression in the bottom flange braced at 10 ft on center.	Calculate C_b for compression in the bottom flange braced at 10 ft on center.
C_b = 2.21 (from computer output)	C_b = 2.22 (from computer output)
Select a W21×44.	Select a W21×44.
With continuous bracing, L_b = 0 ft, from AISC *Manual* Table 6-2:	With continuous bracing, L_b = 0 ft, from AISC *Manual* Table 6-2:
$\phi_b M_n = 358\text{ kip-ft} > 156\text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = 238\text{ kip-ft} > 117\text{ kip-ft}$ **o.k.**
From AISC *Manual* Table 6-2 with L_b = 10 ft and C_b = 2.21:	From AISC *Manual* Table 6-2 with L_b = 10 ft and C_b = 2.22:
$\phi_b M_n C_b = (264\text{ kip-ft})(2.21)$ $= 583\text{ kip-ft}$	$\dfrac{M_n}{\Omega_b} C_b = (176\text{ kip-ft})(2.22)$ $= 391\text{ kip-ft}$
From AISC *Specification* Section F2.2, the nominal flexural strength is limited to M_p.	From AISC *Specification* Section F2.2, the nominal flexural strength is limited to M_p.
$\phi_b M_n \le \phi_b M_p$ $583\text{ kip-ft} > 358\text{ kip-ft}$	$\dfrac{M_n}{\Omega_b} \le \dfrac{M_p}{\Omega_b}$ $391\text{ kip-ft} > 238\text{ kip-ft}$

LRFD	ASD
Therefore: $\phi_b M_n = 358 \text{ kip-ft} > 313 \text{ kip-ft}$ **o.k.**	Therefore: $\dfrac{M_n}{\Omega_b} = 238 \text{ kip-ft} > 234 \text{ kip-ft}$ **o.k.**

From AISC *Manual* Table 6-2, the available shear strength is determined as follows:

LRFD	ASD
$\phi_v V_n = 217 \text{ kips} > 49.5 \text{ kips}$ **o.k.**	$\dfrac{V_n}{\Omega_v} = 145 \text{ kips} > 37.2 \text{ kips}$ **o.k.**

Deflections are determined from a structural analysis of a fixed-end beam. For deflection due to cladding and initial dead load:

$\Delta = 0.295 \text{ in.} < \tfrac{3}{8} \text{ in.}$ **o.k.**

For deflection due to superimposed dead and live loads:

$\Delta = 0.212 \text{ in.} < \tfrac{1}{4} \text{ in.}$ **o.k.**

Note that both of the deflection criteria stated previously for the girder and for the locations on the girder where the floor beams are supported have also been met.

Also, as noted previously, it is not typically recommended to camber beams supporting spandrel panels. The W21×44 is adequate for strength and deflection, but may be increased in size to help with moment frame strength or drift control.

SELECT TYPICAL EAST-WEST INTERIOR GIRDER

Establish loads

The dead load reaction from the floor beams is:

$$P_D = (0.750\text{ kip/ft})\left(\frac{45\text{ ft}+30\text{ ft}}{2}\right)$$
$$= 28.1\text{ kips}$$

Check for live load reduction due to area in accordance with ASCE/SEI 7, Section 4.7.2.

From ASCE/SEI 7, Table 4.7-1, for interior beams:

$$K_{LL} = 2$$

$$A_T = (30\text{ ft})\left(\frac{45\text{ ft}+30\text{ ft}}{2}\right)$$
$$= 1{,}130\text{ ft}^2$$

Using ASCE/SEI 7, Equation 4.7-1:

$$L = L_o\left(0.25+\frac{15}{\sqrt{K_{LL}A_T}}\right) \ge 0.50L_o$$
$$= (80\text{ psf})\left(0.25+\frac{15}{\sqrt{(2)(1{,}130\text{ ft}^2)}}\right) \ge 0.50(80\text{ psf})$$
$$= 45.2\text{ psf} > 40.0\text{ psf}$$

Therefore, use L = 45.2 psf.

The live load from the floor beams is:

$$P_L = (0.0452\text{ kip/ft}^2)\left(\frac{45\text{ ft}+30\text{ ft}}{2}\right)(10\text{ ft})$$
$$= 17.0\text{ kips}$$

The loading is shown in Figure III-13.

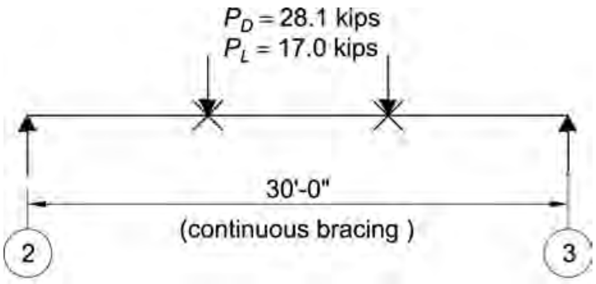


Fig. III-13. Loading and bracing diagram for typical interior girder.

Note: The dead load for this beam is included in the assumed overall dead load.

From ASCE/SEI 7, Chapter 2, the required strengths are determined as follows:

LRFD	ASD
$R_u = 1.2(28.1\text{ kips}) + 1.6(17.0\text{ kips})$ $= 60.9\text{ kips}$ $M_u = (60.9\text{ kips})(10\text{ ft})$ $= 609\text{ kip-ft}$	$R_a = 28.1\text{ kips} + 17.0\text{ kips}$ $= 45.1\text{ kips}$ $M_a = (45.1\text{ kips})(10\text{ ft})$ $= 451\text{ kip-ft}$

Check for beam requirements when carrying wet concrete. Limit wet concrete deflection to 1½ in.

$$\begin{aligned} P_D &= (0.650\text{ kip/ft})\left(\frac{45\text{ ft} + 30\text{ ft}}{2}\right) \\ &= 24.4\text{ kips} \end{aligned}$$

$$\begin{aligned} P_L &= (0.200\text{ kip/ft})\left(\frac{45\text{ ft} + 30\text{ ft}}{2}\right) \\ &= 7.50\text{ kips} \end{aligned}$$

Note: During concrete placement, because the deck is parallel to the beam, the beam will not have continuous lateral support. It will be braced at 10 ft on center by the intermediate beams. Also, during concrete placement, a construction live load of 20 psf will be present. The loading is shown in Figure III-14.

From ASCE/SEI 7, Chapter 2, the required strengths for the typical interior beams with wet concrete only is determined as follows:

LRFD	ASD
$R_u = 1.4(24.4\text{ kips})$ $= 34.2\text{ kips}$ $M_u = (34.2\text{ kips})(10\text{ ft})$ $= 342\text{ kip-ft}$	$R_a = 24.4\text{ kips}$ $M_a = (24.4\text{ kips})(10\text{ ft})$ $= 244\text{ kip-ft}$

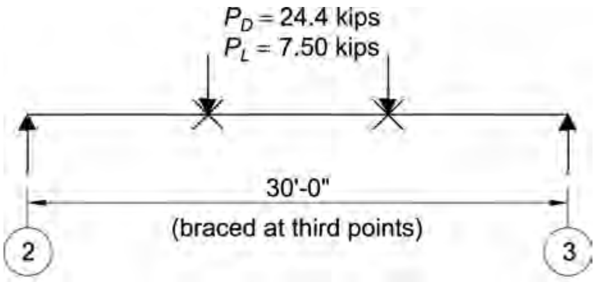


Fig. III-14. Loading and bracing diagram for typical interior girder with wet concrete and construction loads.

From ASCE/SEI 7, Chapter 2, the required strengths for the typical interior beams with wet concrete and construction live load is determined as follows:

LRFD	ASD
$R_u = 1.2(24.4\text{ kips}) + 1.6(7.50\text{ kips})$ $= 41.3\text{ kips}$ $M_u(\text{midspan}) = (41.3\text{ kips})(10\text{ ft})$ $= 413\text{ kip-ft}$	$R_a = 24.4\text{ kips} + 7.50\text{ kips}$ $= 31.9\text{ kips}$ $M_a(\text{midspan}) = (31.9\text{ kips})(10\text{ ft})$ $= 319\text{ kip-ft}$

Assume $I_x \geq 935\text{ in.}^4$, which is determined based on a wet concrete deflection of 1½ in. From AISC *Manual* Table 3-2, select a W21×68 with $I_x = 1{,}480\text{ in.}^4$.

From AISC *Manual* Table 6-2, verify the available flexural strength and shear strength using $L_b = 10$ ft, and $C_b = 1.0$.

LRFD	ASD
$\phi_b M_n = 532\text{ kip-ft} > 413\text{ kip-ft}$ **o.k.** $\phi_v V_n = 272\text{ kips} > 41.3\text{ kips}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = 354\text{ kip-ft} > 319\text{ kip-ft}$ **o.k.** $\dfrac{V_n}{\Omega_v} = 181\text{ kips} > 31.9\text{ kips}$ **o.k.**

Check W21×68 as a composite beam.

From previous calculations:

LRFD	ASD
$R_u = 60.9\text{ kips}$ $M_u(\text{midspan}) = 609\text{ kip-ft}$	$R_a = 45.1\text{ kips}$ $M_a(\text{midspan}) = 451\text{ kip-ft}$

From previous calculations, assuming a = 1 in.:

$$Y2 = 5.50\text{ in.}$$

Enter AISC *Manual* Table 3-19 for a W21×68 with $Y2$ = 5.50 in. Selecting PNA location 7 with ΣQ_n = 250 kips provides an available flexural strength of:

LRFD	ASD
$\phi_b M_n = 844\text{ kip-ft} > 609\text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega_b} = 561\text{ kip-ft} > 451\text{ kip-ft}$ **o.k.**

From AISC Design Guide 3, limit the wet concrete deflection in a bay to L/360, not to exceed 1 in. From AISC *Manual* Table 3-23, Case 9:

$$\begin{aligned}\Delta_{DL(wet\ conc)} &= \frac{23P_L L^3}{648EI} \\ &= \frac{23(24.4\text{ kips})(30\text{ ft})^3(12\text{ in./ft})^3}{648(29{,}000\text{ ksi})(1{,}480\text{ in.}^4)} \\ &= 0.941\text{ in.}\end{aligned}$$

Camber the beam for 80% of the calculated wet concrete deflection.

$$\begin{aligned}Camber &= 0.80(0.941\text{ in.}) \\ &= 0.753\text{ in.}\end{aligned}$$

Round the calculated value down to the nearest ¼ in,: therefore, specify ¾-in. of camber.

$$0.941\text{ in.} - ¾\text{ in.} = 0.191\text{ in.} < 0.400\text{ in.}$$

Therefore, the total deflection limit of 1 in. for the bay has been met.

Determine the effective width, b

From AISC *Specification* Section I3.1a, the effective width of the concrete slab is the sum of the effective widths for each side of the beam centerline, which shall not exceed:

1. one-eighth of the span of the beam, center-to-center of supports

   $$\left(\frac{30\text{ ft}}{8}\right)(2\text{ sides}) = 7.50\text{ ft}\quad \textbf{controls}$$

2. one-half the distance to the centerline of the adjacent beam

   $$\left(\frac{45\text{ ft}}{2} + \frac{30\text{ ft}}{2}\right) = 37.5\text{ ft}$$

3. the distance to the edge of the slab

   The latter is not applicable for an interior member.

Determine the height of the compression block, a

$$\begin{aligned}a &= \frac{\sum Q_n}{0.85 f_c{}'b} \\ &= \frac{250\text{ kips}}{0.85(4\text{ ksi})(7.50\text{ ft})(12\text{ in./ft})} \\ &= 0.817\text{ in.} < 1\text{ in.}\quad \textbf{o.k.}\end{aligned}$$

(*Manual* Eq. 3-7)

From AISC *Manual* Table 6-2, the available shear strength of the W21×68 is determined as follows.

LRFD	ASD
$\phi_v V_n = 272\text{ kips} > 60.9\text{ kips}$ **o.k.**	$\dfrac{V_n}{\Omega_v} = 181\text{ kips} > 45.1\text{ kips}$ **o.k.**

Check live load deflection.

$$\begin{aligned}\Delta_{LL} &= \frac{L}{360}\\ &= \frac{(30\text{ ft})(12\text{ in./ft})}{360}\\ &= 1.00\text{ in.}\end{aligned}$$

Entering AISC *Manual* Table 3-20 for a W21×68, with PNA location 7 and $Y2$ = 5.50 in., provides a lower bound moment of inertia of I_{LB} = 2,510 in.^{4}

$$\begin{aligned}\Delta_{LL} &= \frac{23P_L L^3}{648EI_{LB}}\\ &= \frac{23(17.0\text{ kips})(30\text{ ft})^3(12\text{ in./ft})^3}{648(29{,}000\text{ ksi})(2{,}510\text{ in.}^4)}\\ &= 0.387\text{ in.} < 1.00\text{ in.} \quad \textbf{o.k.}\end{aligned}$$

From AISC Design Guide 3, limit the live load deflection, using 50% of the (unreduced) design live load, to L/360 with a maximum absolute value of 1 in. across the bay.

The maximum deflection is:

$$\begin{aligned}\Delta_{LL} &= \frac{23(0.5)(30.0\text{ kips})(30\text{ ft})^3(12\text{ in./ft})^3}{648(29{,}000\text{ ksi})(2{,}510\text{ in.}^4)}\\ &= 0.341\text{ in.} < 1.00\text{ in.} \quad \textbf{o.k.}\end{aligned}$$

Check the deflection at the location where the floor beams are supported.

$$\begin{aligned}\Delta_{LL} &= \frac{0.5(30.0\text{ kips})(120\text{ in.})}{6(29{,}000\text{ ksi})(2{,}510\text{ in.}^4)}\left[3(360\text{ in.})(120\text{ in.}) - 4(120\text{ in.})^2\right]\\ &= 0.297\text{ in.} > 0.265\text{ in.} \quad \textbf{o.k.}\end{aligned}$$

Therefore, the total deflection in the bay is 0.297 in. + 0.735 in. = 1.03 in., which is acceptably close to the limit of 1 in, where Δ_{LL} = 0.735 in. is from the 45 ft interior composite beam running north-south.

Determine the required shear stud connectors

Using *Manual* Table 3-21, for parallel deck with, $w_r/h_r \geq 1.5$, one ¾-in.-diameter stud in normal weight, 4-ksi concrete:

$$Q_n = 21.5\text{ kips/anchor}$$

$$\begin{aligned}\frac{\sum Q_n}{Q_n} &= \frac{250\text{ kips}}{21.5\text{ kips/anchor}}\\ &= 11.6\text{ anchors (on each side of maximum moment)}\end{aligned}$$

Therefore, use a minimum of 24 studs for horizontal shear.

Per AISC *Specification* Section I8.2d, the maximum stud spacing is 36 in.

Since the load is concentrated at ⅓ points, the studs are to be arranged as follows:

Use 12 studs between supports and supported beams at third points. Between supported beams (middle third of span), use 4 studs to satisfy minimum spacing requirements.

Therefore, 28 studs are required in a 12:4:12 arrangement.

Notes: Although the studs may be placed up to 36 in. on center, the steel deck must still be anchored to be the supporting member at a spacing not to exceed 18 in. in accordance with AISC *Specification* Section I3.2c.

This W21×68 beam, with full lateral support, is very close to having sufficient available strength to support the imposed loads without composite action. A larger noncomposite beam might be a better solution.

COLUMN DESIGN AND SELECTION FOR GRAVITY LOADS

Estimate column loads

Roof loads (from previous calculations):

Dead load	= 20 psf
Snow load	= 25 psf
Total	= 45 psf

The snow drift loads at the perimeter of the roof and at the mechanical screen wall are developed from previous calculations.

Reaction to column (side parapet):

$$\begin{aligned} w &= \left(\frac{3.73\text{ kips}}{6.00\text{ ft}}\right) - \left(0.025\text{ kip/ft}^2\right)\left(23.0\text{ ft}\right) \\ &= 0.0467\text{ kip/ft} \end{aligned}$$

where 3.73 kips is the snow load reaction, including drift, from the 24KCS4 roof joist at the side parapet.

Reaction to column (end parapet):

$$\begin{aligned} w &= \left(\frac{16.0\text{ kips}}{37.5\text{ ft}}\right) - \left(0.025\text{ kip/ft}^2\right)\left(15.5\text{ ft}\right) \\ &= 0.0392\text{ kip/ft} \end{aligned}$$

where 16.0 kips is the snow load reaction, including drift, from the W21×44 roof beam along the interior lines of the building.

Reaction to column (screen wall along lines C & D):

$$\begin{aligned} w &= \left(\frac{4.02\text{ kips}}{6\text{ ft}}\right) - \left(0.025\text{ kip/ft}^2\right)\left(22.5\text{ ft}\right) \\ &= 0.108\text{ kip/ft} \end{aligned}$$

where 4.02 kips is the snow load reaction, including drift, from the 24KCS4 joist at the screen wall.

Mechanical equipment and screen wall (average):

w = 40 psf

The spandrel panel weight was calculated as 0.413 kip/ft as part of the selection process for the W16×26 roof beams at the east and west ends of the building.

The mechanical room dead load of 0.060 kip/ft^2 and snow load of 0.040 kip/ft^2 was determined as part of the selection process for the W14×22 roof beams at the mechanical area.

A summary of the column loads at the roof is given in Table III-2.

Table III-2
Summary of Column Loads at the Roof

Column	Loading Width, ft	Loading Length, ft	Area, ft^2	DL, kip/ft^2	P_D, kips	SL, kip/ft^2	P_S, kips
2A, 2F, 3A, 3F, 4A, 4F, 5A, 5F, 6A ,6F, 7A, 7F	23.0	30.0	690	0.020	13.8	0.025	17.3
Snow drifting at side		30.0				0.0467 kip/ft	1.40
Exterior wall		30.0		0.413 kip/ft	12.4		
					26.2		**18.7**
1B, 1E, 8B, 8E	3.50	22.5	78.8	0.020	1.58	0.025	1.97
Snow drifting at side		22.5				0.0392 kip/ft	0.882
Exterior wall		22.5		0.413 kip/ft	9.29		
					10.9		**2.85**
1A, 1F, 8A, 8F	23.0	15.5	357 $-\dfrac{78.8\text{ ft}^2}{2}$ = 318	0.020	6.36	0.025	7.95
Snow drifting at end		11.8				0.0392 kip/ft	0.463
Snow drifting at side		15.5				0.0467 kip/ft	0.724
Exterior wall		27.3		0.413 kip/ft	11.3		
					17.7		**9.14**
1C, 1D, 8C, 8D	37.5	15.5	581 $-\dfrac{78.8\text{ ft}^2}{2}$ = 542	0.020	10.8	0.025	13.6
Snow drifting at end		26.3				0.0392 kip/ft	1.03
Exterior wall		26.3		0.413 kip/ft	10.9		
					21.7		**14.6**
2C, 2D, 7C, 7D	37.5	30.0	1,125	0.020	**22.5**	0.025	**28.1**
3C, 3D, 4C, 4D, 5C, 5D, 6C, 6D	22.5	30.0	675	0.020	13.5	0.025	16.9
Snow drifting		30.0				0.108 kip/ft	3.24
Mechanical area	15.0	30.0	450	0.060	27.0	0.040	18.0
					40.5		**38.1**

Floor loads (from previous calculations):

Dead load = 75 psf
Snow load = 80 psf
Total = 155 psf

Calculate reduction in live loads, analyzed at the base of three floors (n = 3) using ASCE/SEI 7, Section 4.7.2. Note that the 6-in. cantilever of the floor slab has been ignored for the calculation of K_{LL} for columns in this building because it has a negligible effect.

Columns: 2A, 2F, 3A, 3F, 4A, 4F, 5A, 5F, 6A, 6F, 7A, 7F
Exterior column without cantilever slabs
K_{LL} = 4 (ASCE/SEI 7, Table 4.7-1)
L_o = 80 psf
n = 3 (three floors supported)

$$A_T = (22.5\text{ ft} + 0.5\text{ ft})(30\text{ ft})$$
$$= 690\text{ ft}^2$$

Using ASCE/SEI 7, Equation 4.7-1:

$$L = L_o\left(0.25+\frac{15}{\sqrt{K_{LL}nA_T}}\right) \ge 0.4L_o$$

$$= (80\text{ psf})\left[0.25+\frac{15}{\sqrt{(4)(3)(690\text{ ft}^2)}}\right] > 0.4(80\text{ psf})$$

$$= 33.2\text{ psf} > 32.0\text{ psf}$$

Therefore, use L = 33.2 psf.

Columns: 1B, 1E, 8B, 8E
Exterior column without cantilever slabs
K_{LL} = 4 (ASCE/SEI 7, Table 4.7-1)
L_o = 80 psf
n = 3

$$A_T = (5.00\text{ ft}+0.5\text{ ft})(22.5\text{ ft})$$

$$= 124\text{ ft}^2$$

$$L = L_o\left(0.25+\frac{15}{\sqrt{K_{LL}nA_T}}\right) \ge 0.4L_o$$

$$= (80\text{ psf})\left[0.25+\frac{15}{\sqrt{(4)(3)(124\text{ ft}^2)}}\right] > 0.4(80\text{ psf})$$

$$= 51.1\text{ psf} > 32.0\text{ psf}$$

Use L = 51.1 psf.

Columns: 1A, 1F, 8A, 8F
Corner column without cantilever slabs
K_{LL} = 4 (ASCE/SEI 7, Table 4.7-1)
L_o = 80 psf
n = 3

$$A_T = (15.0\text{ ft}+0.5\text{ ft})(22.5\text{ ft}+0.5\text{ ft}) - \left(\frac{124\text{ ft}^2}{2}\right)$$

$$= 295\text{ ft}^2$$

$$L = L_o\left(0.25+\frac{15}{\sqrt{K_{LL}nA_T}}\right) \ge 0.4L_o$$

$$= (80\text{ psf})\left[0.25+\frac{15}{\sqrt{(4)(3)(295\text{ ft}^2)}}\right] > 0.4(80\text{ psf})$$

$$= 40.2\text{ psf} > 32.0\text{ psf}$$

Therefore, use L = 40.2 psf.

Columns: 1C, 1D, 8C, 8D
Exterior column without cantilever slabs
K_{LL} = 4 (ASCE/SEI 7, Table 4.7-1)
L_o = 80 psf
n = 3

$$\begin{aligned} A_T &= (15.0\text{ ft} + 0.5\text{ ft})\left(\frac{45\text{ ft} + 30\text{ ft}}{2}\right) - \left(\frac{124\text{ ft}^2}{2}\right) \\ &= 519\text{ ft}^2 \end{aligned}$$

$$\begin{aligned} L &= L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}nA_T}}\right) \geq 0.4L_o \\ &= (80\text{ psf})\left[0.25 + \frac{15}{\sqrt{(4)(3)(519\text{ ft}^2)}}\right] > 0.4(80\text{ psf}) \\ &= 35.2\text{ psf} > 32.0\text{ psf} \end{aligned}$$

Therefore, use L = 35.2 psf.

Columns: 2C, 2D, 3C, 3D, 4C, 4D, 5C, 5D, 6C, 6D, 7C, 7D
Interior column
K_{LL} = 4 (ASCE/SEI 7, Table 4.7-1)
L_o = 80 psf
n = 3

$$\begin{aligned} A_T &= \left(\frac{45\text{ ft} + 30\text{ ft}}{2}\right)(30\text{ ft}) \\ &= 1{,}125\text{ ft}^2 \end{aligned}$$

$$\begin{aligned} L &= L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}nA_T}}\right) \geq 0.4L_o \\ &= (80\text{ psf})\left[0.25 + \frac{15}{\sqrt{(4)(3)(1{,}125\text{ ft}^2)}}\right] < 0.4(80\text{ psf}) \\ &= 30.3\text{ psf} < 32.0\text{ psf} \end{aligned}$$

Therefore, use L = 32.0 psf.

A summary of the column loads at the floors is given in Table III-3.

Table III-3
Summary of Column Loads at the Floors

Column	Loading		Area,	DL,	P_D,	LL,	P_L,
	Width, ft	Length, ft	ft^2	kip/ft^2	kips	kip/ft^2	kips
2A, 2F, 3A, 3F, 4A, 4F, 5A, 5F, 6A ,6F, 7A, 7F	23.0	30.0	690	0.075	51.8	0.0332	22.9
Exterior wall		30.0		0.503 kip/ft	15.1		
					66.9		**22.9**
1B, 1E, 8B, 8E	5.50	22.5	124	0.075	9.30	0.0511	6.34
Exterior wall		22.5		0.503 kip/ft	11.3		
					20.6		**6.34**
1A, 1F, 8A, 8F	23.0	15.5	357 $-\dfrac{124\text{ ft}^2}{2}$ $= 295$	0.075	22.1	0.0402	11.9
Exterior wall		27.3		0.503 kip/ft	13.7		
					35.8		**11.9**
1C, 1D, 8C, 8D	37.5	15.5	581 $-\dfrac{124\text{ ft}^2}{2}$ $= 519$	0.075	38.9	0.0352	18.3
Exterior wall		26.3		0.503 kip/ft	13.2		
					52.1		**18.3**
2C, 2D, 3C, 3D, 4C, 4D, 5C, 5D, 6C, 6D, 7C, 7D	37.5	30.0	1,125	0.075	**84.4**	0.0320	**36.0**

The spandrel panel weight was calculated as 0.503 kip/ft as part of the selection process for the W18×35 edge beams at the north and south ends of the building.

The column loads are summarized in Table III-4.

Table III-4
Summary of Column Loads

Column	Floor	P_D, kips	P_L, kips	P_S, kips
2A, 2F, 3A, 3F, 4A, 4F, 5A, 5F, 6A, 6F, 7A, 7F	Roof	26.2		18.7
	4th	66.9	22.9	
	3rd	66.9	22.9	
	2nd	66.9	22.9	
	Total	**227**	**68.7**	**18.7**
1B, 1E, 8B, 8E	Roof	10.9		2.85
	4th	20.6	6.34	
	3rd	20.6	6.34	
	2nd	20.6	6.34	
	Total	**72.7**	**19.0**	**2.85**
1A, 1F, 8A, 8F	Roof	17.7		9.14
	4th	35.8	11.9	
	3rd	35.8	11.9	
	2nd	35.8	11.9	
	Total	**125**	**35.7**	**9.14**
1C, 1D, 8C, 8D	Roof	21.7		14.6
	4th	52.1	18.3	
	3rd	52.1	18.3	
	2nd	52.1	18.3	
	Total	**178**	**54.9**	**14.6**
2C, 2D, 7C, 7D	Roof	22.5		28.1
	4th	84.4	36.0	
	3rd	84.4	36.0	
	2nd	84.4	36.0	
	Total	**276**	**108**	**28.1**
3C, 3D, 4C, 4D, 5C, 5D, 6C, 6D	Roof	40.5		38.1
	4th	84.4	36.0	
	3rd	84.4	36.0	
	2nd	84.4	36.0	
	Total	**294**	**108**	**38.1**

SELECT TYPICAL INTERIOR LEANING COLUMNS

Columns 3C, 3D, 4C, 4D, 5C, 5D, 6C, 6D

Elevation of second floor slab: 113.5 ft
Elevation of first floor slab: 100 ft
Column unbraced length: $L_x = L_y = 13.5$ ft

Note: $K_x = K_y = 1.0$ for a leaning column when using the effective length method.

$$\begin{aligned} L_{cx} &= K_x L_x \\ &= 1.0(13.5 \text{ ft}) \\ &= 13.5 \text{ ft} \end{aligned}$$

$$\begin{aligned} L_{cy} &= K_y L_y \\ &= 1.0(13.5 \text{ ft}) \\ &= 13.5 \text{ ft} \end{aligned}$$

From ASCE/SEI 7, Chapter 2, the required axial strength is determined using the following controlling load combinations (including the 0.5 live load reduction permitted for LRFD):

LRFD	ASD
$P_u = 1.2(294 \text{ kips}) + 1.6(108 \text{ kips}) + 0.5(38.1 \text{ kips})$ $= 545 \text{ kips}$	$P_a = 294 \text{ kips} + 0.75(108 \text{ kips}) + 0.75(38.1 \text{ kips})$ $= 404 \text{ kips}$

Using AISC *Manual* Table 4-1a, enter with $L_c = 14.0$ ft (conservative) and proceed across the table until reaching the lightest size that has sufficient available strength at the required unbraced length. Select a W12×65. The available strength in axial compression is:

LRFD	ASD
$\phi_c P_n = 685 \text{ kips} > 545 \text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_c} = 456 \text{ kips} > 404 \text{ kips}$ **o.k.**

Note: A W14×68 would also be an acceptable selection.

Columns 2C, 2D, 7C, 7D

Elevation of second floor slab: 113.5 ft
Elevation of first floor slab: 100 ft
Column unbraced length: $L_x = L_y = 13.5$ ft

Note: $K_x = K_y = 1.0$ for a leaning column when using the effective length method.

$$\begin{aligned} L_{cx} &= K_x L_x \\ &= 1.0(13.5 \text{ ft}) \\ &= 13.5 \text{ ft} \end{aligned}$$

$$\begin{aligned} L_{cy} &= K_y L_y \\ &= 1.0(13.5 \text{ ft}) \\ &= 13.5 \text{ ft} \end{aligned}$$

From ASCE/SEI 7, Chapter 2, the required axial strength is determined using the following controlling load combinations (including the 0.5 live load reduction permitted for LRFD):

LRFD	ASD
$P_u = 1.2(276\text{ kips}) + 1.6(108\text{ kips}) + 0.5(28.1\text{ kips})$ $= 518\text{ kips}$	$P_a = 276\text{ kips} + 0.75(108\text{ kips}) + 0.75(28.1\text{ kips})$ $= 378\text{ kips}$

Using AISC *Manual* Table 4-1a, enter with L_c = 14.0 ft (conservative) and proceed across the table until reaching the lightest size that has sufficient available strength at the required unbraced length. Select a W12×65. The available strength in axial compression is:

LRFD	ASD
$\phi_c P_n = 685\text{ kips} > 518\text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_c} = 456\text{ kips} > 378\text{ kips}$ **o.k.**

Note: A W14×61 would also be an acceptable selection. However, W12×65 columns were selected to keep sizes consistent for all interior columns.

SELECT TYPICAL EXTERIOR LEANING COLUMNS

Columns 1B, 1E, 8B, 8E

Elevation of second floor slab: 113.5 ft
Elevation of first floor slab: 100 ft
Column unbraced length: $L_x = L_y = 13.5$ ft

Note: $K_x = K_y = 1.0$ for a leaning column when using the effective length method.

$$\begin{aligned} L_{cx} &= K_x L_x \\ &= 1.0(13.5\text{ ft}) \\ &= 13.5\text{ ft} \end{aligned}$$

$$\begin{aligned} L_{cy} &= K_y L_y \\ &= 1.0(13.5\text{ ft}) \\ &= 13.5\text{ ft} \end{aligned}$$

From ASCE/SEI 7, Chapter 2, the required axial strength is determined using the following controlling load combinations (including the 0.5 live load reduction permitted for LRFD):

LRFD	ASD
$P_u = 1.2(72.7\text{ kips}) + 1.6(19.0\text{ kips}) + 0.5(2.85\text{ kips})$ $= 119\text{ kips}$	$P_a = 72.7\text{ kips} + 0.75(19.0\text{ kips}) + 0.75(2.85\text{ kips})$ $= 89.1\text{ kips}$

Using AISC *Manual* Table 4-1a, enter with L_c = 14.0 ft (conservative) and proceed across the table until reaching the lightest size that has sufficient available strength at the required unbraced length. Select a W12×40. The available strength in axial compression is:

LRFD	ASD
$\phi_c P_n = 304\text{ kips} > 119\text{ kips}$ **o.k.**	$\dfrac{P_n}{\Omega_c} = 202\text{ kips} > 89.1\text{ kips}$ **o.k.**

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Note, A W12 column was selected above for ease of erection of framing beams (bolted double-angle connections can be used without bolt staggering). Final column selections at the moment and braced frames are illustrated later in this example.

WIND LOAD DETERMINATION

Use the Envelope Procedure for simple diaphragm buildings from ASCE/SEI 7, Chapter 28, Part 2.

To qualify for the simplified wind load method for low-rise buildings, per ASCE/SEI 7, Section 28.5.2, the following conditions must be met:

1. Simple diaphragm building **o.k.**
2. Low-rise building ≤ 60 ft **o.k.**
3. Enclosed, and conforms to wind borne debris provisions **o.k.**
4. Regular-shaped **o.k.**
5. Not a flexible building **o.k.**
6. Does not have response characteristics requiring special considerations **o.k.**
7. Symmetrical shape with flat or gable roof with $\theta \le 45^{\circ}$ **o.k.**
8. Torsional load cases from ASCE/SEI 7, Figure 28.3-1 do not control design of MWFRS **o.k.**

Define input parameters

1.	Risk category:	II from ASCE/SEI 7, Table 1.5-1
2.	Basic wind speed:	V = 107 mph (3-s) from ASCE/SEI 7, Figure 26.5-1B
3.	Exposure category:	C from ASCE/SEI 7, Section 26.7.3
4.	Topographic factor:	K_{zt} = 1.0 from ASCE/SEI 7, Section 26.8.2
5.	Mean roof height:	55.0 ft
6.	Height and exposure adjustment:	λ = 1.59 from ASCE/SEI 7, Figure 28.5-1
7.	Roof angle:	$\theta = 0^{\circ}$

$$p_s = \lambda K_{zt} p_{s30} \qquad \text{(ASCE/SEI 7, Eq. 28.5-1)}$$

$$\begin{aligned}
&= (1.59)(1.0)(18.2\text{ psf}) = 28.9\text{ psf (Horizontal pressure zone A)}\\
&= (1.59)(1.0)(12.0\text{ psf}) = 19.1\text{ psf (Horizontal pressure zone C)}\\
&= (1.59)(1.0)(-21.9\text{ psf}) = -34.8\text{ psf (Vertical pressure zone E)}\\
&= (1.59)(1.0)(-12.4\text{ psf}) = -19.7\text{ psf (Vertical pressure zone F)}\\
&= (1.59)(1.0)(-15.2\text{ psf}) = -24.2\text{ psf (Vertical pressure zone G)}\\
&= (1.59)(1.0)(-9.59\text{ psf}) = -15.2\text{ psf (Vertical pressure zone H)}
\end{aligned}$$

a = 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft

a = the lesser of:
  10% of the least horizontal dimension = 12.3 ft
  40% of the eave height = 22.0 ft
  but not less than 4% of the least horizontal dimension or 3 ft = 4.92 ft

Thus, a = 12.3 ft and 2a = 24.6 ft.

Zone A: End zone of wall (width = 2a)
Zone C: Interior zone of wall
Zone E: End zone of windward roof (width = 2a)
Zone F: End zone of leeward roof (width = 2a)
Zone G: Interior zone of windward roof
Zone H: Interior zone of leeward roof

Calculate load on roof diaphragm

Mechanical screen wall height: 6 ft
Wall height: 0.5[55 ft – 3(13.5 ft)] = 7.25 ft
Parapet wall height: 2 ft
Total wall height at roof at screen wall: $6\text{ ft}+7.25\text{ ft}=13.3\text{ ft}$
Total wall height at roof at parapet: $2\text{ ft}+7.25\text{ ft}=9.25\text{ ft}$

$$\begin{aligned} w_{s(A)} &= (28.9\text{ psf})(9.25\text{ ft}) \\ &= 267\text{ plf} \end{aligned}$$

$$\begin{aligned} w_{s(C)} &= (19.1\text{ psf})(9.25\text{ ft}) \\ &= 177\text{ plf (at parapet)} \end{aligned}$$

$$\begin{aligned} w_{s(C)} &= (19.1\text{ psf})(13.3\text{ ft}) \\ &= 254\text{ plf (at screen wall)} \end{aligned}$$

Calculate load on fourth floor diaphragm

Wall height: $0.5(55.0\text{ ft}-40.5\text{ ft})=7.25\text{ ft}$
$0.5(40.5\text{ ft}-27.0\text{ ft})=6.75\text{ ft}$
Total wall height at floor: $6.75\text{ ft}+7.25\text{ ft}=14.0\text{ ft}$

$$\begin{aligned} w_{s(A)} &= (28.9\text{ psf})(14.0\text{ ft}) \\ &= 405\text{ plf} \end{aligned}$$

$$\begin{aligned} w_{s(C)} &= (19.1\text{ psf})(14.0\text{ ft}) \\ &= 267\text{ plf} \end{aligned}$$

Calculate load on third floor diaphragm

Wall height: $0.5(40.5\text{ ft}-27.0\text{ ft})=6.75\text{ ft}$
$0.5(27.0\text{ ft}-13.5\text{ ft})=6.75\text{ ft}$
Total wall height at floor: $6.75\text{ ft}+6.75\text{ ft}=13.5\text{ ft}$

$$w_{s(A)} = (28.9\text{ psf})(13.5\text{ ft})$$
$$= 390\text{ plf}$$

$$w_{s(C)} = (19.1\text{ psf})(13.5\text{ ft})$$
$$= 258\text{ plf}$$

Calculate load on second floor diaphragm

Wall height: $0.5(27.0\text{ ft} - 13.5\text{ ft}) = 6.75\text{ ft}$

$0.5(13.5\text{ ft} - 0\text{ ft}) = 6.75\text{ ft}$

Total wall height at floor: $6.75\text{ ft} + 6.75\text{ ft} = 13.5\text{ ft}$

$$w_{s(A)} = (28.9\text{ psf})(13.5\text{ ft})$$
$$= 390\text{ plf}$$

$$w_{s(C)} = (19.1\text{ psf})(13.5\text{ ft})$$
$$= 258\text{ plf}$$

Determine the wind load on each frame at each level. Conservatively apply the end zone pressures on both ends of the building simultaneously,

where
l = length of structure, ft
b = width of structure, ft
h = height of wall at building element, ft

For wind from a north or south direction:

Total load to each frame:

$$P_{W(N\text{-}S)} = w_{s(A)}(2a) + w_{s(C)}\left(\frac{l}{2} - 2a\right)$$

Shear in diaphragm:

$$v_{(N\text{-}S)} = \frac{P_{W(N\text{-}S)}}{120\text{ ft}},\ \text{for roof}$$

$$v_{(N\text{-}S)} = \frac{P_{W(N\text{-}S)}}{90\text{ ft}},\ \text{for floors (deduction for stair openings)}$$

For wind from an east or west direction:

Total load to each frame:

$$P_{W(E\text{-}W)} = w_{s(A)}(2a) + w_{s(C)}\left(\frac{b}{2} - 2a\right)$$

Shear in diaphragm:

$$v_{(E\text{-}W)} = \frac{P_{W(E\text{-}W)}}{210\text{ ft}}, \text{ for roof and floors}$$

Table III-5 summarizes the total wind load in each direction acting on a steel frame at each level. The wind load at the ground level has not been included in the chart because it does not affect the steel frame.

The roof level dimensions exclude the screen wall area. The floor level dimensions correspond to the outside dimensions of the cladding.

Table III-5
Summary of Wind Loads at Each Level

	l, ft	b, ft	$2a$, ft	h, ft	$p_{s(A)}$, psf	$p_{s(C)}$, psf	$w_{s(A)}$, plf	$w_{s(C)}$, plf	$P_{W(N\text{-}S)}$, kips	$P_{W(E\text{-}W)}$, kips	$v_{(N\text{-}S)}$, plf	$v_{(E\text{-}W)}$, plf
Screen	93.0	33.0	0	13.3	0	19.1	0	254	11.8	4.19	–	–
Roof	120	90.0	24.6	9.25	28.9	19.1	267	177	12.8	10.2	205	68.5
4th	213	123	24.6	14.0	28.9	19.1	405	267	31.8	19.8	353	94.3
3rd	213	123	24.6	13.5	28.9	19.1	390	258	30.7	19.1	341	91.0
2nd	213	123	24.6	13.5	28.9	19.1	390	258	30.7	19.1	341	91.0
Base									118	72.4		

SEISMIC LOAD DETERMINATION

The floor plan area: 120 ft, column center line to column center line, by 210 ft, column centerline to column center line, with the edge of floor slab or roof deck 6 in. beyond the column center line.

$$\begin{aligned}\text{Area} &= (121\text{ ft})(211\text{ ft})\\ &= 25{,}500\text{ ft}^2\end{aligned}$$

The perimeter cladding system length:

$$\begin{aligned}\text{Length} &= (2)(123\text{ ft})+(2)(213\text{ ft})\\ &= 672\text{ ft}\end{aligned}$$

The perimeter cladding weight at floors:

Brick spandrel panel with metal stud backup:	$(7.50\text{ ft})(0.055\text{ kip/ft}^2)$	$= 0.413\text{ kip/ft}$
Window wall system:	$(6.00\text{ ft})(0.015\text{ kip/ft}^2)$	$= 0.090\text{ kip/ft}$
Total:		**0.503 kip/ft**

Typical roof dead load (from previous calculations):

Although 40 psf was used to account for the mechanical units and screen wall for the beam and column design, the entire mechanical area will not be uniformly loaded. Use 30% of the uniform 40 psf mechanical area load to determine the total weight of all of the mechanical equipment and screen wall for the seismic load determination.

Roof area:	$(25{,}500\text{ ft}^2)(0.020\text{ kip/ft}^2)$	$= 510\text{ kips}$
Wall perimeter:	$(672\text{ ft})(0.413\text{ kip/ft})$	$= 278\text{ kips}$
Mechanical area:	$(2{,}700\text{ ft}^2)(0.3)(0.040\text{ kip/ft}^2)$	$= 32.4\text{ kips}$
Total:		**820 kips**

Typical third and fourth floor dead load:

Note: An additional 10 psf has been added to the floor dead load to account for partitions per ASCE/SEI 7, Section 12.7.2.

Floor area:	$(25{,}500\text{ ft}^2)(0.085\text{ kip/ft}^2)$	$= 2{,}170\text{ kips}$
Wall perimeter:	$(672\text{ ft})(0.503\text{ kip/ft})$	$= 338\text{ kips}$
Total:		**2,510 kips**

Second floor dead load (the floor area is reduced because of the open atrium):

Floor area:	$(24{,}700\text{ ft}^2)(0.085\text{ kip/ft}^2)$	$= 2{,}100\text{ kips}$
Wall perimeter:	$(672\text{ ft})(0.503\text{ kip/ft})$	$= 338\text{ kips}$
Total:		**2,440 kips**

Total dead load of the building:

Roof	820 kips
Fourth floor	2,510 kips
Third floor	2,510 kips
Second floor	2,440 kips
Total	**8,280 kips**

Calculate the seismic forces.

Determine the seismic risk category and importance factors.

Office Building:	Risk Category II, from ASCE/SEI 7, Table 1.5-1
Seismic Importance Factor:	I_e = 1.00, from ASCE/SEI 7, Table 1.5-2

The site coefficients are given in this example. S_S and S_1 can also be determined from ASCE/SEI 7, Figures 22-1 and 22-2, respectively.

$S_S = 0.121g$
$S_1 = 0.060g$

Soil, Site Class D (given)

F_a @ $S_S \leq 0.25 = 1.6$ from ASCE/SEI 7, Table 11.4-1
F_v @ $S_1 \leq 0.1 = 2.4$ from ASCE/SEI 7, Table 11.4-2

Determine the maximum considered earthquake accelerations.

From ASCE/SEI 7, Equation 11.4-1:

$$\begin{aligned} S_{MS} &= F_a S_S \\ &= 1.6\left(0.121g\right) \\ &= 0.194g \end{aligned}$$

From ASCE/SEI 7, Equation 11.4-2:

$$\begin{aligned} S_{M1} &= F_v S_1 \\ &= 2.4\left(0.060g\right) \\ &= 0.144g \end{aligned}$$

Determine the design earthquake accelerations.

From ASCE/SEI 7, Equation 11.4-3:

$$\begin{aligned} S_{DS} &= \tfrac{2}{3} S_{MS} \\ &= \tfrac{2}{3}\left(0.194g\right) \\ &= 0.129g \end{aligned}$$

From ASCE/SEI 7, Equation 11.4-4:

$$
\begin{aligned}
S_{D1} &= \tfrac{2}{3} S_{M1} \\
&= \tfrac{2}{3}(0.144g) \\
&= 0.096g
\end{aligned}
$$

Determine the seismic design category from ASCE/SEI 7, Table 11.6-1.

With $S_{DS} < 0.167g$ and Risk Category II, Seismic Design Category A applies.

With $0.067g \le S_{D1} < 0.133g$ and Risk Category II, Seismic Design Category B applies.

Select the seismic force-resisting system from ASCE/SEI 7, Table 12.2-1. For Seismic Design Category B it is permissible to select a structural steel system not specifically detailed for seismic resistance (Item H). The response modification coefficient, R, is 3.

Determine the approximate fundamental period, T_a.

Building height, h_n = 55.0 ft

C_t = 0.02 and x = 0.75 from ASCE/SEI 7, Table 12.8-2 (“All other structural systems”)

From ASCE/SEI 7, Equation 12.8-7:

$$
\begin{aligned}
T_a &= C_t h_n^x \\
&= (0.02)(55.0\text{ ft})^{0.75} \\
&= 0.404\text{ s}
\end{aligned}
$$

(ASCE/SEI 7, Eq. 12.8-7)

Determine the redundancy factor from ASCE/SEI 7, Section 12.3.4.1.

ρ = 1.0, for Seismic Design Category B

From ASCE/SEI 7, Equation 12.4-4a, determine the vertical seismic effect term:

$$
\begin{aligned}
E_v &= 0.2 S_{DS} D \\
&= 0.2(0.129g)D \\
&= 0.0258D
\end{aligned}
$$

(ASCE/SEI 7, Eq. 12.4-4a)

From ASCE/SEI 7, Equation 12.4-3, determine the horizontal seismic effect term:

$$
\begin{aligned}
E_h &= \rho Q_E \\
&= 1.0(Q_E)
\end{aligned}
$$

(ASCE/SEI 7, Eq. 12.4-3)

The following seismic load combinations are as specified in ASCE/SEI 7, Sections 2.3.6 and 2.4.5 as directed by Section 12.4.2. Where the prescribed seismic load effect, $E = f(E_v, E_h)$, is combined with the effects of other loads, the following load combinations apply. Note that $L = 0.5L$ for LRFD per ASCE/SEI 7, Section 2.3.6 Exception 1.

LRFD	ASD
$1.2D + E_v + E_h + L + 0.2S$ $= 1.2D + 0.2S_{DS}D + \rho Q_E + 0.5L + 0.2S$ $= (1.2 + 0.0258)D + 1.0Q_E + 0.5L + 0.2S$ $= 1.23D + 1.0Q_E + 0.5L + 0.2S$	$1.0D + 0.7E_v + 0.7E_h$ $= 1.0D + 0.7(0.2S_{DS}D) + 0.7\rho Q_E$ $= [1.0 + 0.7(0.0258)]D + 0.7(1.0)Q_E$ $= 1.02D + 0.7Q_E$

LRFD	ASD
$0.9D - E_v + E_h$ $= 0.9D - 0.2S_{DS}D + \rho Q_E$ $= (0.9 - 0.0258)D + 1.0Q_E$ $= 0.874D + 1.0Q_E$	$1.0D + 0.525E_v + 0.525E_h + 0.75L + 0.75S$ $= 1.0D + 0.525(0.2S_{DS}D) + 0.525\rho Q_E + 0.75L + 0.75S$ $= [1.0 + 0.525(0.0258)]D + 0.525(1.0)Q_E + 0.75L$ $+ 0.75S$ $= 1.01D + 0.525Q_E + 0.75L + 0.75S$ $0.6D - 0.7E_v + 0.7E_h$ $= 0.6D - 0.7(0.2S_{DS}D) + 0.7\rho Q_E$ $= [0.6 - 0.7(0.0258)]D + 0.7(1.0)Q_E$ $= 0.582D + 0.7Q_E$

Where the prescribed seismic load effect with overstrength, $E = f(E_v, E_{mh})$, is combined with the effects of other loads, the following load combinations apply.

The overstrength factor, Ω_o, is determined from ASCE/SEI 7, Table 12.2-1. $\Omega_o = 3$ for steel systems not specifically detailed for seismic resistance, excluding cantilever column systems.

Determine the horizontal seismic effect term including overstrength.

$$E_{mh} = \Omega_o Q_E \le E_{cl} \qquad \text{(from ASCE/SEI 7, Eq. 12.4-7)}$$

$$= 3(Q_E)$$

where Q_E is the effect from seismic forces from seismic base shear, V, as calculated per ASCE/SEI 7, Section 12.8.1; diaphragm design forces, F_{px}, as calculated per ASCE/SEI 7, Section 12.10; or seismic design force, F_p, as calculated per Section 13.3.1. The capacity-limited horizontal seismic load effect, E_{cl}, is defined in ASCE/SEI 7, Section 11.3.

LRFD	ASD
$1.2D + E_v + E_{mh} + L + 0.2S$ $= 1.2D + 0.2S_{DS}D + \Omega_o Q_E + 0.5L + 0.2S$ $= (1.2 + 0.0258)D + 3Q_E + 0.5L + 0.2S$ $= 1.23D + 3.0Q_E + 0.5L + 0.2S$ $0.9D - E_v + E_{mh}$ $= 0.9D - 0.2S_{DS}D + \Omega_o Q_E$ $= (0.9 - 0.0258)D + 3Q_E$ $= 0.874D + 3.0Q_E$	$1.0D + 0.7E_v + 0.7E_{mh}$ $= 1.0D + 0.7(0.2S_{DS}D) + 0.7\Omega_o Q_E$ $= [1.0 + 0.7(0.0258)]D + 0.7(3)Q_E$ $= 1.02D + 2.1Q_E$ $1.0D + 0.525E_v + 0.525E_{mh} + 0.75L + 0.75S$ $= 1.0D + 0.525(0.2S_{DS}D) + 0.525\Omega_o Q_E + 0.75L + 0.75S$ $= [1.0 + 0.525(0.0258)]D + 0.525(3)Q_E + 0.75L$ $+ 0.75S$ $= 1.01D + 1.58Q_E + 0.75L + 0.75S$ $0.6D - 0.7E_v + 0.7E_{mh}$ $= 0.6D - 0.7(0.2S_{DS}D) + 0.7\Omega_o Q_E$ $= [0.6 - 0.7(0.0258)]D + 0.7(3)Q_E$ $= 0.582D + 2.1Q_E$

Calculate the seismic base shear using ASCE/SEI 7, Section 12.8.1.

Determine the seismic response coefficient, C_s, from ASCE/SEI 7, Equation 12.8-2:

$$\begin{aligned} C_s &= \frac{S_{DS}}{\left(\dfrac{R}{I_e}\right)} \\ &= \frac{0.129}{\left(\dfrac{3}{1.00}\right)} \\ &= 0.0430 \end{aligned}$$

Let T_a = T, as is permitted in Section 12.8.2. From ASCE/SEI 7, Figure 22-14, T_L = 12 > T (midwestern city); therefore, use ASCE/SEI 7, Section 12.8.1.1, to determine the upper limit of C_s.

$$\begin{aligned} C_s &= \frac{S_{D1}}{T\left(\dfrac{R}{I_e}\right)} \\ &= \frac{0.096}{0.404\left(\dfrac{3}{1.00}\right)} \\ &= 0.0792 \end{aligned}$$ (ASCE/SEI 7, Eq. 12.8-3)

C_s shall not be taken less than:

$$\begin{aligned} C_s &= 0.044 S_{DS} I_e \geq 0.01 \\ &= 0.044(0.129)(1.00) < 0.01 \\ &= 0.00568 < 0.01 \end{aligned}$$ (ASCE/SEI 7, Eq. 12.8-5)

Therefore, C_s = 0.0430.

Calculate the seismic base shear from ASCE/SEI 7, Section 12.8.1:

$$\begin{aligned} V &= C_s W \\ &= 0.0430(8,280 \text{ kips}) \\ &= 356 \text{ kips} \end{aligned}$$ (ASCE/SEI 7, Eq. 12.8-1)

Determine vertical distribution of seismic forces from ASCE/SEI 7, Section 12.8.3.

$$\begin{aligned} F_x &= C_{vx} V \\ &= C_{vx}(356 \text{ kips}) \end{aligned}$$ (ASCE/SEI 7, Eq. 12.8-11)

$$C_{vx} = \frac{w_x {h_x}^k}{\sum_{i=1}^{n} w_i {h_i}^k}$$ (ASCE/SEI 7, Eq. 12.8-12)

for structures having a period of 0.5 s or less, k = 1.

Determine horizontal shear distribution at each level per ASCE/SEI 7, Section 12.8.4.

$$V_x = \sum_{i=x}^{n} F_i \qquad \text{(ASCE/SEI, Eq. 12.8-13)}$$

Determine the overturning moment at each level per ASCE/SEI 7, Section 12.8.5.

$$M_x = \sum_{i=x}^{n} F_i (h_i - h_x)$$

The seismic forces at each level are summarized in Table III-6.

Table III-6
Summary of Seismic Forces at Each Level

	w_x, kips	**h_x^k, ft**	**$w_x h_x^k$, kip-ft**	**C_{vx}**	**F_x, kips**	**V_x, kips**	**M_x, kip-ft**
Roof	820	55.0	45,100	0.182	64.8	64.8	
4th	2,510	40.5	102,000	0.411	146	211	940
3rd	2,510	27.0	67,800	0.273	97.2	308	3,790
2nd	2,440	13.5	32,900	0.133	47.3	355	7,940
Base	**8,280**		**248,000**		**355**		**12,700**

Calculate strength and determine rigidity of diaphragms.

Determine the diaphragm design forces from ASCE/SEI 7, Section 12.10.1.1.

F_{px} is the largest of:

1. The force F_x at each level determined by the vertical distribution above

2. $F_{px} = \dfrac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \le 0.4 S_{DS} I_e w_{px}$, from ASCE/SEI 7, Equations 12.10-1 and 12.10-3

$$\begin{aligned} &\le 0.4(0.129)(1.00)\,w_{px} \\ &\le 0.0516 w_{px} \end{aligned}$$

3. $F_{px} = 0.2 S_{DS} I_e w_{px}$, from ASCE/SEI 7, Equation 12.10-2

$$\begin{aligned} &= 0.2(0.129)(1.00)\,w_{px} \\ &= 0.0258 w_{px} \end{aligned}$$

The diaphragm shear forces include the effects of openings in the diaphragm (such as stair shafts) and an accidental torsion calculated using an eccentricity of 5% of the building dimension per ASCE/SEI 7, Section 12.8.4. The accidental torsion resulted in a 10% increase in the shear force.

A summary of the diaphragm forces is given in Table III-7,

where

F_{px} = max(A, B, C)

A = force at a level based on the vertical distribution of seismic forces

$$B = F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \le 0.4 S_{DS} I_e w_{px}$$

C $= 0.2 S_{DS} I_e w_{px}$

L = the length of the frame connected to the diaphragm (in the N-S or E-W direction)

V = shear force in the diaphragm

Table III-7
Summary of Diaphragm Forces

	w_{px}, kips	A, kips	B, kips	C, kips	F_{px}, kips	$L_{(N\text{-}S)}$, ft	$L_{(E\text{-}W)}$, ft	$v_{(N\text{-}S)}$, plf	$v_{(E\text{-}W)}$, plf
Roof	820	64.8	42.3	21.2	64.8	240	420	297	170
4th	2,510	146	130	64.8	146	180	420	892	382
3rd	2,510	97.2	130	64.8	130	180	420	794	340
2nd	2,440	47.3	105	63.0	105	180	420	642	275

Roof

Roof deck:	1½-in.-deep, 22 gage, wide rib
Support fasteners:	⅝-in. puddle welds in 36/5 pattern
Sidelap fasteners:	(3) #10 TEK screws
Joist spacing:	s = 6.00 ft
Diaphragm length:	210 ft
Diaphragm width:	l_v =120 ft

By inspection, the critical condition for the diaphragm is loading from the north or south directions.

LRFD	ASD
From the ASCE/SEI 7 load combinations for strength design, the earthquake load is: $v_r = E_h$ $= \rho Q_E$ $= 1.0(0.297\text{ klf})$ $= 0.297\text{ klf}$ The wind load is: $v_r = 1.0W$ $= 1.0(0.205\text{ klf})$ $= 0.205\text{ klf}$	From the ASCE/SEI 7 load combinations for allowable stress design, the earthquake load is: $v_r = 0.7E_h$ $= 0.7\rho Q_E$ $= 0.7(1.0)(0.297\text{ klf})$ $= 0.208\text{ klf}$ The wind load is: $v_r = 0.6W$ $= 0.6(0.205\text{ klf})$ $= 0.123\text{ klf}$

From the SDI *Diaphragm Design Manual* (SDI, 2015), the available shear strengths are determined as follows:

For panel buckling strength: v_n = 3.88 klf
For connection strength: v_n = 0.815 klf

LRFD	ASD
Panel buckling strength: $\phi v_n = 0.80(3.88\text{ klf})$ $= 3.10\text{ klf} > 0.297\text{ ksf}$ **o.k.**	Panel buckling strength: $\dfrac{v_n}{\Omega} = \dfrac{3.88\text{ klf}}{2.00}$ $= 1.94\text{ klf} > 0.208\text{ klf}$ **o.k.**
Connection strength: Earthquake $\phi v_n = 0.55(0.815\text{ klf})$ $= 0.448\text{ klf} > 0.297\text{ ksf}$ **o.k.**	Connection strength: Earthquake $\dfrac{v_n}{\Omega} = \dfrac{0.815\text{ klf}}{3.00}$ $= 0.272\text{ klf} > 0.208\text{ ksf}$ **o.k.**
Wind $\phi v_n = 0.70(0.815\text{ klf})$ $= 0.571\text{ klf} > 0.205\text{ ksf}$ **o.k.**	Wind $\dfrac{v_n}{\Omega} = \dfrac{0.815\text{ klf}}{2.35}$ $= 0.347\text{ klf} > 0.123\text{ ksf}$ **o.k.**

Check diaphragm flexibility.

From the SDI *Diaphragm Design Manual* (SDI, 2015):

$$D_{xx} = 607\text{ ft}$$
$$K_1 = 0.286\text{ ft}^{-1}$$
$$K_2 = 870\text{ kip/in.}$$
$$K_4 = 3.55$$

From SDI *Diaphragm Design Manual*, Section 9:

$$\begin{aligned} G' &= \frac{K_2}{K_4 + \dfrac{0.3D_{xx}}{s} + 3K_1 s} \\ &= \frac{870\text{ kip/in.}}{3.55 + \dfrac{0.3(607\text{ ft})}{6.00\text{ ft}} + 3\left(\dfrac{0.286}{\text{ft}}\right)(6.00\text{ ft})} \\ &= 22.3\text{ kip/in.} \end{aligned}$$

Seismic loading on diaphragm.

$$\begin{aligned} w &= \frac{64.8\text{ kips}}{210\text{ ft}} \\ &= 0.309\text{ klf} \end{aligned}$$

Calculate the maximum diaphragm deflection.

$$
\begin{aligned}
\Delta &= \frac{wL^2}{8l_v G'} \\
&= \frac{(0.309\text{ klf})(210\text{ ft})^2}{8(120\text{ ft})(22.3\text{ kip/in.})} \\
&= 0.637\text{ in.}
\end{aligned}
$$

Story drift = 0.154 in. (from computer output)

The diaphragm deflection exceeds two times the story drift; therefore, the diaphragm may be considered to be flexible in accordance with ASCE/SEI 7, Section 12.3.1.3.

The roof diaphragm is flexible in the N-S direction, but using a rigid diaphragm distribution is more conservative for the analysis of this building. By similar reasoning, the roof diaphragm will also be treated as a rigid diaphragm in the E-W direction.

Third and fourth floors

Floor deck:	3-in.-deep, 22 gage, composite deck with normal weight concrete
Support fasteners:	⅝-in. puddle welds in a 36/4 pattern
Sidelap fasteners:	(3) #10 TEK screws
Beam spacing:	s = 10 ft
Diaphragm length:	210 ft
Diaphragm width:	120 ft

l_v = 120 ft – 30 ft = 90 ft, to account for the stairwell

By inspection, the critical condition for the diaphragm is loading from the north or south directions.

LRFD	ASD
From the ASCE/SEI 7 load combinations for strength design, the earthquake load for the fourth floor is: $v_r = E_h$ $= \rho Q_E$ $= 1.0(0.892\text{ klf})$ $= 0.892\text{ klf}$ For the fourth floor, the wind load is: $v_r = 1.0W$ $= 1.0(0.353\text{ klf})$ $= 0.353\text{ klf}$	From the ASCE/SEI 7 load combinations for strength design, the earthquake load for the fourth floor is: $v_r = E_h$ $= 0.7\rho Q_E$ $= 0.7(1.0)(0.892\text{ klf})$ $= 0.624\text{ klf}$ For the fourth floor, the wind load is: $v_r = 0.6W$ $= 0.6(0.353\text{ klf})$ $= 0.212\text{ klf}$

LRFD	ASD
From the ASCE/SEI 7 load combinations for strength design, the earthquake load for the third floor is: $v_r = E_h$ $= \rho Q_E$ $= 1.0(0.794 \text{ klf})$ $= 0.794 \text{ klf}$ For the third floor, the wind load is: $v_r = 1.0W$ $= 1.0(0.341 \text{ klf})$ $= 0.341 \text{ klf}$	From the ASCE/SEI 7 load combinations for strength design, the earthquake load for the third floor is: $v_r = E_h$ $= 0.7\rho Q_E$ $= 0.7(1.0)(0.794 \text{ klf})$ $= 0.556 \text{ klf}$ For the third floor, the wind load is: $v_r = 0.6W$ $= 0.6(0.341 \text{ klf})$ $= 0.205 \text{ klf}$

From the SDI *Diaphragm Design Manual* (SDI, 2015), the nominal connection shear strength is v_n = 5.38 klf.

Calculate the available strengths.

LRFD	ASD
Connection Strength (same for earthquake or wind) (SDI, 2015) $\phi v_n = 0.5(5.38 \text{ klf})$ $= 2.69 \text{ klf} > 0.892 \text{ klf}$ **o.k.**	Connection Strength (same for earthquake or wind) (SDI, 2015) $\dfrac{v_n}{\Omega} = \dfrac{5.38 \text{ klf}}{3.25}$ $= 1.66 \text{ klf} > 0.624 \text{ klf}$ **o.k.**

Check diaphragm flexibility.

From the SDI *Diaphragm Design Manual* (SDI, 2015):

$$K_1 = 0.318 \text{ ft}^{-1}$$
$$K_2 = 870 \text{ kip/in.}$$
$$K_3 = 2{,}380 \text{ kip/in.}$$
$$K_4 = 3.54$$

$$\begin{aligned} G' &= \left(\frac{K_2}{K_4 + 3K_1 s}\right) + K_3 \\ &= \left[\frac{870 \text{ kip/in.}}{3.54 + 3\left(\dfrac{0.318}{\text{ft}}\right)(10 \text{ ft})}\right] + 2{,}380 \text{ kip/in.} \\ &= 2{,}450 \text{ kip/in.} \end{aligned}$$

Fourth floor

Calculate seismic loading on the diaphragm based on the fourth floor seismic load.

$$w = \frac{146\text{ kips}}{210\text{ ft}}$$
$$= 0.695\text{ klf}$$

Calculate the maximum diaphragm deflection on the fourth floor.

$$\Delta = \frac{wL^2}{8l_vG'}$$
$$= \frac{(0.695\text{ klf})(210\text{ ft})^2}{8(90\text{ ft})(2,450\text{ kip/in.})}$$
$$= 0.0174\text{ in.}$$

Third floor

Calculate seismic loading on the diaphragm based on the third floor seismic load.

$$w = \frac{130\text{ kips}}{210\text{ ft}}$$
$$= 0.619\text{ klf}$$

Calculate the maximum diaphragm deflection on the third floor.

$$\Delta = \frac{wL^2}{8l_vG'}$$
$$= \frac{(0.619\text{ klf})(210\text{ ft})^2}{8(90\text{ ft})(2,450\text{ kip/in.})}$$
$$= 0.0155\text{ in.}$$

The diaphragm deflection at the third and fourth floors is less than two times the story drift (story drift = 0.268 in. from computer output); therefore, the diaphragm is considered rigid in accordance with ASCE/SEI 7, Section 12.3.1.3. By inspection, the floor diaphragm will also be rigid in the E-W direction.

Second floor

Floor deck:	3-in.-deep, 22 gage, composite deck with normal weight concrete
Support fasteners:	⅝-in. puddle welds in a 36/4 pattern
Sidelap fasteners:	(3) #10 TEK screws
Beam spacing:	s = 10 ft
Diaphragm length:	210 ft
Diaphragm width:	120 ft

Because of the atrium opening in the floor diaphragm, an effective diaphragm depth of 75 ft will be used for the deflection calculations.

By inspection, the critical condition for the diaphragm is loading from the north or south directions.

LRFD	ASD
From the ASCE/SEI 7 load combinations for strength design, the earthquake load is: $v_r = E_h$ $= \rho Q_E$ $= 1.0(0.642\text{ klf})$ $= 0.642\text{ klf}$ The wind load is: $v_r = 1.0W$ $= 1.0(0.341\text{ klf})$ $= 0.341\text{ klf}$	From the ASCE/SEI 7 load combinations for strength design, the earthquake load is: $v_r = E_h$ $= 0.7\rho Q_E$ $= 0.7(1.0)(0.642\text{ klf})$ $= 0.449\text{ klf}$ The wind load is: $v_r = 0.6W$ $= 0.6(0.341\text{ klf})$ $= 0.205\text{ klf}$

From the SDI *Diaphragm Design Manual* (SDI, 2015), the nominal connection shear strength is: v_n = 5.38 klf.

Calculate the available strengths.

LRFD	ASD
Connection Strength (same for earthquake or wind) (SDI, 2015) $\phi v_n = 0.50(5.38\text{ klf})$ $= 2.69\text{ klf} > 0.642\text{ klf}$ **o.k.**	Connection Strength (same for earthquake or wind) (SDI, 2015) $\dfrac{v_n}{\Omega} = \dfrac{5.38\text{ klf}}{3.25}$ $= 1.66\text{ klf} > 0.449\text{ klf}$ **o.k.**

Check diaphragm flexibility.

From the SDI *Diaphragm Design Manual* (SDI, 2015):

$$K_1 = 0.318\text{ ft}^{-1}$$
$$K_2 = 870\text{ kip/in.}$$
$$K_3 = 2{,}380\text{ kip/in.}$$
$$K_4 = 3.54$$

$$\begin{aligned} G' &= \left(\frac{K_2}{K_4 + 3K_1 s}\right) + K_3 \\ &= \left[\frac{870\text{ kip/in.}}{3.54 + 3\left(\dfrac{0.318}{\text{ft}}\right)(10\text{ ft})}\right] + 2{,}380\text{ kip/in.} \\ &= 2{,}450\text{ kip/in.} \end{aligned}$$

Calculate seismic loading on the diaphragm.

$$\begin{aligned} w &= \frac{105\text{ kips}}{210\text{ ft}} \\ &= 0.500\text{ klf} \end{aligned}$$

Calculate the maximum diaphragm deflection.

$$\Delta = \frac{wL^2}{8bG'}$$

$$= \frac{(0.500\text{ klf})(210\text{ ft})^2}{8(75\text{ ft})(2,450\text{ kip/in.})}$$

$$= 0.0150\text{ in.}$$

Story drift = 0.210 in. (from computer output)

The diaphragm deflection is less than two times the story drift; therefore, the diaphragm is considered rigid in accordance with ASCE/SEI 7, Section 12.3.1.3. By inspection, the floor diaphragm will also be rigid in the E-W direction.

Horizontal Shear Distribution and Torsion

The seismic forces to be applied in the frame analysis in each direction, including the effect of accidental torsion, in accordance with ASCE/SEI 7, Section 12.8.4, are shown in Tables III-8 and III-9.

Table III-8
Horizontal Shear Distribution including Accidental Torsion—Grids 1 and 8

	F_y	Load on Frame		Load to Grids 1 and 8 Accidental Torsion		Total
	kips	**%**	**kips**	**%**	**kips**	**kips**
Roof	64.8	50	32.4	5	3.24	35.6
4th	146	50	73.0	5	7.30	80.3
3rd	97.2	50	48.6	5	4.86	53.5
2nd	47.3	50	23.7	5	2.37	26.1
Base						196

Table III-9
Horizontal Shear Distribution including Accidental Torsion—Grids A and F

	F_y	Load on Frame		Load to Grids A and F Accidental Torsion		Total
	kips	**%**	**kips**	**%**	**kips**	**kips**
Roof	64.8	50	32.4	5	3.24	35.6
4th	146	50	73.0	5	7.30	80.3
3rd	97.2	50	48.6	5	4.86	53.5
2nd	47.3	50.8[1]	24.0	5	2.37	26.4
Base						196

[1] Note: In this example, Grids A and F have both been conservatively designed for the slightly higher load on Grid A due to the atrium opening. The increase in load is calculated Table III-10.

Table III-10

	Area, ft^2	Mass, kips	y-dist, ft	M_y, kip-ft
I	25,500	2,170	60.5	131,000
II	841	71.5	90.5	6,470
Base	24,700	2,100		125,000

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$$y = \frac{125,000 \text{ kip-ft}}{2,100 \text{ kips}}$$
$$= 59.5 \text{ ft}$$

$$(100\%)\frac{(121 \text{ ft} - 59.5 \text{ ft})}{121 \text{ ft}} = 50.8\%$$

MOMENT FRAME MODEL

Grids 1 and 8 were modeled in conventional structural analysis software as two-dimensional models. The second-order option in the structural analysis program was not used. Rather, for illustration purposes, second-order effects are calculated separately, using the "Approximate Second-Order Analysis" method described in AISC *Specification* Appendix 8.

The column and beam layouts for the moment frames follow. Although the frames on Grids A and F are the same, slightly heavier seismic loads accumulate on Grid F after accounting for the atrium area and accidental torsion. The models are half-building models. The frame was originally modeled with W14×82 interior columns and W21×44 non-composite beams, but was revised because the beams and columns did not meet the strength requirements. The W14×82 column size was increased to a W14×90 and the W21×44 beams were upsized to W24×55 beams. Minimum composite studs are specified for the beams (corresponding to $\Sigma Q_n = 0.25F_yA_s$). Since the span does not exceed 30 ft, the ductility requirement is met per AISC *Specification* Commentary Section I3.2d.1. The beams were modeled with a stiffness of $I_{eq} = I_s$.

The frame was checked for both wind and seismic story drift limits. Based on the results on the computer analysis, the frame meets the L/400 drift criterion for a 10-year wind (0.7W) indicated in ASCE/SEI 7, Commentary Section CC.2.2. In addition, the frame meets the $0.025h_{sx}$ allowable story drift limit given in ASCE/SEI 7, Table 12.12-1, for Risk Category II.

All of the vertical loads on the frame were modeled as point loads on the frame. The dead load and live load are shown in the load cases that follow. The wind, seismic and notional loads from leaning columns are modeled and distributed 1/14 to exterior columns and 1/7 to the interior columns. This approach minimizes the tendency to accumulate too much load in the lateral system nearest an externally applied load.

Also shown in the following models are the remainder of the half-building model gravity loads from the interior leaning columns accumulated in a single leaning column which was connected to the frame portion of the model with pinned ended links. Because the second-order analyses that follow will use the "Approximate Second-Order Analysis" (amplified first-order) approach given in the AISC *Specification* Appendix 8, the inclusion of the leaning column is unnecessary, but serves to summarize the leaning column loads and illustrate how these might be handled in a full second-order analysis. See "A Practical Approach to the 'Leaning' Column" (Geschwindner, 1994).

There are five lateral load cases. Two are the wind load and seismic load, per the previous discussion. In addition, notional loads of $N_i = 0.002Y_i$ were established. The model layout, nominal dead, live, and snow loads with associated notional loads, wind loads and seismic loads are shown in Figures III-15 through III-23.

The same modeling procedures were used in the braced frame analysis. If column bases are not fixed in construction, they should not be fixed in the analysis.

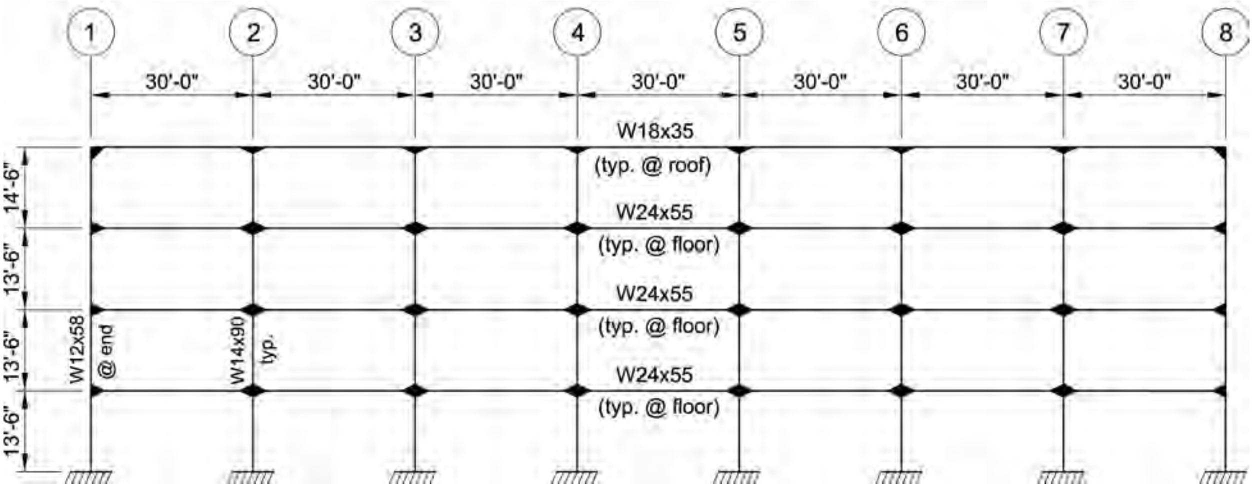

Fig. III-15. Frame layout—Grid A and F.

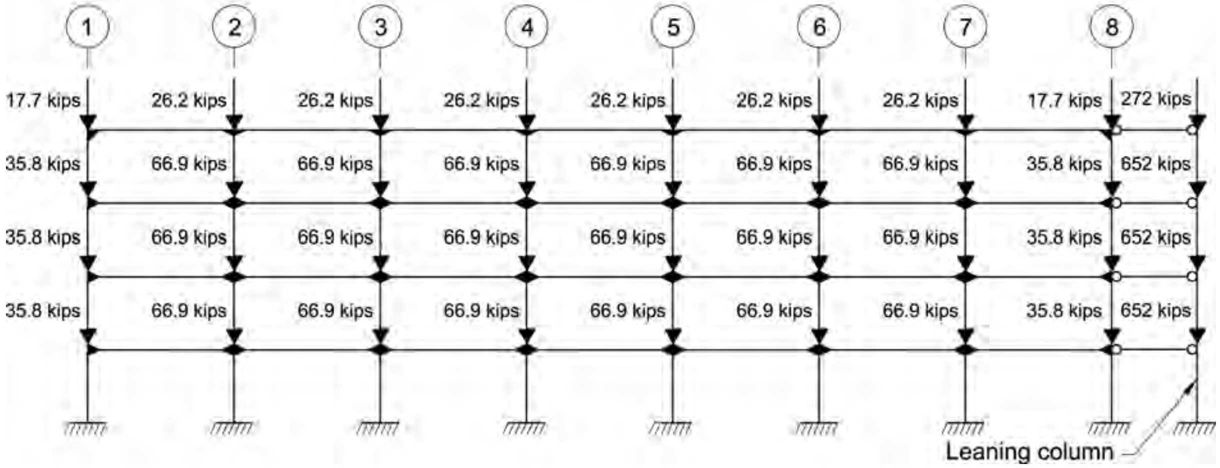

Fig. III-16. Nominal dead loads—Grid A and F.

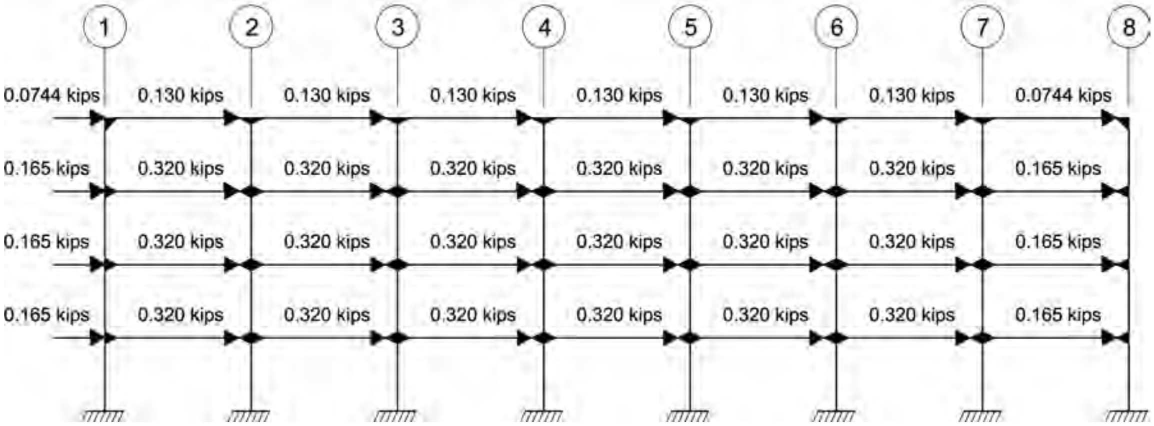

Fig. III-17. Notional dead loads—Grid A and F.

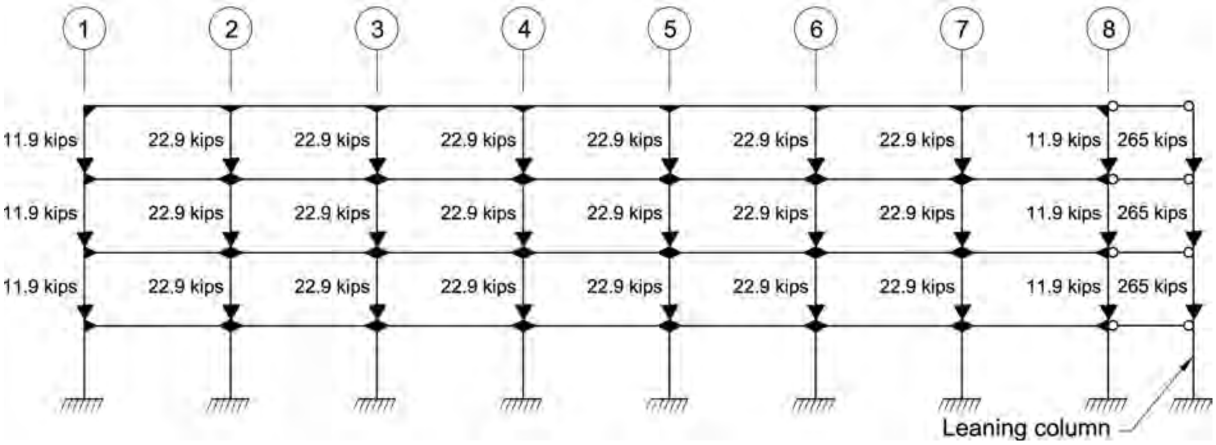

Fig. III-18. Nominal live loads—Grid A and F.

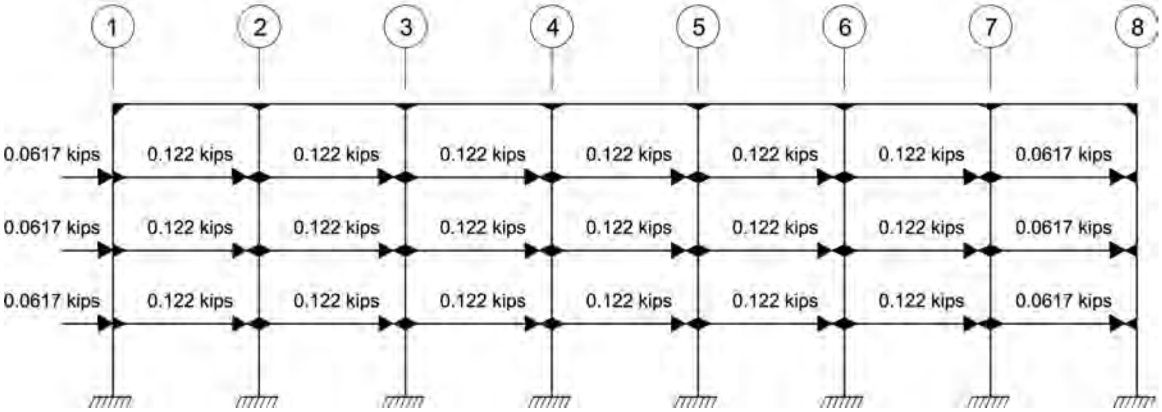

Fig. III-19. Notional live loads—Grid A and F.

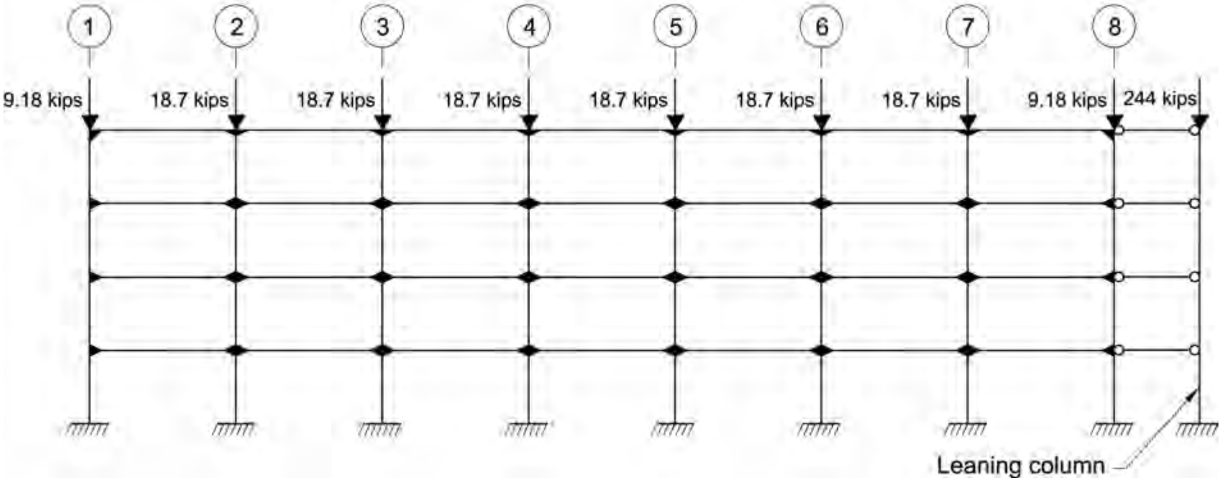

Fig. III-20. Nominal snow loads—Grid A and F.

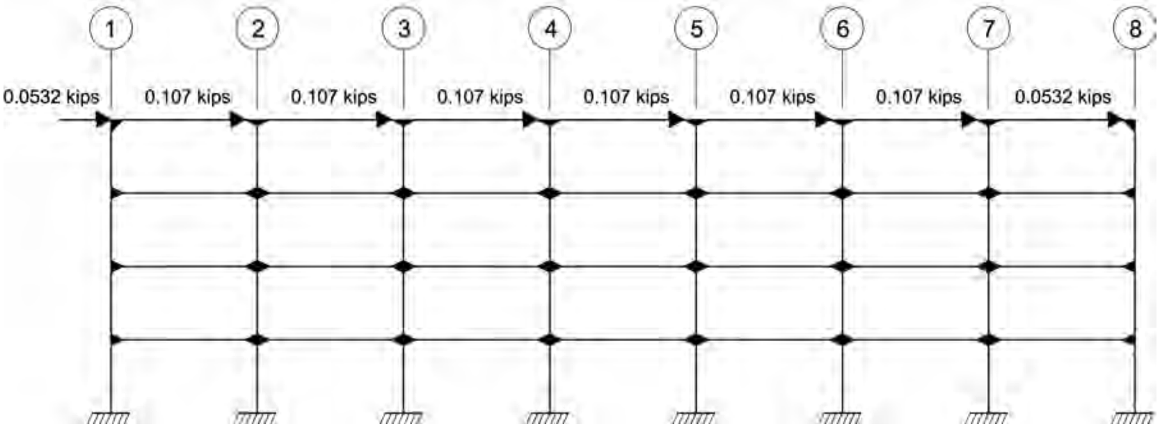

Fig. III-21. Notional snow loads—Grid A and F.

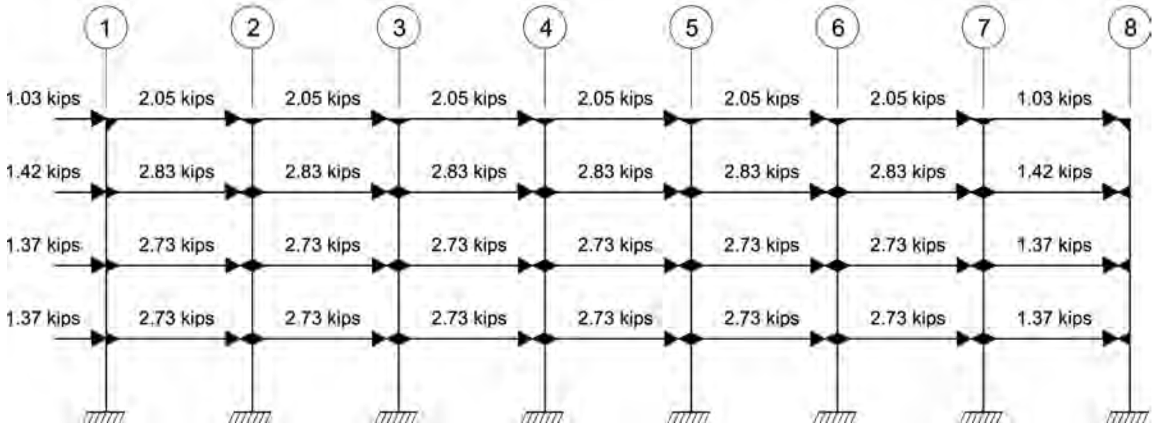

Fig. III-22. Nominal wind loads (1.0W)—Grid A and F.

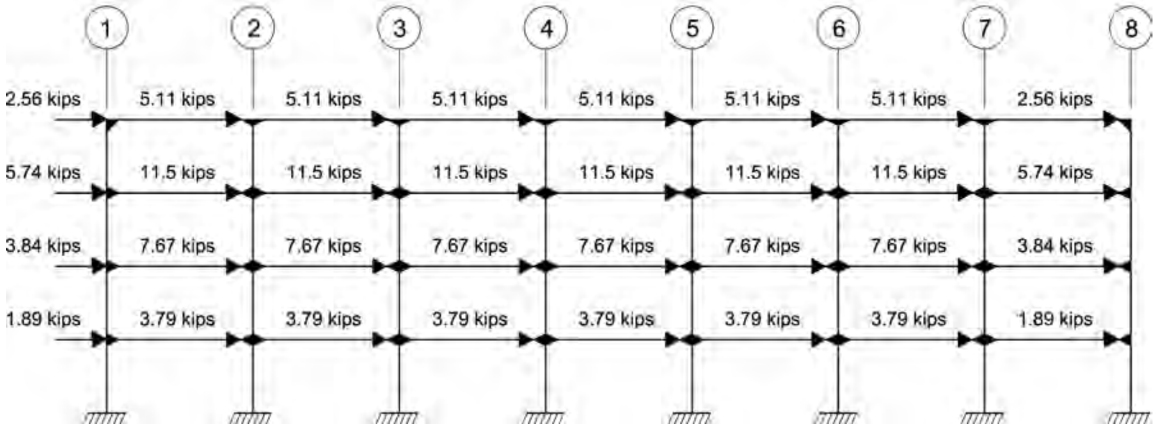

Fig. III-23. Seismic loads (1.0Q_E)—Grid A and F.

CALCULATION OF REQUIRED STRENGTH—THREE METHODS

Three methods for checking one of the typical interior column designs at the base of the building are presented below. All three of the methods presented require a second-order analysis (either direct via computer analysis techniques or by amplifying a first-order analysis). A fourth method called the “First-Order Analysis Method” is also an option. This method does not require a second-order analysis; however, this method is not presented below. For additional guidance on applying any of these methods, see the discussion in AISC *Manual* Part 2 titled Required Strength, Stability, Effective Length, and Second-Order Effects.

GENERAL INFORMATION FOR ALL THREE METHODS

Seismic load combinations controlled over wind load combinations in the direction of the moment frames in the example building. The frame analysis was run for all LRFD and ASD load combinations; however, only the controlling combinations have been illustrated in the following examples. A lateral load of 0.2% of gravity load was included for all gravity-only load combinations per AISC *Manual* Part 2.

The second-order analysis for all of the following examples were carried out by doing a first-order analysis and then amplifying the results to achieve a set of second-order design forces using the approximate second-order analysis procedure from AISC *Specification* Appendix 8.

METHOD 1—DIRECT ANALYSIS METHOD

Design for stability by the direct analysis method is found in AISC *Specification* Chapter C. This method requires that both the flexural and axial stiffness are reduced and that 0.2% notional lateral loads are applied in the analysis to account for geometric imperfections and inelasticity, per AISC *Specification* Section C2.2b(a). Any general second-order analysis method that considers both P-δ and P-Δ effects is permitted. The amplified first-order analysis method of AISC *Specification* Appendix 7 is also permitted provided that the B_1 and B_2 factors are based on the reduced flexural and axial stiffnesses. A summary of the axial loads, moments and first floor drifts from the first-order analysis is shown in the following. The floor diaphragm deflection in the east-west direction was previously determined to be very small and will thus be neglected in these calculations. Second-order member forces are determined using the approximate procedure of AISC *Specification* Appendix 8.

It was assumed, subject to verification, that B_2 is less than 1.7 for each load combination; therefore, per AISC *Specification* Section C2.2b(d), the notional loads were applied to the gravity-only load combinations. The required seismic load combinations, as given in ASCE/SEI 7, Section 12.4, were derived previously.

LRFD	ASD
$1.23D \pm 1.0Q_E + 0.5L + 0.2S$ (Controls columns and beams)	$1.01D + 0.525Q_E + 0.75L + 0.75S$ (Controls columns and beams)
From a first-order analysis with notional loads where appropriate and reduced stiffnesses:	From a first-order analysis with notional loads where appropriate and reduced stiffnesses:
For interior column design	For interior column design
$P_u = 317$ kips	$P_a = 295$ kips
$M_{u1} = 148$ kip-ft (from first-order analysis)	$M_{a1} = 77.9$ kip-ft
$M_{u2} = 233$ kip-ft (from first-order analysis)	$M_{a2} = 122$ kip-ft
First story drift with reduced stiffnesses = 0.718 in.	First story drift with reduced stiffnesses = 0.377 in.

Note: For ASD, ordinarily the second-order analysis must be carried out under 1.6 times the ASD load combinations and the results must be divided by 1.6 to obtain the required strengths. For this example, second-order analysis by the approximate B_1-B_2 analysis method is used. This method incorporates the 1.6 multiplier directly in the B_1 and B_2 amplifiers, such that no other modification is needed.

The required second-order flexural strength, M_r, and required axial strength, P_r, are determined as follows. For typical interior columns, the gravity-load moments are approximately balanced, therefore, $M_{nt} = 0$ kip-ft.

Calculate the amplified forces and moments in accordance with AISC *Specification* Appendix 8 at the ground floor. The required second-order flexural strength is determined as follows:

$$M_r = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(}\textit{Spec}\text{. Eq. A-8-1)}$$

Determine B_1

Per AISC *Specification* Appendix 8, Section 8.2.1, note that for members subject to axial compression, B_1 may be calculated based on the first-order estimate; therefore:

$$P_r = P_{nt} + P_{lt}$$

where
P_r = required second-order axial strength using LRFD or ASD load combinations

From AISC *Specification* Appendix 8, Section 8.2.1, the B_1 multiplier for the W14×90 column is determined as follows:

LRFD	ASD
$B_1 = \dfrac{C_m}{1-\dfrac{\alpha P_r}{P_{e1}}} \geq 1$ (*Spec.* Eq. A-8-3)	$B_1 = \dfrac{C_m}{1-\dfrac{\alpha P_r}{P_{e1}}} \geq 1$ (*Spec.* Eq. A-8-3)
where $P_r = 317$ kips (from first-order computer analysis) $I_x = 999\text{ in.}^4$ $\tau_b = 1.0$ (to be verified per *Spec.* Section C2.3(b)) $\alpha = 1.0$	where $P_r = 295$ kips (from first-order computer analysis) $I_x = 999\text{ in.}^4$ $\tau_b = 1.0$ (to be verified per *Spec.* Section C2.3(b)) $\alpha = 1.6$
As discussed in AISC *Specification* Appendix 8, Section 8.2.1, $EI^* = 0.8\tau_b EI$ when using the direct analysis method.	As discussed in AISC *Specification* Appendix 8, Section 8.2.1, $EI^* = 0.8\tau_b EI$ when using the direct analysis method.
$P_{e1} = \dfrac{\pi^2 EI^*}{(L_{c1})^2}$ (*Spec.* Eq. A-8-5) $= \dfrac{\pi^2(0.8)(1.0)(29{,}000\text{ ksi})(999\text{ in.}^4)}{\left[(1.0)(13.5\text{ ft})(12\text{ in./ft})\right]^2}$ $= 8{,}720$ kips	$P_{e1} = \dfrac{\pi^2 EI^*}{(L_{c1})^2}$ (*Spec.* Eq. A-8-5) $= \dfrac{\pi^2(0.8)(1.0)(29{,}000\text{ ksi})(999\text{ in.}^4)}{\left[(1.0)(13.5\text{ ft})(12\text{ in./ft})\right]^2}$ $= 8{,}720$ kips
$C_m = 0.6 - 0.4(M_1/M_2)$ (*Spec.* Eq. A-8-4) $= 0.6 - 0.4(148\text{ kip-ft}/233\text{ kip-ft})$ $= 0.346$	$C_m = 0.6 - 0.4(M_1/M_2)$ (*Spec.* Eq. A-8-4) $= 0.6 - 0.4(77.9\text{ kip-ft}/122\text{ kip-ft})$ $= 0.345$

LRFD	ASD
$B_1 = \dfrac{0.346}{1-\dfrac{1.0(317\text{ kips})}{8{,}720\text{ kips}}} < 1$ $= 0.359 < 1$ Therefore, use $B_1 = 1$	$B_1 = \dfrac{0.345}{1-\dfrac{1.6(295\text{ kips})}{8{,}720\text{ kips}}} < 1$ $= 0.365 < 1$ Therefore, use $B_1 = 1$

Determine B_2

LRFD	ASD
$P_{mf} = 2{,}250$ kips (gravity load in moment frame) $P_{story} = 5{,}440$ kips (from computer output) $\Delta_H = 0.718$ in. (from computer output) $\alpha = 1.0$ $R_M = 1-0.15\left(\dfrac{P_{mf}}{P_{story}}\right)$ (*Spec*. Eq. A-8-8) $= 1-0.15\left(\dfrac{2{,}250\text{ kips}}{5{,}440\text{ kips}}\right)$ $= 0.938$ From previous seismic force distribution calculations: $H = 1.0Q_E$ (Lateral) $= 1.0(196\text{ kips})$ $= 196\text{ kips}$ $P_{e\,story} = R_M \dfrac{HL}{\Delta_H}$ (*Spec*. Eq. A-8-7) $= (0.938)\dfrac{(196\text{ kips})(13.5\text{ ft})(12\text{ in./ft})}{0.718\text{ in.}}$ $= 41{,}500\text{ kips}$ $B_2 = \dfrac{1}{1-\dfrac{\alpha P_{story}}{P_{e\,story}}} \ge 1$ (*Spec*. Eq. A-8-6) $= \dfrac{1}{1-\dfrac{1.0(5{,}440\text{ kips})}{41{,}500\text{ kips}}} > 1$ $= 1.15 > 1$ Because $B_2 < 1.7$, it is verified that it was unnecessary to add the notional loads to the lateral loads for this load combination.	$P_{mf} = 2{,}090$ kips (gravity load in moment frame) $P_{story} = 5{,}120$ kips (from computer output) $\Delta_H = 0.377$ in. (from computer output) $\alpha = 1.6$ $R_M = 1-0.15\left(\dfrac{P_{mf}}{P_{story}}\right)$ (*Spec*. Eq. A-8-8) $= 1-0.15\left(\dfrac{2{,}090\text{ kips}}{5{,}120\text{ kips}}\right)$ $= 0.939$ From previous seismic force distribution calculations: $H = 0.525Q_E$ (Lateral) $= 0.525(196\text{ kips})$ $= 103\text{ kips}$ $P_{e\,story} = R_M \dfrac{HL}{\Delta_H}$ (*Spec*. Eq. A-8-7) $= (0.939)\dfrac{(103\text{ kips})(13.5\text{ ft})(12\text{ in./ft})}{0.377\text{ in.}}$ $= 41{,}600\text{ kips}$ $B_2 = \dfrac{1}{1-\dfrac{\alpha P_{story}}{P_{e\,story}}} \ge 1$ (*Spec*. Eq. A-8-6) $= \dfrac{1}{1-\dfrac{1.6(5{,}120\text{ kips})}{41{,}600\text{ kips}}} > 1$ $= 1.25 > 1$ Because $B_2 < 1.7$, it is verified that it was unnecessary to add the notional loads to the lateral loads for this load combination.

Calculate amplified moment and axial load

From AISC *Specification* Equation A-8-1, the required second-order flexural strength is determined as follows:

LRFD	ASD
$M_r = B_1 M_{nt} + B_2 M_{lt}$ $= (1.0)(0 \text{ kip-ft}) + (1.15)(233 \text{ kip-ft})$ $= 268 \text{ kip-ft}$	$M_r = B_1 M_{nt} + B_2 M_{lt}$ $= (1.0)(0 \text{ kip-ft}) + (1.25)(122 \text{ kip-ft})$ $= 153 \text{ kip-ft}$

The required second-order axial strength is determined using AISC *Specification* Equation A-8-2 as follows. Note, for a long frame, such as this one, the change in load to the interior columns associated with lateral load is negligible.

LRFD	ASD
$P_{nt} = 317 \text{ kips (from computer analysis)}$ $P_r = P_{nt} + B_2 P_{lt}$ $= 317 \text{ kips} + (1.15)(0 \text{ kips})$ $= 317 \text{ kips}$	$P_{nt} = 295 \text{ kips (from computer analysis)}$ $P_r = P_{nt} + B_2 P_{lt}$ $= 295 \text{ kips} + (1.25)(0 \text{ kips})$ $= 295 \text{ kips}$

Note the flexural and axial stiffness of all members in the moment frame were reduced using 0.8E in the computer analysis.

Check that the flexural stiffness was adequately reduced for the analysis per AISC *Specification* Section C2.3(b)(1).

LRFD	ASD
$\alpha = 1.0$ $P_r = 317 \text{ kips}$ Because the W14×90 column is nonslender: $P_{ns} = F_y A_g$ $= (50 \text{ ksi})(26.5 \text{ in.}^2)$ $= 1{,}330 \text{ kips}$ $\dfrac{\alpha P_r}{P_{ns}} = \dfrac{1.0(317 \text{ kips})}{1{,}330 \text{ kips}}$ $= 0.238$ Because $\alpha P_r/P_{ns} \le 0.5$: $\tau_b = 1.0$ Therefore, the previous assumption is verified.	$\alpha = 1.6$ $P_r = 295 \text{ kips}$ Because the W14×90 column is nonslender: $P_{ns} = F_y A_g$ $= (50 \text{ ksi})(26.5 \text{ in.}^2)$ $= 1{,}330 \text{ kips}$ $\dfrac{\alpha P_r}{P_{ns}} = \dfrac{1.6(295 \text{ kips})}{1{,}330 \text{ kips}}$ $= 0.355$ Because $\alpha P_r/P_{ns} \le 0.5$: $\tau_b = 1.0$ Therefore, the previous assumption is verified.

Note: By inspection $\tau_b = 1.0$ for all of the beams in the moment frame.

Interaction of Flexure and Axial

From AISC *Specification* Section H1, interaction of flexure and axial are checked as follows. From AISC *Specification* Section C3, K = 1.0 using the direct analysis method, therefore:

$$\begin{aligned} L_c &= KL \\ &= 1.0\left(13.5\text{ ft}\right) \\ &= 13.5\text{ ft} \end{aligned}$$

LRFD	ASD
From AISC *Manual* Table 6-2, for a W14×90, with L_c = 13.5 ft: $\begin{aligned} P_c &= \phi_c P_n \\ &= 1{,}040\text{ kips} \end{aligned}$	From AISC *Manual* Table 6-2, for a W14×90, with L_c = 13.5 ft: $\begin{aligned} P_c &= \dfrac{P_n}{\Omega_b} \\ &= 690\text{ kips} \end{aligned}$
From AISC *Manual* Table 6-2, for a W14×90, with L_b = 13.5 ft: $\begin{aligned} M_{cx} &= \phi_b M_{nx} \\ &= 574\text{ kip-ft} \end{aligned}$	From AISC *Manual* Table 6-2, for a W14×90, with L_b = 13.5 ft: $\begin{aligned} M_{cx} &= \dfrac{M_{nx}}{\Omega_b} \\ &= 382\text{ kip-ft} \end{aligned}$
$\begin{aligned} \dfrac{P_r}{P_c} &= \dfrac{317\text{ kips}}{1{,}040\text{ kips}} \\ &= 0.305 \end{aligned}$	$\begin{aligned} \dfrac{P_r}{P_c} &= \dfrac{295\text{ kips}}{690\text{ kips}} \\ &= 0.428 \end{aligned}$
Because $\dfrac{P_r}{P_c} \geq 0.2$, use AISC *Specification* Equation H1-1a: $\dfrac{P_r}{P_c} + \left(\dfrac{8}{9}\right)\left(\dfrac{M_{rx}}{M_{cx}} + \dfrac{M_{ry}}{M_{cy}}\right) \leq 1.0$ $0.305 + \left(\dfrac{8}{9}\right)\left(\dfrac{268\text{ kip-ft}}{574\text{ kip-ft}} + 0\right) < 1.0$ $0.720 < 1.0$ **o.k.**	Because $\dfrac{P_r}{P_c} \geq 0.2$, use AISC *Specification* Equation H1-1a: $\dfrac{P_r}{P_c} + \left(\dfrac{8}{9}\right)\left(\dfrac{M_{rx}}{M_{cx}} + \dfrac{M_{ry}}{M_{cy}}\right) \leq 1.0$ $0.428 + \left(\dfrac{8}{9}\right)\left(\dfrac{153\text{ kip-ft}}{382\text{ kip-ft}} + 0\right) < 1.0$ $0.784 < 1.0$ **o.k.**

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METHOD 2—EFFECTIVE LENGTH METHOD

Required strengths of frame members must be determined from a second-order analysis. In this example, the second-order analysis is performed by amplifying the axial forces and moments in members and connections from an approximate analysis using the provisions of AISC *Specification* Appendix 8. The available strengths of compression members are calculated using effective length factors computed from a sidesway stability analysis.

A first-order frame analysis is conducted using the load combinations for LRFD or ASD. A minimum lateral load (notional load) equal to 0.2% of the gravity loads is included for any gravity-only load combination as summarized in AISC *Manual* Part 2 titled "Required Strength, Stability, Effective Length, and Second-Order Effects." The required load combinations are given in ASCE/SEI 7.

A summary of the axial loads, moments and 1st floor drifts from the first-order computer analysis is shown below. The floor diaphragm deflection in the east-west direction was previously determined to be very small and will thus be neglected in these calculations.

LRFD	ASD
$1.23D \pm 1.0Q_E + 0.5L + 0.2S$ (Controls columns and beams)	$1.01D + 0.525Q_E + 0.75L + 0.75S$ (Controls columns and beams)
For interior column design:	For interior column design:
$P_u = 317$ kips $M_{u1} = 148$ kip-ft (from first-order analysis) $M_{u2} = 233$ kip-ft (from first-order analysis)	$P_a = 295$ kips $M_{a1} = 77.9$ kip-ft (from first-order analysis) $M_{a2} = 122$ kip-ft (from first-order analysis)
First-order story drift = 0.575 in.	First-order story drift = 0.302 in.

The required second-order flexural strength, M_r, and axial strength, P_r, are calculated as follows. For typical interior columns, the gravity load moments are approximately balanced; therefore, $M_{nt} = 0$ kip-ft.

Calculate the amplified forces and moments in accordance with AISC *Specification* Appendix 8 at the ground floor. The required second-order flexural strength is determined as follows:

$$M_r = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(\textit{Spec.} Eq. A-8-1)}$$

Determine B$_1$

Per AISC *Specification* Appendix 8, Section 8.2.1, note that for members subject to axial compression, B_1 may be calculated based on the first-order estimate; therefore:

$$P_r = P_{nt} + P_{lt}$$

where

P_r = required second-order axial strength using LRFD or ASD load combinations

From AISC *Specification* Appendix 8, Section 8.2.1, the B_1 multiplier for the W14×90 column is determined as follows:

LRFD	ASD
$B_1 = \dfrac{C_m}{1 - \dfrac{\alpha P_r}{P_{e1}}} \geq 1$ (*Spec.* Eq. A-8-3)	$B_1 = \dfrac{C_m}{1 - \dfrac{\alpha P_r}{P_{e1}}} \geq 1$ (*Spec.* Eq. A-8-3)

LRFD	ASD
where $P_r = 317$ kips (from first-order computer analysis) $I_x = 999\text{ in.}^4$ $\tau_b = 1.0$ (to be verified per *Spec*. Section C2.3(b)) $\alpha = 1.0$	where $P_r = 295$ kips (from first-order computer analysis) $I_x = 999\text{ in.}^4$ $\tau_b = 1.0$ (to be verified per *Spec*. Section C2.3(b)) $\alpha = 1.6$
$P_{e1} = \dfrac{\pi^2 EI^*}{(L_{c1})^2}$ (*Spec*. Eq. A-8-5) $= \dfrac{\pi^2(29{,}000\text{ ksi})(999\text{ in.}^4)}{[(1.0)(13.5\text{ ft})(12\text{ in./ft})]^2}$ $= 10{,}900\text{ kips}$	$P_{e1} = \dfrac{\pi^2 EI^*}{(L_{c1})^2}$ (*Spec*. Eq. A-8-5) $= \dfrac{\pi^2(29{,}000\text{ ksi})(999\text{ in.}^4)}{[(1.0)(13.5\text{ ft})(12\text{ in./ft})]^2}$ $= 10{,}900\text{ kips}$
$C_m = 0.6 - 0.4(M_1/M_2)$ (*Spec*. Eq. A-8-4) $= 0.6 - 0.4(148\text{ kip-ft}/233\text{ kip-ft})$ $= 0.346$	$C_m = 0.6 - 0.4(M_1/M_2)$ (*Spec*. Eq. A-8-4) $= 0.6 - 0.4(77.9\text{ kip-ft}/122\text{ kip-ft})$ $= 0.345$
$B_1 = \dfrac{0.346}{1 - \dfrac{1.0(317\text{ kips})}{10{,}900\text{ kips}}} < 1$ $= 0.356 < 1$	$B_1 = \dfrac{0.345}{1 - \dfrac{1.6(295\text{ kips})}{10{,}900\text{ kips}}} < 1$ $= 0.361 < 1$
Therefore, use $B_1 = 1$	Therefore, use $B_1 = 1$

Determine B$_2$

LRFD	ASD
$P_{mf} = 2{,}250$ kips (gravity load in moment frame) $P_{story} = 5{,}440$ kips (from computer output) $\Delta_H = 0.575$ in. (from computer output) $\alpha = 1.0$	$P_{mf} = 2{,}090$ kips (gravity load in moment frame) $P_{story} = 5{,}120$ kips (from computer output) $\Delta_H = 0.302$ in. (from computer output) $\alpha = 1.6$
$R_M = 1 - 0.15\left(\dfrac{P_{mf}}{P_{story}}\right)$ (*Spec*. Eq. A-8-8) $= 1 - 0.15\left(\dfrac{2{,}250\text{ kips}}{5{,}440\text{ kips}}\right)$ $= 0.938$	$R_M = 1 - 0.15\left(\dfrac{P_{mf}}{P_{story}}\right)$ (*Spec*. Eq. A-8-8) $= 1 - 0.15\left(\dfrac{2{,}090\text{ kips}}{5{,}120\text{ kips}}\right)$ $= 0.939$
From previous seismic force distribution calculations:	From previous seismic force distribution calculations:
$H = 1.0Q_E$ (Lateral) $= 1.0(196\text{ kips})$ $= 196\text{ kips}$	$H = 0.525Q_E$ (Lateral) $= 0.525(196\text{ kips})$ $= 103\text{ kips}$

LRFD	ASD
$P_{e\ story} = R_M \dfrac{HL}{\Delta_H}$ (*Spec*. Eq. A-8-7) $= 0.938 \dfrac{(196 \text{ kips})(13.5 \text{ ft})(12 \text{ in./ft})}{0.575 \text{ in.}}$ $= 51{,}800 \text{ kips}$ $B_2 = \dfrac{1}{1 - \dfrac{\alpha P_{story}}{P_{e\,story}}} \geq 1$ (*Spec*. Eq. A-8-6) $= \dfrac{1}{1 - \dfrac{1.0(5{,}440 \text{ kips})}{51{,}800 \text{ kips}}} > 1$ $= 1.12 > 1$ Note, B_2 < 1.5, therefore use of the effective length method is acceptable per AISC *Specification* Appendix 7, Section 7.2.1(b).	$P_{e\ story} = R_M \dfrac{HL}{\Delta_H}$ (*Spec*. Eq. A-8-7) $= 0.939 \dfrac{(103 \text{ kips})(13.5 \text{ ft})(12 \text{ in./ft})}{0.302 \text{ in.}}$ $= 51{,}900 \text{ kips}$ $B_2 = \dfrac{1}{1 - \dfrac{\alpha P_{story}}{P_{e\,story}}} \geq 1$ (*Spec*. Eq. A-8-6) $= \dfrac{1}{1 - \dfrac{1.6(5{,}120 \text{ kips})}{51{,}900 \text{ kips}}} > 1$ $= 1.19 > 1$ Note, B_2 < 1.5, therefore use of the effective length method is acceptable per AISC *Specification* Appendix 7, Section 7.2.1(b).

Calculate amplified moment and axial load

From AISC *Specification* Equation A-8-1, the required second-order flexural strength is determined as follows:

LRFD	ASD
$M_r = B_1 M_{nt} + B_2 M_{lt}$ $= (1)(0 \text{ kip-ft}) + (1.12)(233 \text{ kip-ft})$ $= 261 \text{ kip-ft}$	$M_r = B_1 M_{nt} + B_2 M_{lt}$ $= (1)(0 \text{ kip-ft}) + (1.19)(122 \text{ kip-ft})$ $= 145 \text{ kip-ft}$

The required second-order axial strength is determined using AISC *Specification* Equation A-8-2 as follows. Note, for a long frame, such as this one, the change in load to the interior columns associated with lateral load is negligible.

LRFD	ASD
$P_{nt} = 317$ kips (from computer analysis) $P_r = P_{nt} + B_2 P_{lt}$ $= 317 \text{ kips} + (1.12)(0 \text{ kips})$ $= 317 \text{ kips}$	$P_{nt} = 295$ kips (from computer analysis) $P_r = P_{nt} + B_2 P_{lt}$ $= 295 \text{ kips} + (1.19)(0 \text{ kips})$ $= 295 \text{ kips}$

Determine the Controlling Effective Length

For out-of-plane buckling in the moment frame, K_y = 1.0; therefore:

$$\begin{aligned} K_y L_y &= 1.0(13.5 \text{ ft}) \\ &= 13.5 \text{ ft} \end{aligned}$$

For in-plane buckling in the moment frame, use the story stiffness procedure from AISC *Specification* Commentary Appendix 7 to determine K_x.

$$K_2 = \sqrt{\left(\frac{P_{story}}{R_M P_r}\right)\left(\frac{\pi^2 EI}{L^2}\right)\left(\frac{\Delta_H}{HL}\right)} \geq \sqrt{\left(\frac{\pi^2 EI}{L^2}\right)\left(\frac{\Delta_H}{1.7 H_{col} L}\right)} \qquad \text{(}\textit{Spec}\text{. Eq. C-A-7-5)}$$

Simplifying and substituting terms previously calculated results in:

$$K_x = \sqrt{\left(\frac{P_{story}}{R_M}\right)\left(\frac{P_e}{P_r}\right)\left(\frac{ratio}{H}\right)} \geq \sqrt{P_e\left(\frac{ratio}{1.7H}\right)}$$

where

$$P_e = P_{e1}$$

$$ratio = \frac{\Delta_H}{L}$$

LRFD	ASD
$P_e = P_{e1}$ $= 10{,}900 \text{ kips}$ $ratio = \dfrac{\Delta_H}{L}$ $= \dfrac{0.575 \text{ in.}}{(13.5 \text{ ft})(12 \text{ in./ft})}$ $= 0.00355$ $K_x = \sqrt{\left(\dfrac{5{,}440 \text{ kips}}{0.938}\right)\left(\dfrac{10{,}900 \text{ kips}}{317 \text{ kips}}\right)\left(\dfrac{0.00355}{196 \text{ kips}}\right)} \geq$ $\sqrt{(10{,}900 \text{ kips})\left[\dfrac{0.00355}{1.7(196 \text{ kips})}\right]}$ $= 1.90 > 0.341$ Therefore, use K_x = 1.90. From AISC *Manual* Table 4-1a, for a W14×90: $r_x/r_y = 1.66$ $L_{cy\ eq} = \dfrac{KL_x}{r_x/r_y}$ (from *Manual* Eq. 4-1) $= \dfrac{1.90(13.5 \text{ ft})}{1.66}$ $= 15.5 \text{ ft}$ Because $L_{cy\ eq} > L_{cy}$, use L_c = 15.5 ft.	$P_e = P_{e1}$ $= 10{,}900 \text{ kips}$ $ratio = \dfrac{\Delta_H}{L}$ $= \dfrac{0.302 \text{ in.}}{(13.5 \text{ ft})(12 \text{ in./ft})}$ $= 0.00186$ $K_x = \sqrt{\left(\dfrac{5{,}120 \text{ kips}}{0.939}\right)\left(\dfrac{10{,}900 \text{ kips}}{295 \text{ kips}}\right)\left(\dfrac{0.00186}{103 \text{ kips}}\right)} \geq$ $\sqrt{(10{,}900 \text{ kips})\left[\dfrac{0.00186}{1.7(103 \text{ kips})}\right]}$ $= 1.91 > 0.340$ Therefore, use K_x = 1.91. From AISC *Manual* Table 4-1a, for a W14×90: $r_x/r_y = 1.66$ $L_{cy\ eq} = \dfrac{KL_x}{r_x/r_y}$ (from *Manual* Eq. 4-1) $= \dfrac{1.91(13.5 \text{ ft})}{1.66}$ $= 15.5 \text{ ft}$ Because $L_{cy\ eq} > L_{cy}$, use L_c = 15.5 ft.

Interaction of Flexure and Axial

From AISC *Specification* Section H1, interaction of flexure and axial are checked as follows:

LRFD	ASD
From AISC *Manual* Table 6-2, for a W14×90, with L_c = 15.5 ft: $P_c = \phi_c P_n$ $= 990\text{ kips}$	From AISC *Manual* Table 6-2, for a W14×90, with L_c = 15.5 ft: $P_c = \dfrac{P_n}{\Omega_c}$ $= 660\text{ kips}$
From AISC *Manual* Table 6-2, for a W14×90, with L_b = 13.5 ft: $M_{cx} = \phi_b M_{nx}$ $= 574\text{ kip-ft}$	From AISC *Manual* Table 6-2, for a W14×90, with L_b = 13.5 ft: $M_{cx} = \dfrac{M_{nx}}{\Omega_b}$ $= 382\text{ kip-ft}$
$\dfrac{P_r}{P_c} = \dfrac{317\text{ kips}}{990\text{ kips}}$ $= 0.320$	$\dfrac{P_r}{P_c} = \dfrac{295\text{ kips}}{660\text{ kips}}$ $= 0.447$
Because $\dfrac{P_r}{P_c} \geq 0.2$, use AISC *Specification* Equation H1-1a: $\dfrac{P_r}{P_c} + \left(\dfrac{8}{9}\right)\left(\dfrac{M_{rx}}{M_{cx}} + \dfrac{M_{ry}}{M_{cy}}\right) \leq 1.0$ $0.320 + \left(\dfrac{8}{9}\right)\left(\dfrac{261\text{ kip-ft}}{574\text{ kip-ft}}\right) < 1.0$ $0.724 < 1.0$ **o.k.**	Because $\dfrac{P_r}{P_c} \geq 0.2$, use AISC *Specification* Equation H1-1a: $\dfrac{P_r}{P_c} + \left(\dfrac{8}{9}\right)\left(\dfrac{M_{rx}}{M_{cx}} + \dfrac{M_{ry}}{M_{cy}}\right) \leq 1.0$ $0.447 + \left(\dfrac{8}{9}\right)\left(\dfrac{145\text{ kip-ft}}{382\text{ kip-ft}}\right) < 1.0$ $0.784 < 1.0$ **o.k.**

METHOD 3—SIMPLIFIED EFFECTIVE LENGTH METHOD

A simplification of the effective length method using a method of second-order analysis based upon drift limits and other assumptions is described in Part 2 of the AISC *Manual* titled “Simplified Determination of Required Strength.” A first-order frame analysis is conducted using the load combinations for LRFD or ASD. A minimum lateral load (notional load) equal to 0.2% of the gravity loads is included for all gravity-only load combinations. The floor diaphragm deflection in the east-west direction was previously determined to be very small and will thus be neglected in these calculations.

LRFD	ASD
$1.23D \pm 1.0Q_E + 0.5L + 0.2S$ (Controls columns and beams) For interior column design: $P_u = 317$ kips $M_{u1} = 148$ kip-ft (from first-order analysis) $M_{u2} = 233$ kip-ft (from first-order analysis) First-order first story drift = 0.575 in.	$1.01D + 0.525Q_E + 0.75L + 0.75S$ (Controls columns and beams) For interior column design: $P_a = 295$ kips $M_{a1} = 77.9$ kip-ft (from first-order analysis) $M_{a2} = 122$ kip-ft (from first-order analysis) First-order first story drift = 0.302 in.

Calculate the amplified forces and moments in accordance with AISC *Manual* Part 2 at the ground floor. The following steps are executed.

LRFD	ASD
Step 1: Lateral load = 196 kips Deflection due to first-order elastic analysis Δ = 0.575 in., between first and second floor Floor height = 13.5 ft $\text{Drift ratio} = \dfrac{(13.5\text{ ft})(12\text{ in./ft})}{0.575\text{ in.}} = 282$ *Step 2:* Design story drift limit = *H*/400 $\text{Adjusted lateral load} = \left(\dfrac{282}{400}\right)(196\text{ kips}) = 138\text{ kips}$	*Step 1:* Lateral load = 103 kips Deflection due to first-order elastic analysis Δ = 0.302 in., between first and second floor Floor height = 13.5 ft $\text{Drift ratio} = \dfrac{(13.5\text{ ft})(12\text{ in./ft})}{0.302\text{ in.}} = 536$ *Step 2:* Design story drift limit = *H*/400 $\text{Adjusted lateral load} = \left(\dfrac{536}{400}\right)(103\text{ kips}) = 138\text{ kips}$

LRFD	ASD
Step 3:	*Step 3:* (for an ASD design the ratio must be multiplied by 1.6)
$\text{Load ratio} = (1.0)\left(\dfrac{\text{total story load}}{\text{lateral load}}\right)$ $= (1.0)\left(\dfrac{5{,}440\text{ kips}}{138\text{ kips}}\right)$ $= 39.4$	$\text{Load ratio} = (1.6)\left(\dfrac{\text{total story load}}{\text{lateral load}}\right)$ $= (1.6)\left(\dfrac{5{,}120\text{ kips}}{138\text{ kips}}\right)$ $= 59.4$
From AISC *Manual* Table 2-1:	From AISC *Manual* Table 2-1:
$B_2 = 1.1$	$B_2 = 1.2$
Which matches the value obtained in Method 2 to the two significant figures of the table	Which matches the value obtained in Method 2 to the two significant figures of the table

Note: Intermediate values are not interpolated from the table because the precision of the table is two significant digits. Additionally, the design story drift limit used in Step 2 need not be the same as other strength or serviceability drift limits used during the analysis and design of the structure.

Step 4:

Multiply all the forces and moment from the first-order analysis by the value of B_2 obtained from the table. This presumes that B_1 is less than or equal to B_2, which is usually the case for members without transverse loading between their ends.

LRFD	ASD
Step 5:	*Step* 5:
Since the selection is in the shaded area of the chart, ($B_2 \le 1.1$), use $K = 1.0$.	Since the selection is in the unshaded area of the chart ($B_2 > 1.1$), the effective length factor, K, must be determined through analysis. From previous analysis, use an effective length of 15.5 ft.
Multiply both sway and nonsway moments by B_2.	Multiply both sway and nonsway moments by B_2.
$M_r = B_2\left(M_{nt} + M_{lt}\right)$ $= 1.1(0\text{ kip-ft} + 233\text{ kip-ft})$ $= 256\text{ kip-ft}$	$M_r = B_2\left(M_{nt} + M_{lt}\right)$ $= 1.2(0\text{ kip-ft} + 122\text{ kip-ft})$ $= 146\text{ kip-ft}$
$P_r = B_2\left(P_{nt} + P_{lt}\right)$ $= 1.1(317\text{ kips} + 0\text{ kips})$ $= 349\text{ kips}$	$P_r = B_2\left(P_{nt} + P_{lt}\right)$ $= 1.2(295\text{ kips} + 0\text{ kips})$ $= 354\text{ kips}$
From AISC *Manual* Table 6-2, for a W14×90, with $L_c = 13.5$ ft:	From AISC *Manual* Table 6-2, for a W14×90, with $L_c = 15.5$ ft:
$P_c = \phi_c P_n$ $= 1{,}040\text{ kips}$	$P_c = \dfrac{P_n}{\Omega_c}$ $= 660\text{ kips}$

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LRFD	ASD
From AISC *Manual* Table 6-2, for a W14×90, with L_b = 13.5 ft:	From AISC *Manual* Table 6-2, for a W14×90, with L_b = 13.5 ft:
$M_{cx} = \phi_b M_{nx}$ $= 574 \text{ kip-ft}$	$M_{cx} = \dfrac{M_{nx}}{\Omega_b}$ $= 382 \text{ kip-ft}$
$\dfrac{P_r}{P_c} = \dfrac{349 \text{ kips}}{1{,}040 \text{ kips}}$ $= 0.336$	$\dfrac{P_r}{P_c} = \dfrac{354 \text{ kips}}{660 \text{ kips}}$ $= 0.536$
Because $\dfrac{P_r}{P_c} \geq 0.2$, use AISC *Specification* Equation H1-1a:	Because $\dfrac{P_r}{P_c} \geq 0.2$, use AISC *Specification* Equation H1-1a:
$\dfrac{P_r}{P_c} + \left(\dfrac{8}{9}\right)\left(\dfrac{M_{rx}}{M_{cx}} + \dfrac{M_{ry}}{M_{cy}}\right) \leq 1.0$ $0.336 + \left(\dfrac{8}{9}\right)\left(\dfrac{256 \text{ kip-ft}}{574 \text{ kip-ft}} + 0\right) < 1.0$ $0.732 < 1.0$ **o.k.**	$\dfrac{P_r}{P_c} + \left(\dfrac{8}{9}\right)\left(\dfrac{M_{rx}}{M_{cx}} + \dfrac{M_{ry}}{M_{cy}}\right) \leq 1.0$ $0.536 + \left(\dfrac{8}{9}\right)\left(\dfrac{146 \text{ kip-ft}}{382 \text{ kip-ft}} + 0\right) < 1.0$ $0.876 < 1.0$ **o.k.**

BEAM ANALYSIS IN THE MOMENT FRAME

The controlling load combinations for the beams in the moment frames are shown in Tables III-11 and III-12, and evaluated for the second floor beam. The dead load, live load and seismic moments were taken from a computer analysis. These tables summarizes the calculation of B_2 for the stories above and below the second floor.

Table III-11
Summary of B_2 Calculation for Controlling Load Combination—First to Second Floor

1st – 2nd	LRFD Combination	ASD Combination 1	ASD Combination 2
	$1.23D + 1.0Q_E + 0.5L + 0.2S$	$1.02D + 0.7Q_E$	$1.01D + 0.525Q_E + 0.75L + 0.75S$
H	196 kips	137 kips	103 kips
L	13.5 ft	13.5 ft	13.5 ft
Δ_H	0.575 in.	0.402 in.	0.302 in.
P_{mf}	2,250 kips	1,640 kips	2,090 kips
R_M	0.938	0.937	0.939
$P_{e\ story}$	51,800 kips	51,700 kips	51,900 kips
P_{story}	5,440 kips	3,920 kips	5,120 kips
B_2	1.12	1.14	1.19

Table III-12
Summary of B_2 Calculation for Controlling Load Combination—Second to Third Floor

2nd – 3rd	LRFD Combination	ASD Combination 1	ASD Combination 2
	$1.23D + 1.0Q_E + 0.5L + 0.2S$	$1.02D + 0.7Q_E$	$1.01D + 0.525Q_E + 0.75L + 0.75S$
H	170 kips	119 kips	89.3 kips
L	13.5 ft	13.5 ft	13.5 ft
Δ_H	0.728 in.	0.509 in.	0.382 in.
P_{mf}	1,590 kips	1,160 kips	1,490 kips
R_M	0.938	0.937	0.939
$P_{e\ story}$	35,500 kips	35,500 kips	35,600 kips
P_{story}	3,840 kips	2,770 kips	3,660 kips
B_2	1.12	1.14	1.20

For beam members, the larger of the B_2 values from the story above or below is used.

From computer output at the controlling beam:

$$M_{dead} = 153 \text{ kip-ft}$$
$$M_{live} = 80.6 \text{ kip-ft}$$
$$M_{snow} = 0 \text{ kip-ft}$$
$$M_{earthquake} = 154 \text{ kip-ft}$$

LRFD	ASD
$B_2M_{lt} = 1.12(154 \text{ kip-ft})$ $= 172 \text{ kip-ft}$ $M_u = \left[\begin{array}{l} 1.23(153 \text{ kip-ft}) + 1.0(172 \text{ kip-ft}) \\ +0.5(80.6 \text{ kip-ft}) \end{array}\right]$ $= 400 \text{ kip-ft}$	Combination 1: $B_2M_{lt} = 1.14(154 \text{ kip-ft})$ $= 176 \text{ kip-ft}$ $M_a = 1.02(153 \text{ kip-ft}) + 0.7(176 \text{ kip-ft})$ $= 279 \text{ kip-ft}$

LRFD	ASD
	Combination 2: $B_2M_{lt} = 1.20(154 \text{ kip-ft})$ $= 185 \text{ kip-ft}$ $M_a = \begin{bmatrix} 1.01(153 \text{ kip-ft}) + 0.525(185 \text{ kip-ft}) \\ +0.75(80.6 \text{ kip-ft}) \end{bmatrix}$ $= 312 \text{ kip-ft}$

Calculate C_b for W24×55 beam with compression in the bottom flange braced at 10 ft on center.

LRFD	ASD
For load combination 1.23D + 1.0Q_E + 0.5L + 0.2S:	For load combination 1.02D + 0.7Q_E:
From AISC *Manual* Table 6-2 with L_b = 0 ft (fully braced):	From AISC *Manual* Table 6-2 with L_b = 0 ft (fully braced):
$\phi_b M_n = 503 \text{ kip-ft}$	$\dfrac{M_n}{\Omega_b} = 334 \text{ kip-ft}$
C_b = 1.86 (from computer output)	C_b = 1.86 (from computer output)
From AISC *Manual* Table 6-2 with L_b = 10 ft:	From AISC *Manual* Table 6-2 with L_b = 10 ft:
$\phi_b M_n C_b \le \phi_b M_p$ $(386 \text{ kip-ft})(1.86) = 718 \text{ kip-ft} > 503 \text{ kip-ft}$	$\dfrac{M_n}{\Omega_b} C_b \le \dfrac{M_p}{\Omega_b}$ $(257 \text{ kip-ft})(1.86) = 478 \text{ kip-ft} > 334 \text{ kip-ft}$
Therefore:	Therefore:
$\phi M_n = 503 \text{ kip-ft} > 400 \text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega} = 334 \text{ kip-ft} > 279 \text{ kip-ft}$ **o.k.**

LRFD	ASD
	For load combination $1.01D + 0.525Q_E + 0.75L$: From AISC *Manual* Table 6-2 with $L_b = 0$ ft (fully braced): $\dfrac{M_n}{\Omega_b} = 334 \text{ kip-ft}$ $C_b = 2.01$ (from computer output) From AISC *Manual* Table 6-2 with $L_b = 10$ ft : $\dfrac{M_n}{\Omega_b} C_b \le \dfrac{M_p}{\Omega_b}$ $(257 \text{ kip-ft})(2.01) = 517 \text{ kip-ft} > 334 \text{ kip-ft}$ Therefore: $\dfrac{M_n}{\Omega} = 334 \text{ kip-ft} > 312 \text{ kip-ft}$ **o.k.**
From AISC *Manual* Table 6-2, a W24×55 has a design shear strength of 252 kips and an I_x of 1,350 in.^{4}	From AISC *Manual* Table 6-2, a W24×55 has an allowable shear strength of 167 kips and an I_x of 1,350 in.^{4}

The moments and shears on the roof beams due to the lateral loads were also checked but do not control the design.

The connections of these beams can be designed by one of the techniques illustrated in the Chapter IIB of the design examples.

BRACED FRAME ANALYSIS

The braced frames at Grids 1 and 8 were analyzed for the required load combinations. The stability design requirements from Chapter C were applied to this system.

The model layout is shown in Figure III-24. The nominal dead, live, and snow loads with associated notional loads, wind loads and seismic loads are shown in Figures III-25 and III-26.

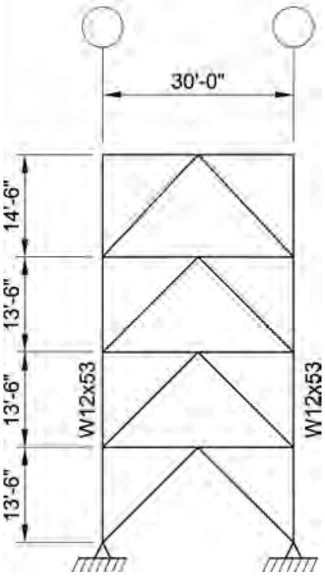
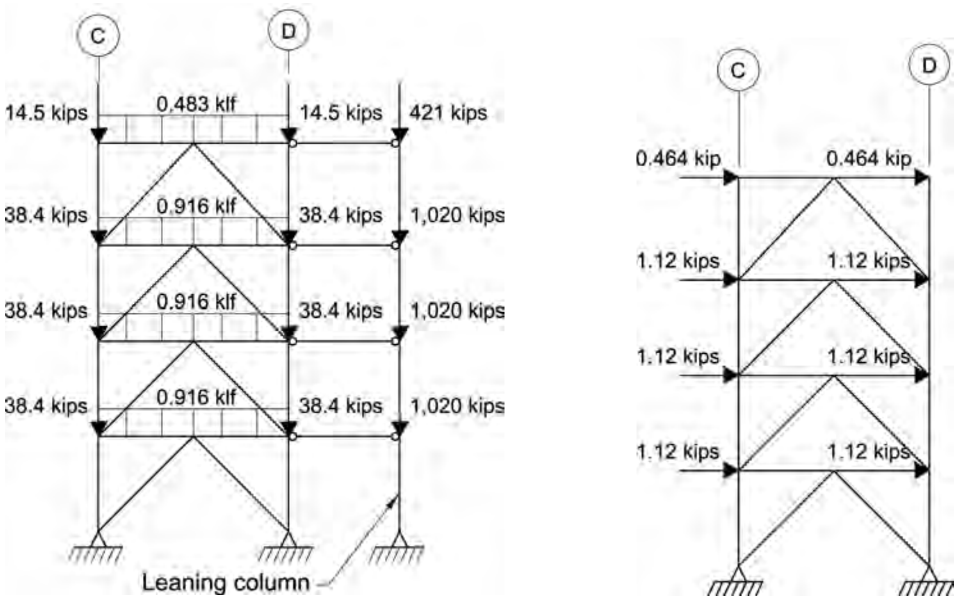
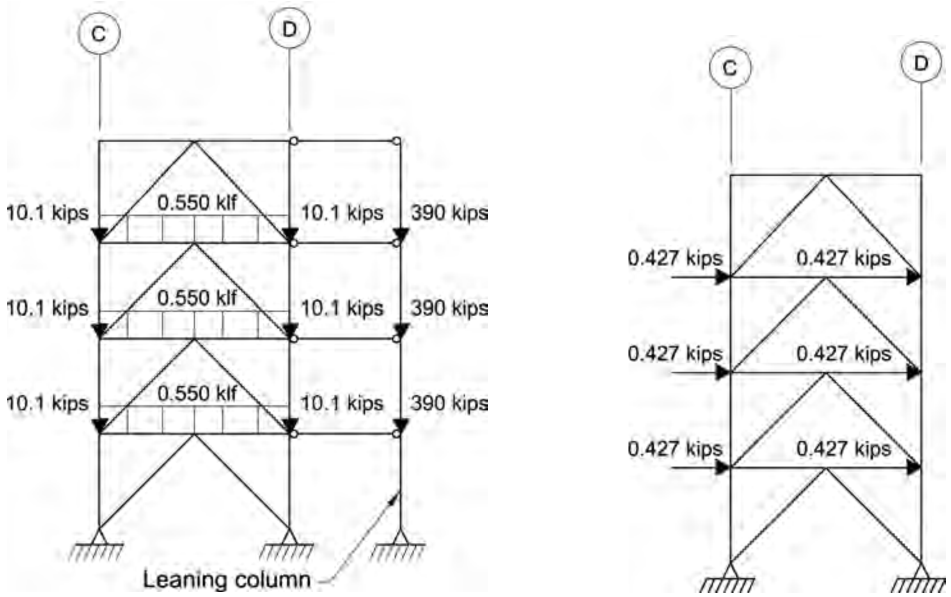


Fig. III-24. Braced frame layout—Grid 1 and 8.



(a) Nominal dead loads *(b) Notional dead loads*



(c) Nominal live loads *(d) Notional live loads*

Fig. III-25. Dead and live loads.

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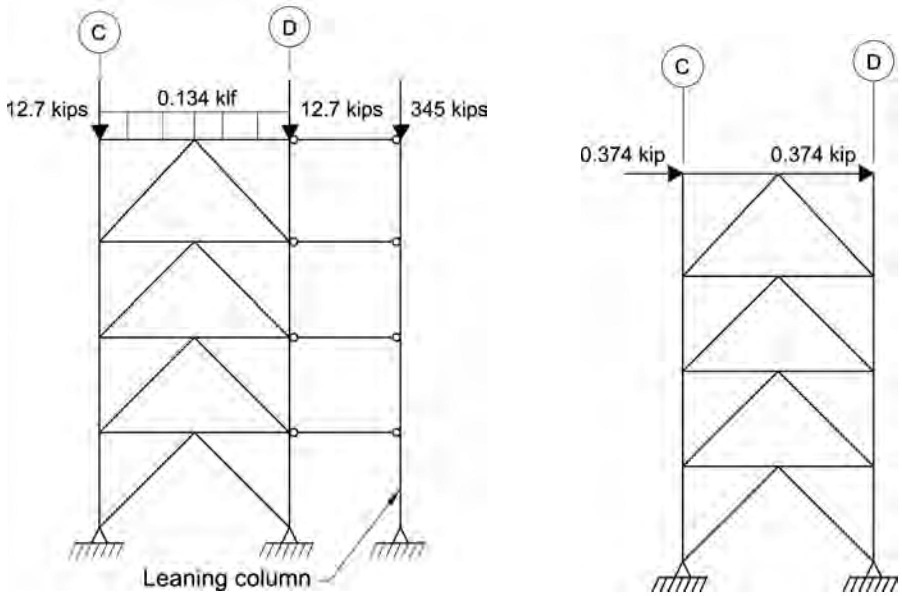
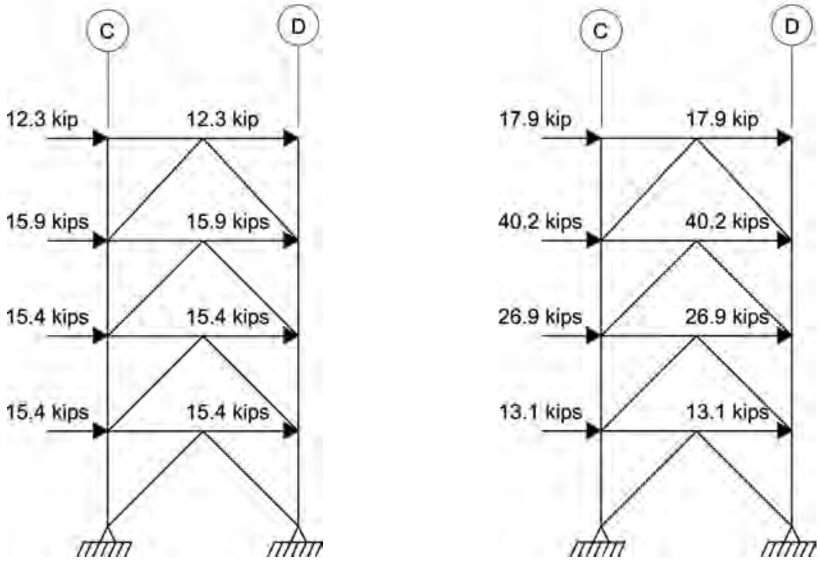


(a) Nominal snow loads *(b) Notional snow loads*



(c) Wind loads (1.0W) *(d) Seismic loads (1.0Q_E)*

Fig. III-26. Snow, wind and seismic loads.

Second-order analysis by amplified first-order analysis

In the following, the approximate second-order analysis method from AISC *Specification* Appendix 8 is used to account for second-order effects in the braced frames by amplifying the axial forces in members and connections from a first-order analysis.

A first-order frame analysis is conducted using the load combinations for LRFD and ASD. From this analysis the critical axial loads, moments and deflections are obtained.

A summary of the axial loads and first floor drifts from the first-order computer analysis is shown below. The floor diaphragm deflection in the north-south direction was previously determined to be very small and will thus be neglected in these calculations.

The required seismic load combinations, as given in ASCE/SEI 7, Section 12.4, were derived previously.

LRFD	ASD
$1.23D \pm 1.0Q_E + 0.5L + 0.2S$ (Controls columns and beams)	$1.01D + 0.525Q_E + 0.75L + 0.75S$ (Controls columns and beams)
From first-order analysis.	From first-order analysis.
For interior column design:	For interior column design:
$P_{nt} = 236$ kips $P_{lt} = 146$ kips	$P_{nt} = 219$ kips $P_{lt} = 76.6$ kips
The moments are negligible.	The moments are negligible.
First-order first story drift = 0.211 in.	First-order first story drift = 0.111 in.

The required second-order axial strength, P_r, is computed as follows:

LRFD	ASD
$P_r = P_{nt} + B_2 P_{lt}$ (*Spec*. Eq. A-8-2)	$P_r = P_{nt} + B_2 P_{lt}$ (*Spec*. Eq. A-8-2)
Determine B_2.	Determine B_2.
$B_2 = \dfrac{1}{1 - \dfrac{\alpha P_{story}}{P_{e\,story}}} \geq 1$ (*Spec*. Eq. A-8-6)	$B_2 = \dfrac{1}{1 - \dfrac{\alpha P_{story}}{P_{e\,story}}} \geq 1$ (*Spec*. Eq. A-8-6)
$P_{story} = 5{,}440$ kips (previously calculated)	$P_{story} = 5{,}120$ kips (previously calculated)
$P_{e\,story} = R_M \dfrac{HL}{\Delta_H}$ (*Spec*. Eq. A-8-7)	$P_{e\,story} = R_M \dfrac{HL}{\Delta_H}$ (*Spec*. Eq. A-8-7)
where H = 196 kips (from previous calculations) Δ_H = 0.211 in. (from computer output) R_M = 1.0 for braced frames	where H = 103 kips (from previous calculations) Δ_H = 0.111 in. (from computer output) R_M = 1.0 for braced frames

LRFD	ASD
$P_{e\,story} = (1.0)\left[\dfrac{(196\text{ kips})(13.5\text{ ft})(12\text{ in./ft})}{0.211\text{ in.}}\right]$ $= 150{,}000\text{ kips}$	$P_{e\,story} = (1.0)\left[\dfrac{(103\text{ kips})(13.5\text{ ft})(12\text{ in./ft})}{0.111\text{ in.}}\right]$ $= 150{,}000\text{ kips}$
$B_2 = \dfrac{1}{1-\dfrac{1.0(5{,}440\text{ kips})}{150{,}000\text{ kips}}} > 1$ $= 1.04 > 1$	$B_2 = \dfrac{1}{1-\dfrac{1.6(5{,}120\text{ kips})}{150{,}000\text{ kips}}} > 1$ $= 1.06 > 1$
Therefore, use B_2 = 1.04.	Therefore, use B_2 = 1.06.
$P_r = P_{nt} + B_2P_{lt}$ (*Spec*. Eq. A-8-2) $= 236\text{ kips} + (1.04)(146\text{ kips})$ $= 388\text{ kips}$	$P_r = P_{nt} + B_2P_{lt}$ (*Spec*. Eq. A-8-2) $= 219\text{ kips} + (1.06)(76.6\text{ kips})$ $= 300\text{ kips}$
From AISC *Manual* Table 6-2 for a W12×53 with L_c = 13.5 ft:	From AISC *Manual* Table 6-2 for a W12×53 with L_c = 13.5 ft:
$P_c = \phi_c P_n$ $= 514\text{ kips}$	$P_c = \dfrac{P_n}{\Omega_c}$ $= 342\text{ kips}$
From AISC *Specification* Equation H1-1a:	From AISC *Specification* Equation H1-1a:
$\dfrac{P_r}{P_c} = \dfrac{388\text{ kips}}{514\text{ kips}} \le 1.0$ $= 0.755 < 1.0$ **o.k.**	$\dfrac{P_r}{P_c} = \dfrac{300\text{ kips}}{342\text{ kips}} \le 1.0$ $= 0.877 < 1.0$ **o.k.**

Note: Notice that the lower sidesway displacements of the braced frame produce much lower values of B_2 than those of the moment frame. Similar results could be expected for the other two methods of analysis.

Although not presented here, second-order effects should be accounted for in the design of the beams and diagonal braces in the braced frames at Grids 1 and 8.

ANALYSIS OF DRAG STRUTS

The fourth floor delivers the highest diaphragm force to the braced frames at the ends of the building: Q_E = 80.3 kips (from previous calculations). This force is transferred to the braced frame through axial loading of the W18×35 beams at the end of the building.

The gravity dead loads for the edge beams are the floor loading of 75 psf (5.50 ft) plus the exterior wall loading of 0.503 kip/ft, giving a total dead load of 0.916 kip/ft. The gravity live load for these beams is the floor loading of 80 psf (5.50 ft) = 0.440 kip/ft. The resulting midspan moments are M_D = 58.0 kip-ft and M_L = 27.8 kip-ft.

The required seismic load combinations, as given in ASCE/SEI 7, Section 12.4, were derived previously. The controlling load combination for LRFD is 1.23D + 1.0Q_E + 0.5L. The controlling load combinations for ASD are 1.01D + 0.525Q_E + 0.75L or 1.02D + 0.7Q_E.

LRFD	ASD
$M_u = 1.23M_D + 0.5M_L$ $= 1.23(58.0\text{ kip-ft}) + 0.5(27.8\text{ kip-ft})$ $= 85.2\text{ kip-ft}$	$M_a = 1.01M_D + 0.75M_L$ $= 1.01(58.0\text{ kip-ft}) + 0.75(27.8\text{ kip-ft})$ $= 79.4\text{ kip-ft}$ or $M_a = 1.02M_D$ $= 1.02(58.0\text{ kip-ft})$ $= 59.2\text{ kip-ft}$
Load from the diaphragm shear due to earthquake loading $F_p = 1.0Q_E$ $= 1.0(80.3\text{ kips})$ $= 80.3\text{ kips}$	Load from the diaphragm shear due to earthquake loading $F_p = 0.525Q_E$ $= 0.525(80.3\text{ kips})$ $= 42.2\text{ kips}$ or $F_p = 0.7Q_E$ $= 0.7(80.3\text{ kips})$ $= 56.2\text{ kips}$

Only the two 45-ft-long segments on either side of the brace can transfer load into the brace, because the stair opening is in front of the brace.

Use AISC *Specification* Section H2 to check the combined bending and axial stresses.

LRFD	ASD
$V = \dfrac{80.3\text{ kips}}{2(45\text{ ft})}$ $= 0.892\text{ kip/ft}$	$V = \dfrac{42.2\text{ kips}}{2(45\text{ ft})}$ $= 0.469\text{ kip/ft}$ or $V = \dfrac{56.2\text{ kips}}{2(45\text{ ft})}$ $= 0.624\text{ kip/ft}$

From AISC *Manual* Table 1-1, for a W18×35:

$$S_x = 57.6\text{ in.}^3$$

LRFD	ASD
The top flange bending stress is: $f_{rbw} = \dfrac{M_u}{S_x}$ $= \dfrac{(85.2\text{ kip-ft})(12\text{ in./ft})}{57.6\text{ in.}^3}$ $= 17.8\text{ ksi}$	The top flange bending stress is: $f_{rbw} = \dfrac{M_a}{S_x}$ $= \dfrac{(79.4\text{ kip-ft})(12\text{ in./ft})}{57.6\text{ in.}^3}$ $= 16.5\text{ ksi}$ or $f_{rbw} = \dfrac{M_a}{S_x}$ $= \dfrac{(59.2\text{ kip-ft})(12\text{ in./ft})}{57.6\text{ in.}^3}$ $= 12.3\text{ ksi}$

Note: It is often possible to resist the drag strut force using the slab directly. For illustration purposes, this solution will instead use the beam to resist the force independently of the slab. The full cross section can be used to resist the force if the member is designed as a column braced at one flange only (plus any other intermediate bracing present, such as from filler beams). Alternatively, a reduced cross section consisting of the top flange plus a portion of the web can be used. Arbitrarily use the top flange and 8 times an area of the web equal to its thickness times a depth equal to its thickness, as an area to carry the drag strut component.

$$\begin{aligned}\text{Area} &= b_f t_f + 8(t_w)^2\\ &= (6.00\text{ in.})(0.425\text{ in.}) + 8(0.300\text{ in.})^2\\ &= 3.27\text{ in.}^2\end{aligned}$$

Ignoring the small segment of the beam between Grid C and D, the axial stress due to the drag strut force is:

LRFD	ASD
$f_{ra} = \dfrac{80.3\text{ kips}}{2\left(3.27\text{ in.}^2\right)}$ $= 12.3\text{ ksi}$	$f_{ra} = \dfrac{42.2\text{ kips}}{2\left(3.27\text{ in.}^2\right)}$ $= 6.45\text{ ksi}$ or $f_{ra} = \dfrac{56.2\text{ kips}}{2\left(3.27\text{ in.}^2\right)}$ $= 8.59\text{ ksi}$

LRFD	ASD
Using AISC *Specification* Section H2, assuming the top flange is continuously braced: $F_{ca} = \phi_c F_y$ $= 0.90(50\text{ ksi})$ $= 45.0\text{ ksi}$ $F_{cbw} = \phi_b F_y$ $= 0.90(50\text{ ksi})$ $= 45.0\text{ ksi}$ $\dfrac{f_{ra}}{F_{ca}} + \dfrac{f_{rbw}}{F_{cbw}} \leq 1.0$ (from *Spec*. Eq. H2-1) $\dfrac{12.3\text{ ksi}}{45.0\text{ ksi}} + \dfrac{17.8\text{ ksi}}{45.0\text{ ksi}} = 0.669 < 1.0$ **o.k.**	From AISC *Specification* Section H2, assuming the top flange is continuously braced: $F_{ca} = F_y / \Omega_c$ $= 50\text{ ksi}/1.67$ $= 29.9\text{ ksi}$ $F_{cbw} = \dfrac{F_y}{\Omega_b}$ $= 50\text{ ksi}/1.67$ $= 29.9\text{ ksi}$ $\dfrac{f_{ra}}{F_{ca}} + \dfrac{f_{rbw}}{F_{cbw}} \leq 1.0$ (from *Spec*. Eq. H2-1) Load Combination 1: $\dfrac{6.45\text{ ksi}}{29.9\text{ ksi}} + \dfrac{16.5\text{ ksi}}{29.9\text{ ksi}} = 0.768 < 1.0$ **o.k.** Load Combination 2: $\dfrac{8.59\text{ ksi}}{29.9\text{ ksi}} + \dfrac{12.3\text{ ksi}}{29.9\text{ ksi}} = 0.699 < 1.0$ **o.k.**

Note: Because the drag strut load is a horizontal load, the method of transfer into the strut, and the extra horizontal load that must be accommodated by the beam end connections should be indicated on the drawings.

PART III EXAMPLE REFERENCES

ASCE (2014), *Design Loads on Structures During Construction*, ASCE/SEI 37-14, American Society of Civil Engineers, Reston, VA.

Geschwindner, L.F. (1994), “A Practical Approach to the Leaning Column,” *Engineering Journal*, AISC, Vol. 31, No. 4, pp. 141–149.

SDI (2014), *Floor Deck Design Manual*, 1st Ed., Steel Deck Institute, Glenshaw, PA.

SDI (2015), *Diaphragm Design Manual*, 4th Ed., Steel Deck Institute, Glenshaw, PA.

SJI (2015), *Load Tables and Weight Tables for Steel Joists and Joist Girders*, 44th Ed., Steel Joist Institute, Forest, VA.

West, M.A. and Fisher, J.M. (2003), *Serviceability Design Considerations for Steel Buildings*, Design Guide 3, 2nd Ed., AISC, Chicago, IL.

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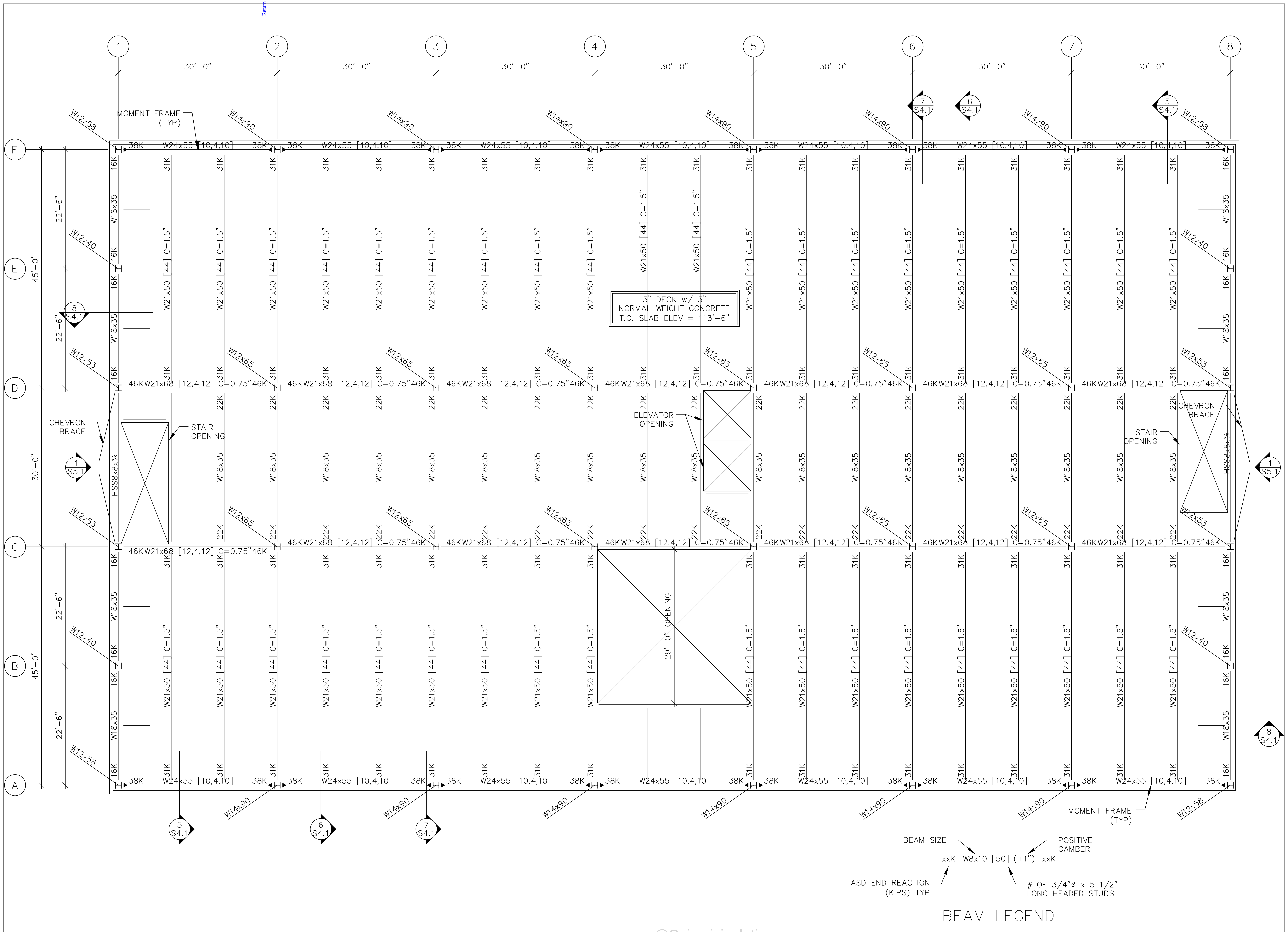

2ND FLOOR FRAMING PLAN
1/8" = 1'–0"
1 S2.1
NORTH

ABC ARCHITECTS ASSOCIATES
123456 STREET ADDRESS
SUITE • CITY, STATE ZIP
PHONE NUMBER – FAX NUMBER
ON THE WEB

– ABLE –
STRUCTURAL
ENGINEERS
654321 ADDRESS STREET
SUITE • CITY, STATE ZIP
PHONE NUMBER – FAX NUMBER
ON THE WEB

BUILDING 3
ANYWHERE

SEAL

REVISIONS

DESCRIPTION	DATE
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DATE:
PROJECT #: 123456
DRAWN BY: ME
CHECKED BY: YOU
ISSUE:

2ND FLOOR FRAMING PLAN

S2.1

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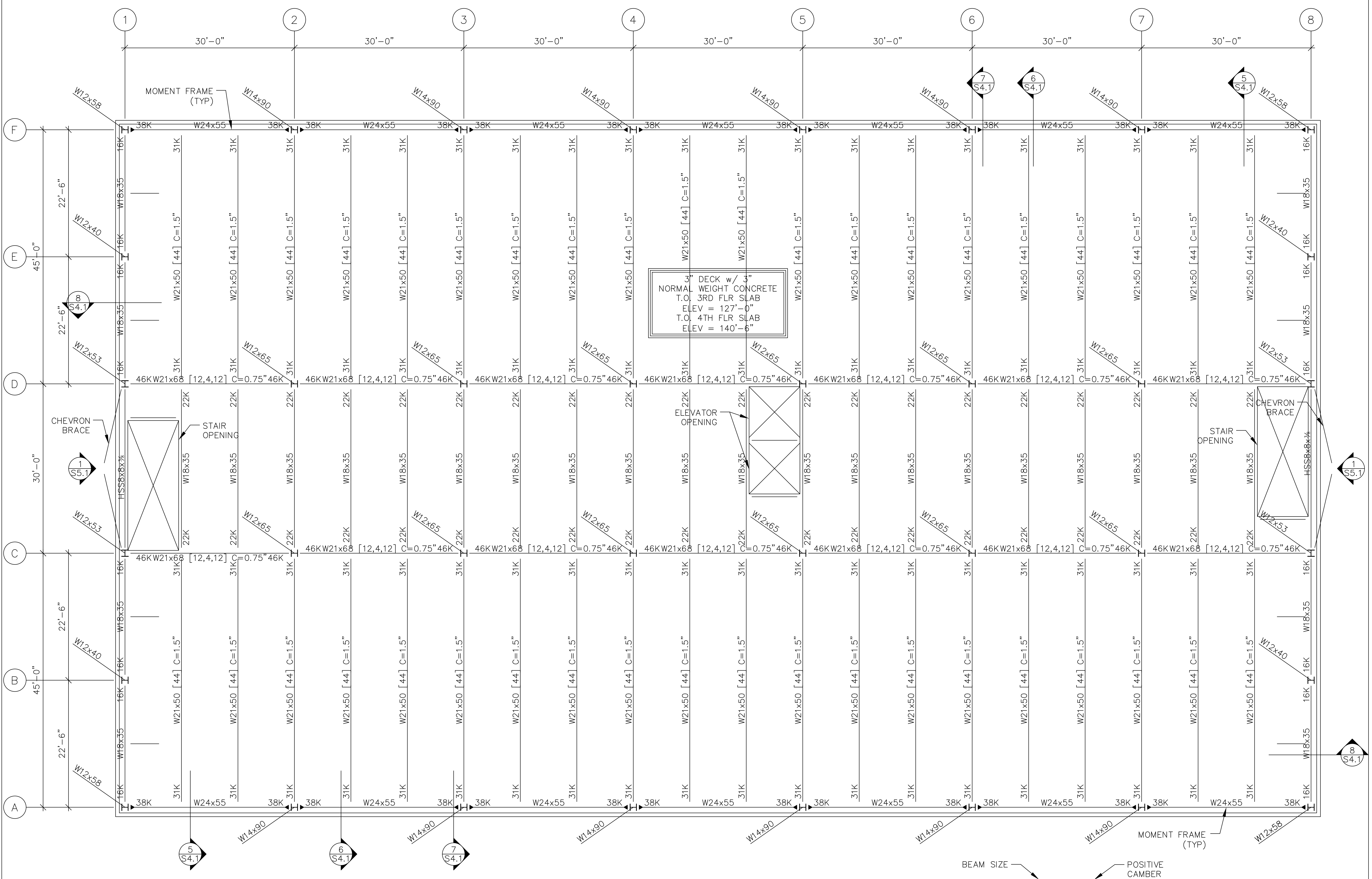

3RD & 4TH FLOOR FRAMING PLAN

1/8" = 1'-0"

1 S2.2

ABC ARCHITECTS ASSOCIATES
123456 STREET ADDRESS
SUITE · CITY, STATE ZIP
PHONE NUMBER – FAX NUMBER
ON THE WEB

— ABLE —
STRUCTURAL
ENGINEERS
654321 ADDRESS STREET
SUITE · CITY, STATE ZIP
PHONE NUMBER – FAX NUMBER
ON THE WEB

BUILDING 3
ANYWHERE

SEAL

REVISIONS

DESCRIPTION	DATE
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DATE:
PROJECT # 123456
DRAWN BY: ME
CHECKED BY: YOU
ISSUE:

3RD & 4TH FLOOR FRAMING PLAN

S2.2

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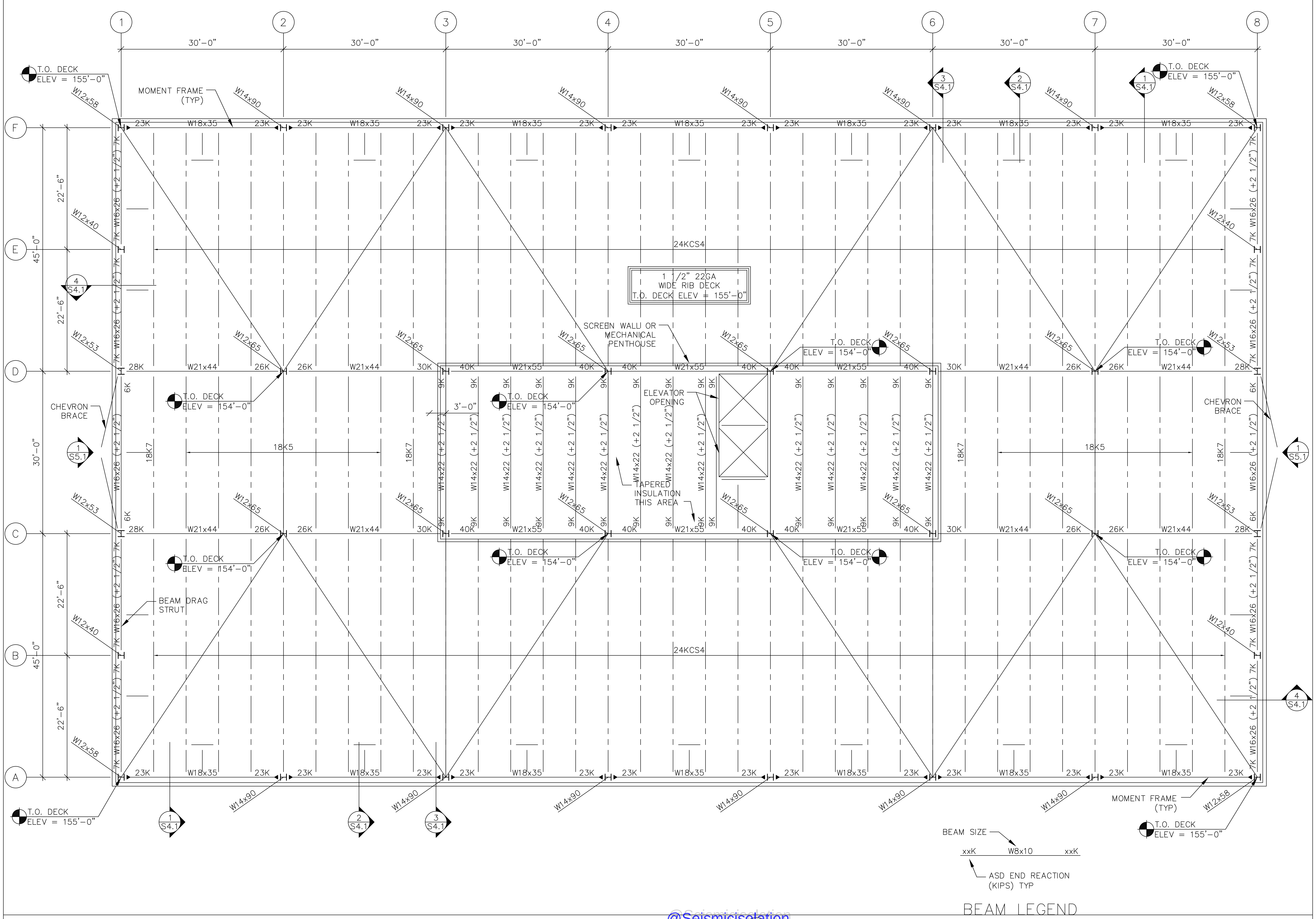

ROOF FRAMING PLAN
1/8" = 1'-0"
1
S2.3

ABC
ARCHITECTS
ASSOCIATES
123456 STREET ADDRESS
SUITE • CITY, STATE ZIP
PHONE NUMBER – FAX NUMBER
ON THE WEB

– ABLE –
STRUCTURAL
ENGINEERS
654321 ADDRESS STREET
SUITE • CITY, STATE ZIP
PHONE NUMBER – FAX NUMBER
ON THE WEB

BUILDING 3
ANYWHERE

SEAL

REVISIONS

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PROJECT # 123456
DRAWN BY: ME
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ROOF
FRAMING
PLAN

S2.3

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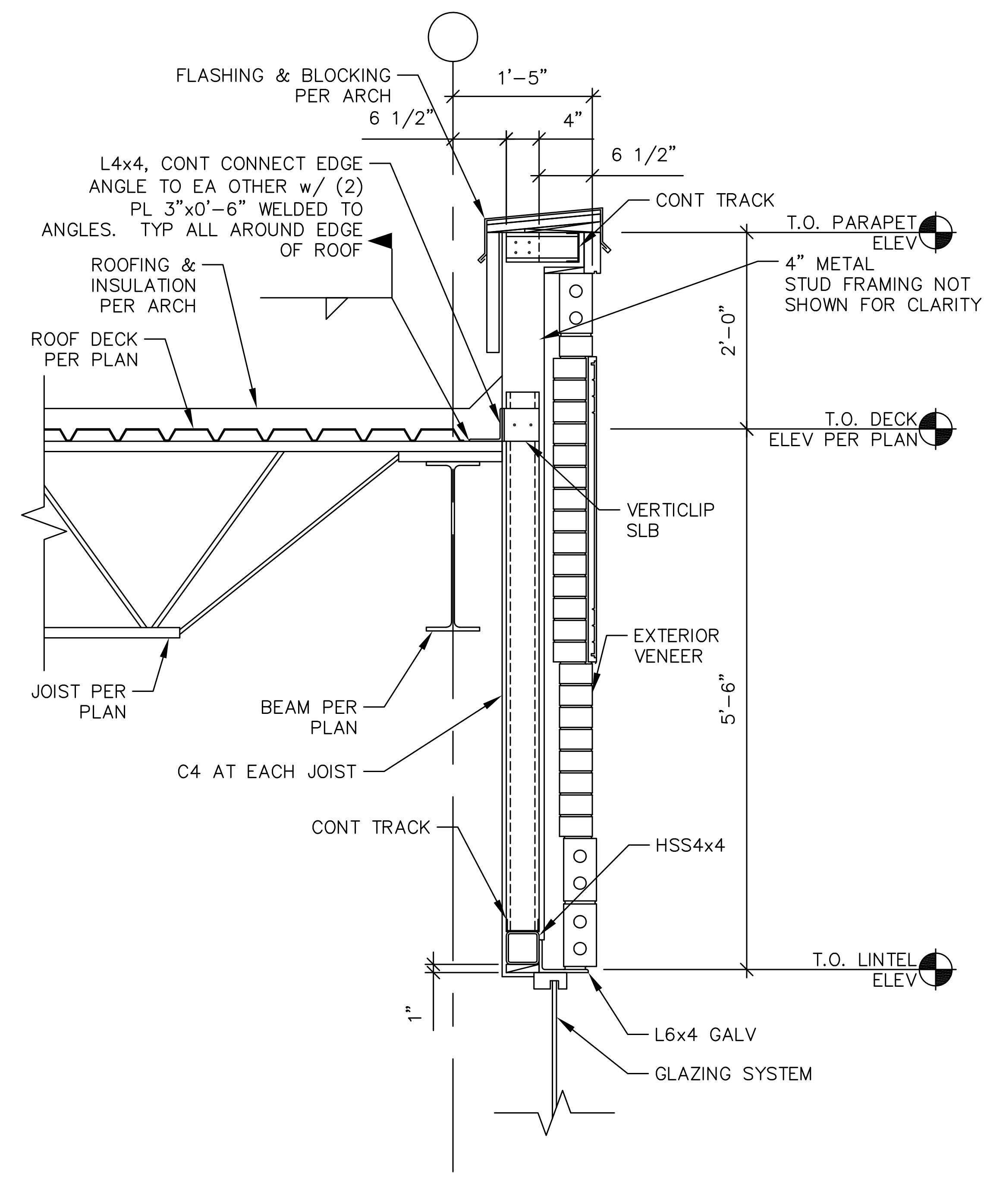
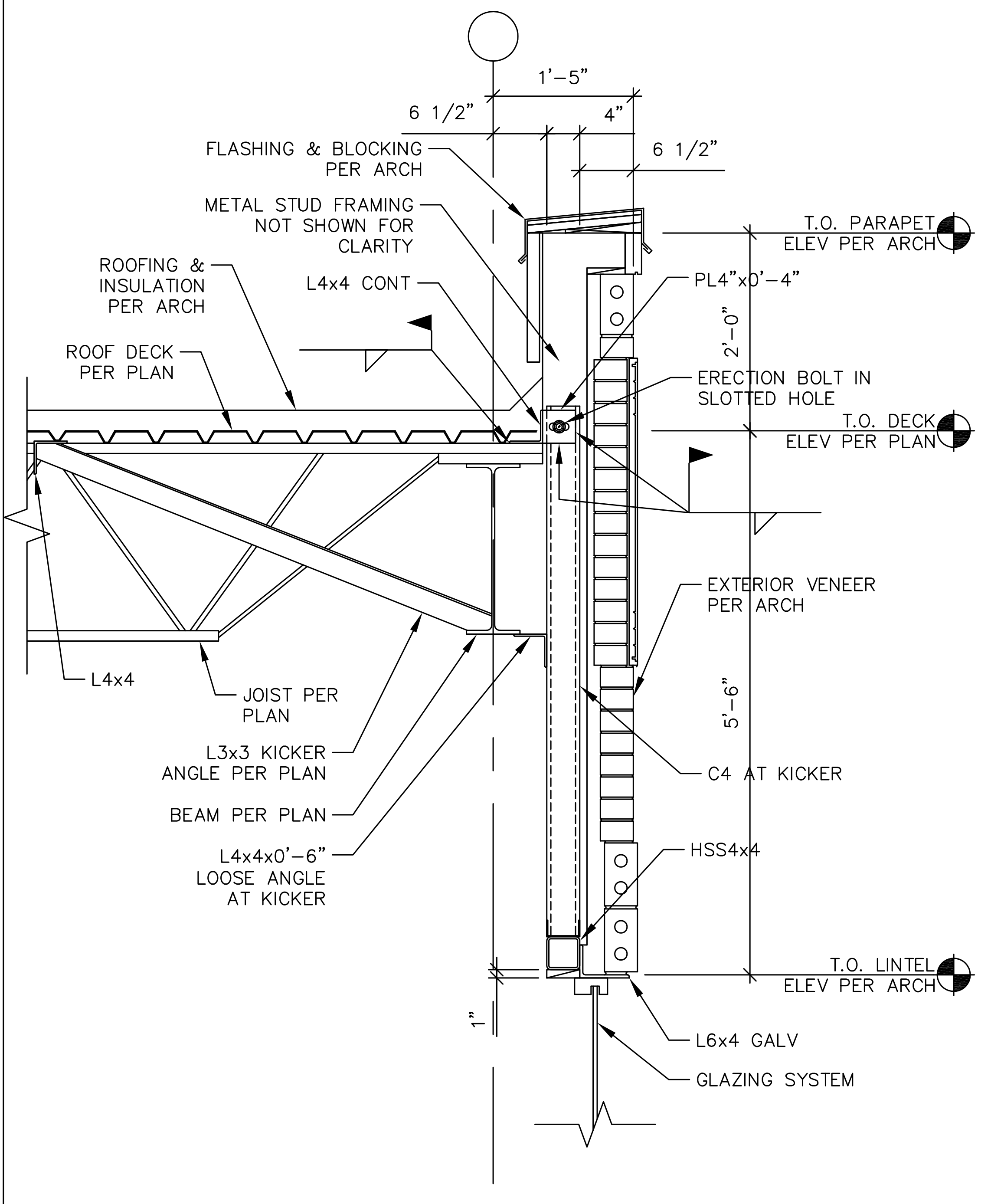


SECTION 1 S4.1
3/4" = 1'-0"



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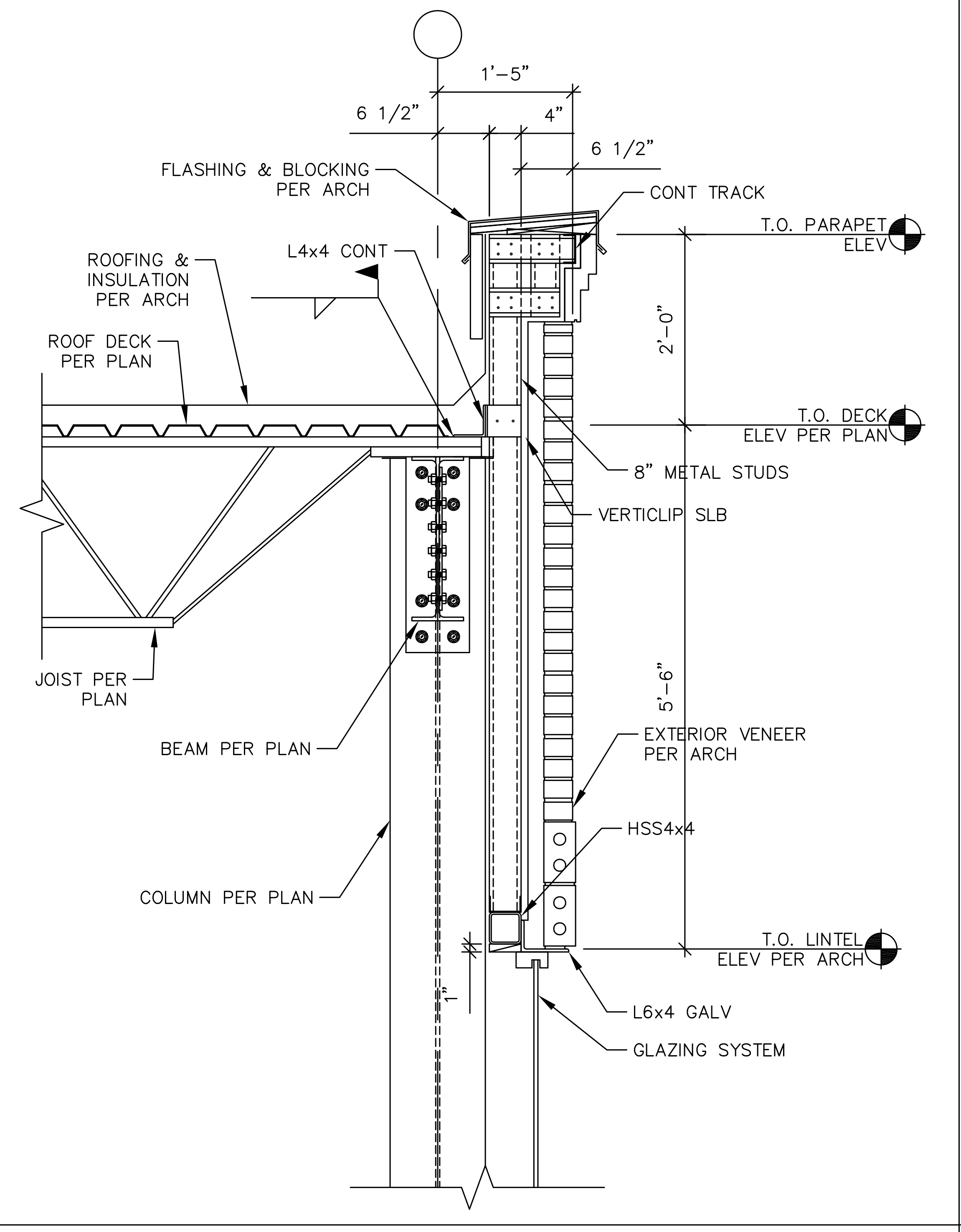
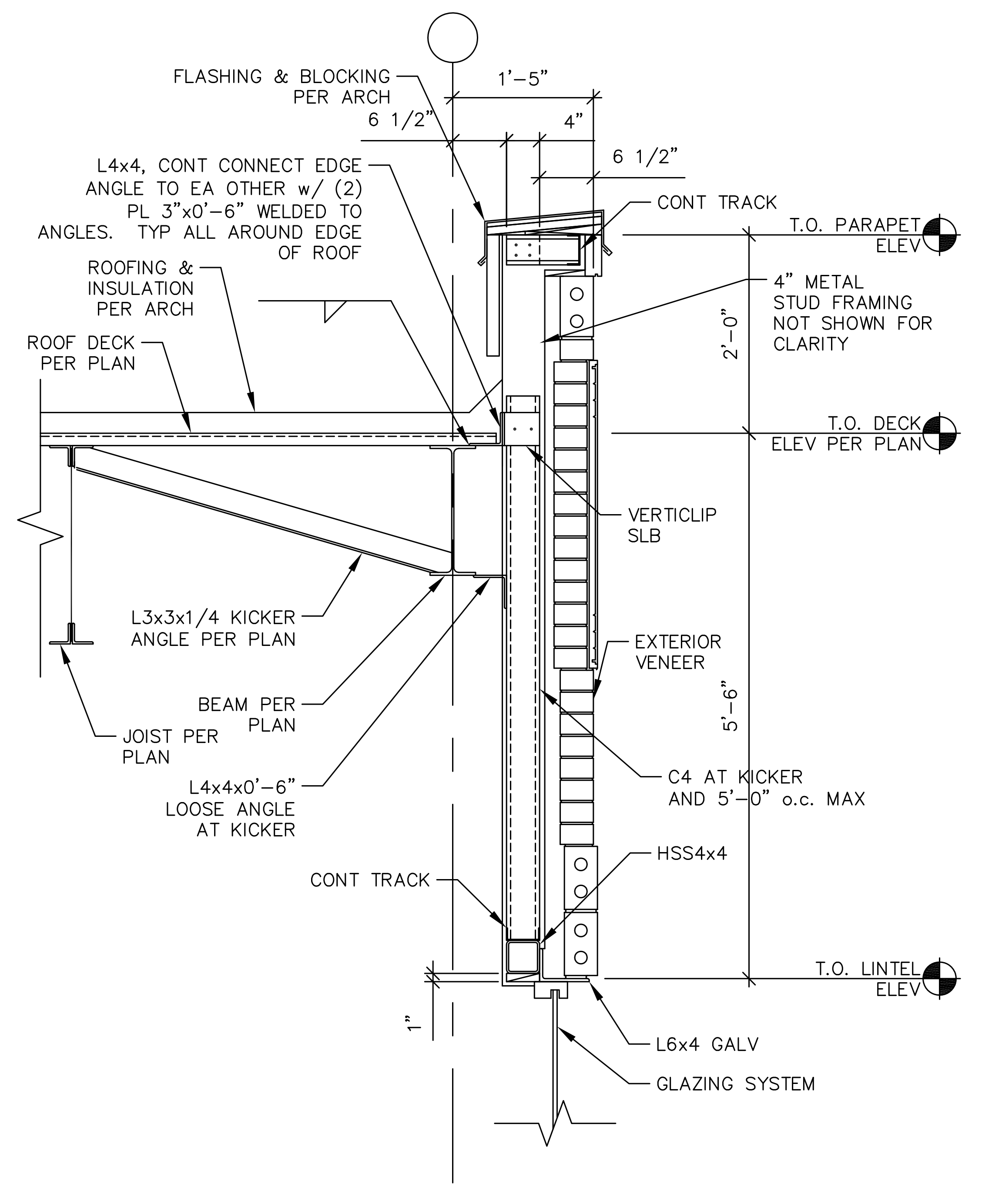


SECTION 3 S4.1
3/4" = 1'-0"



SECTION 4 S4.1
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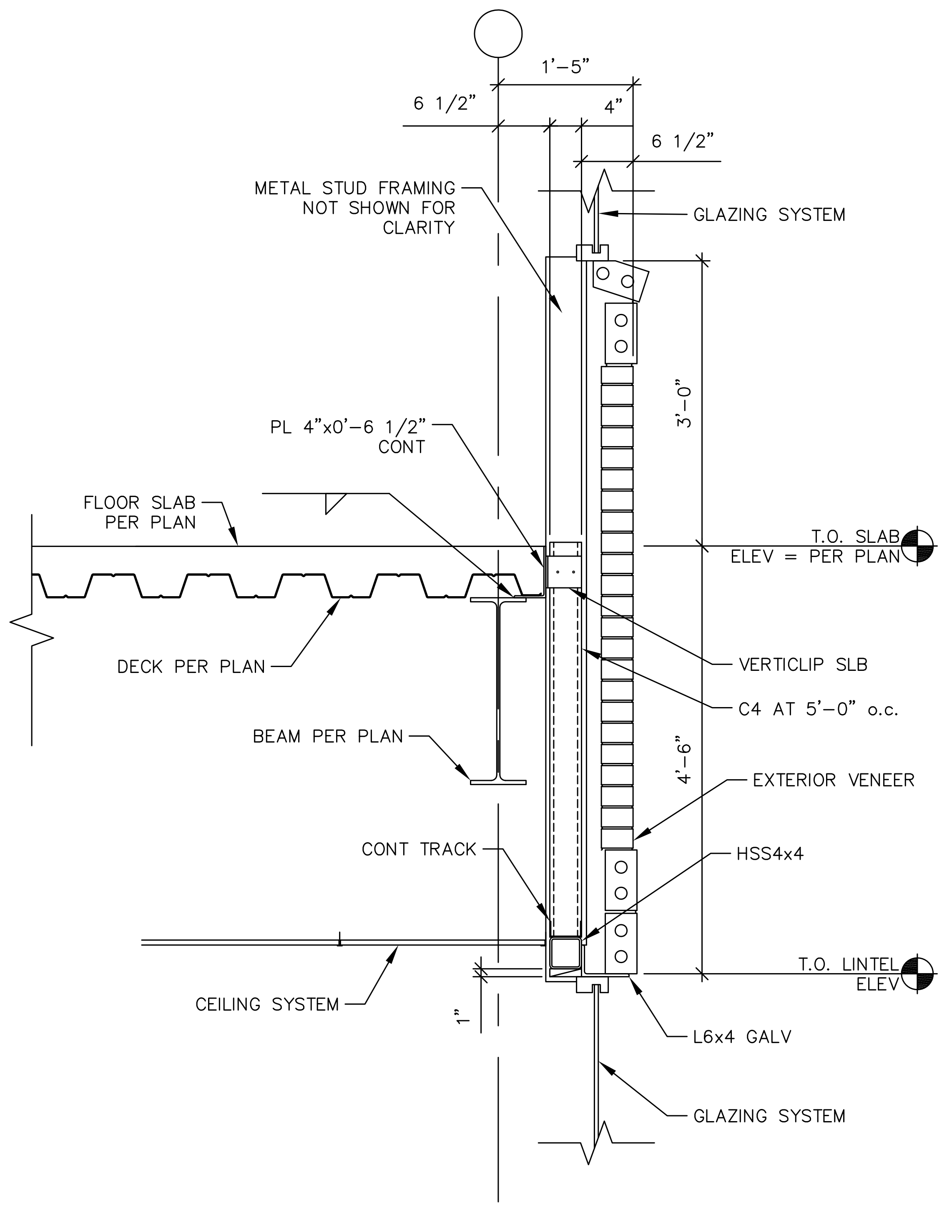

SECTION 5 S4.1
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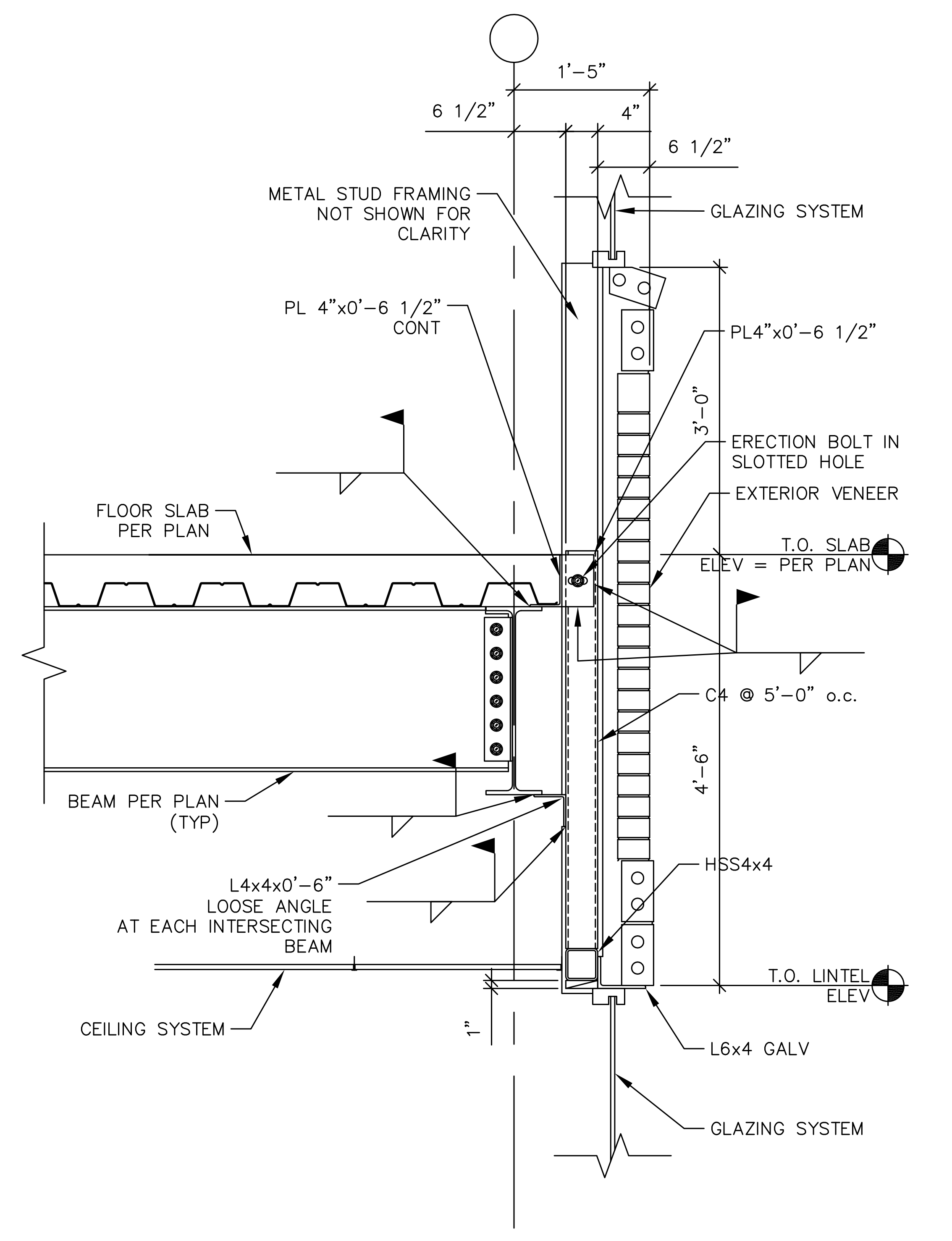
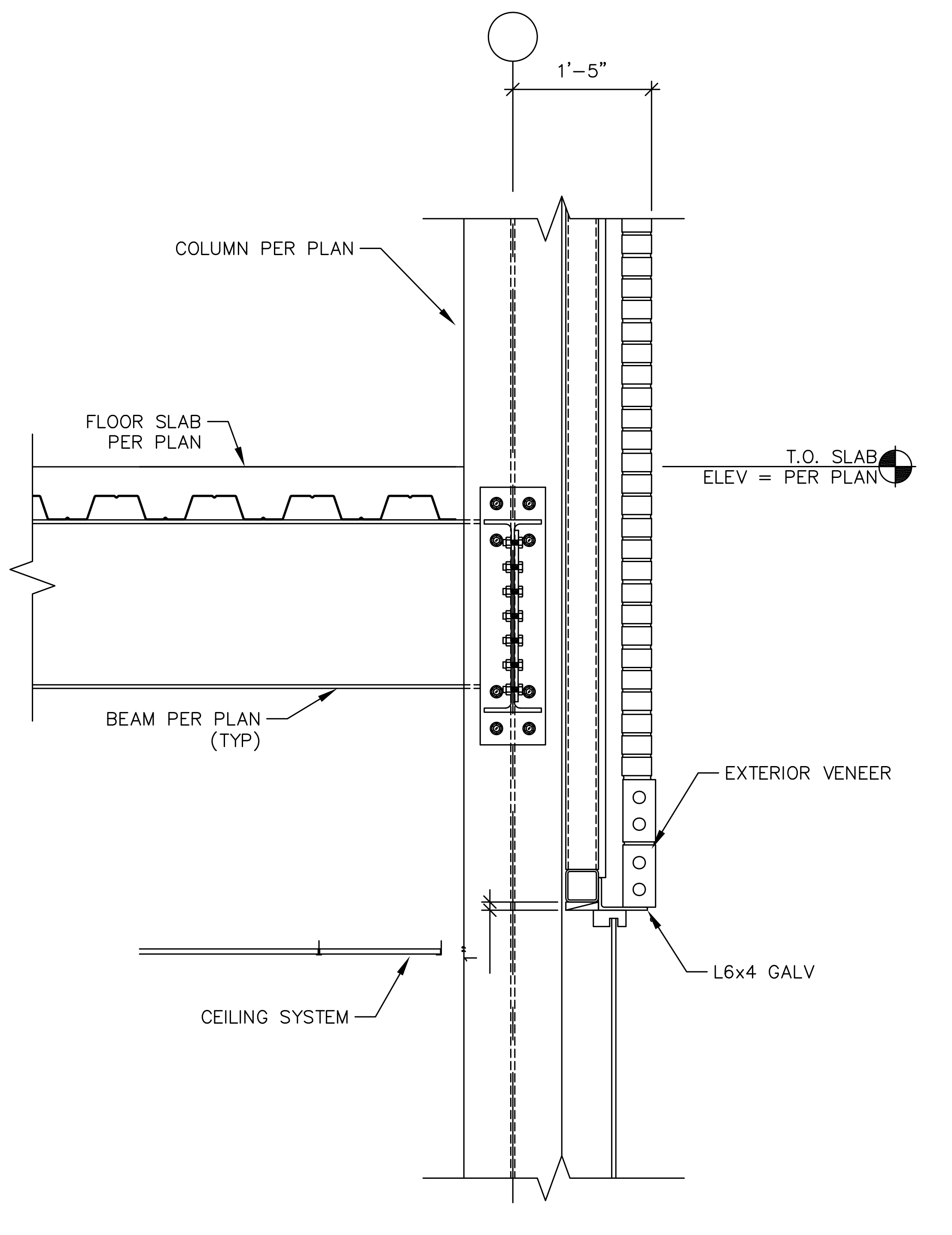
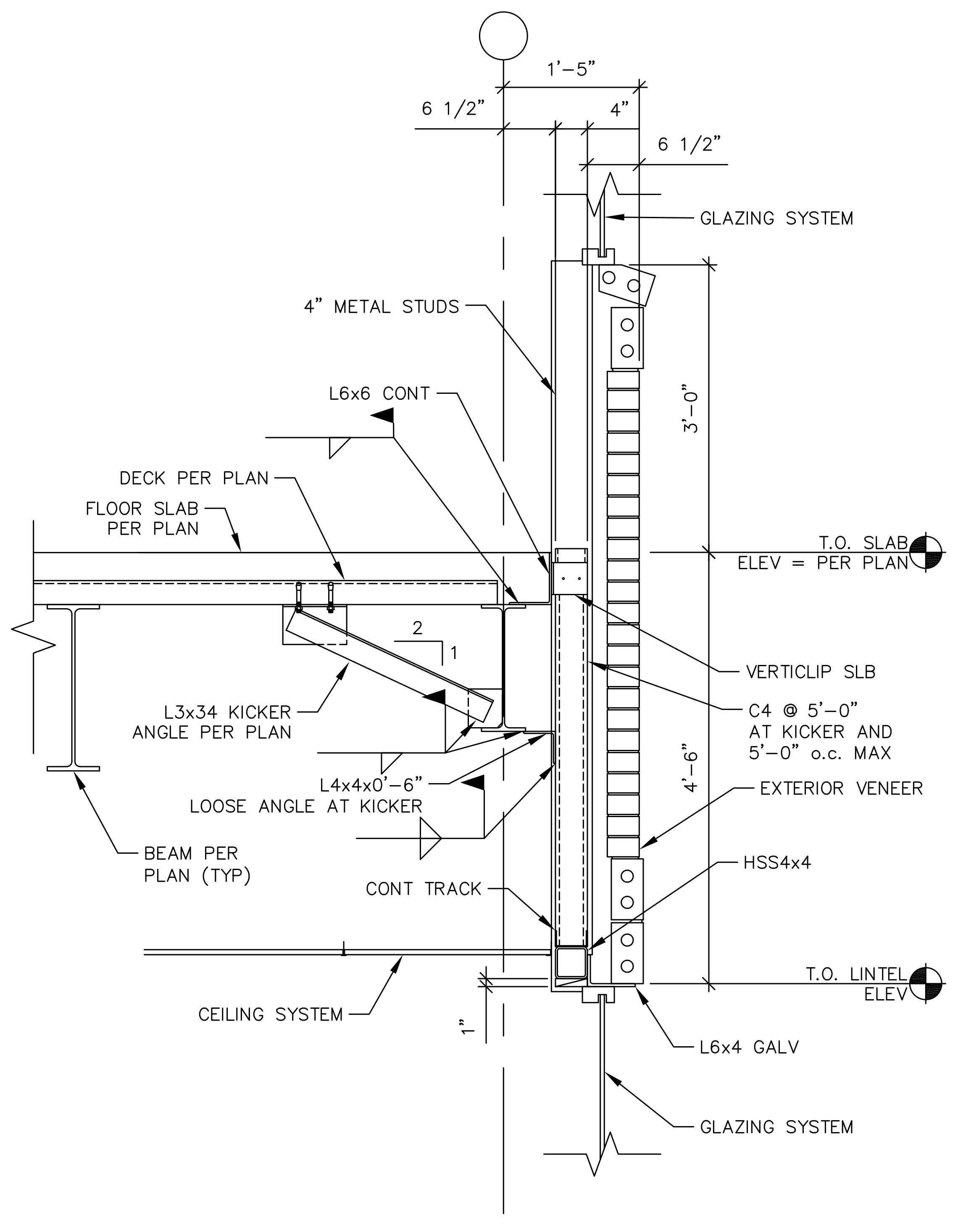


SECTION 6 S4.1
3/4" = 1'-0"



SECTION 7 S4.1
3/4" = 1'-0"



SECTION 8 S4.1
3/4" = 1'-0"

ABC
ARCHITECTS
ASSOCIATES
123456 STREET ADDRESS
SUITE • CITY, STATE ZIP
PHONE NUMBER – FAX NUMBER
ON THE WEB

— ABLE —
STRUCTURAL
ENGINEERS
654321 ADDRESS STREET
SUITE • CITY, STATE ZIP
PHONE NUMBER – FAX NUMBER
ON THE WEB

BUILDING 3
ANYWHERE

SEAL

REVISIONS

DESCRIPTION	DATE
△	
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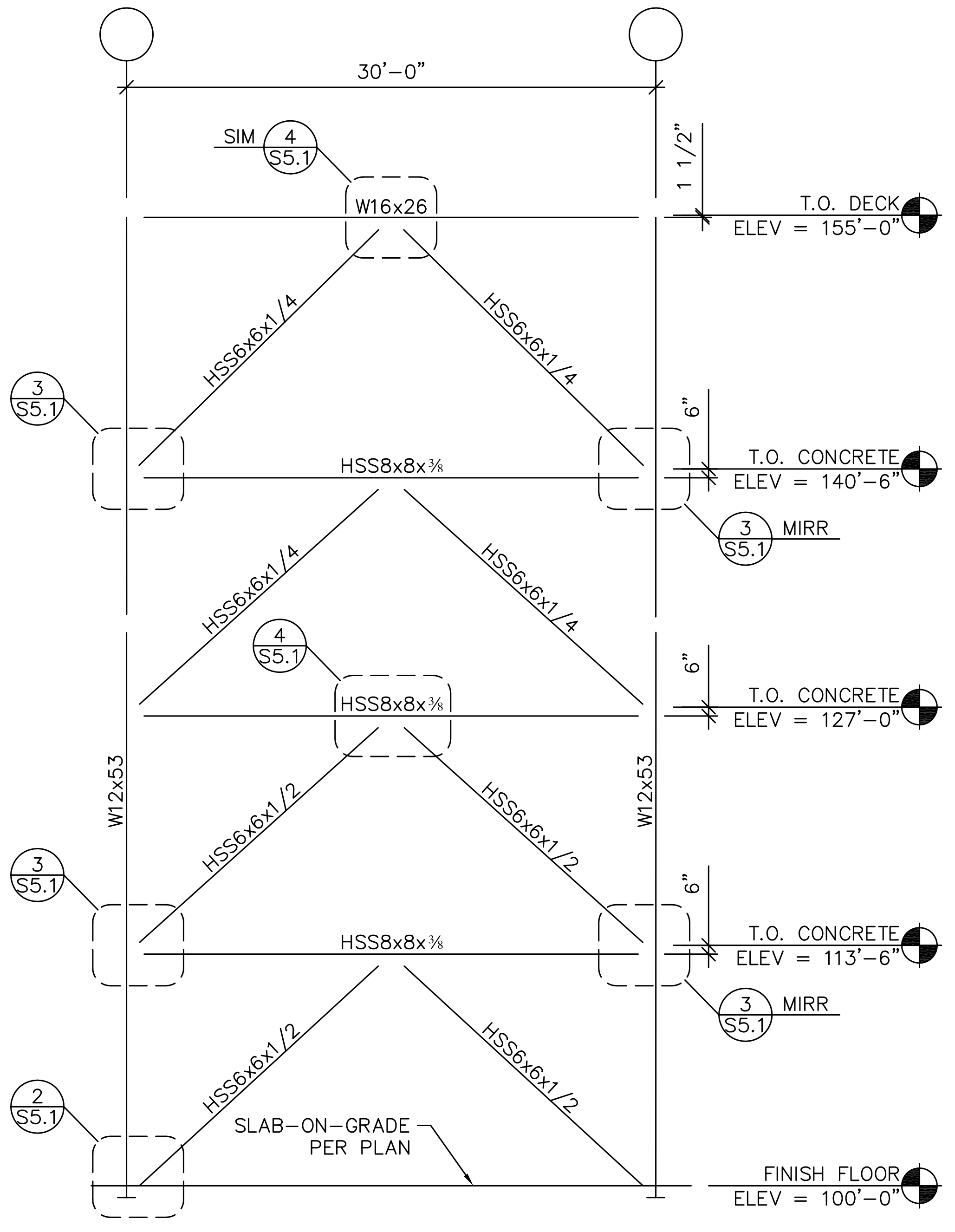
△	
△	

DATE:	PROJECT # 123456
DRAWN BY: ME	CHECKED BY: YOU
ISSUE:	

ROOF
FRAMING
SECTIONS

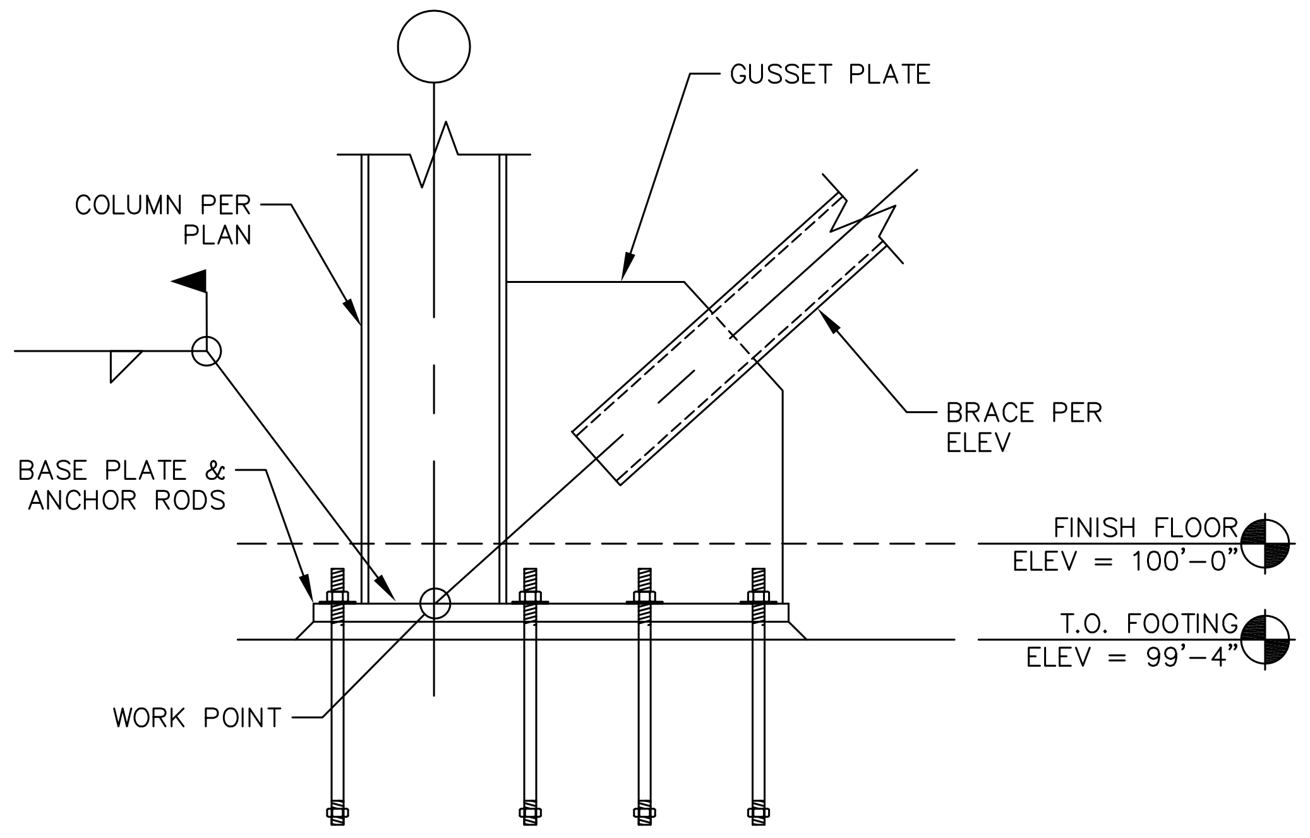
S4.1

Return to Table of



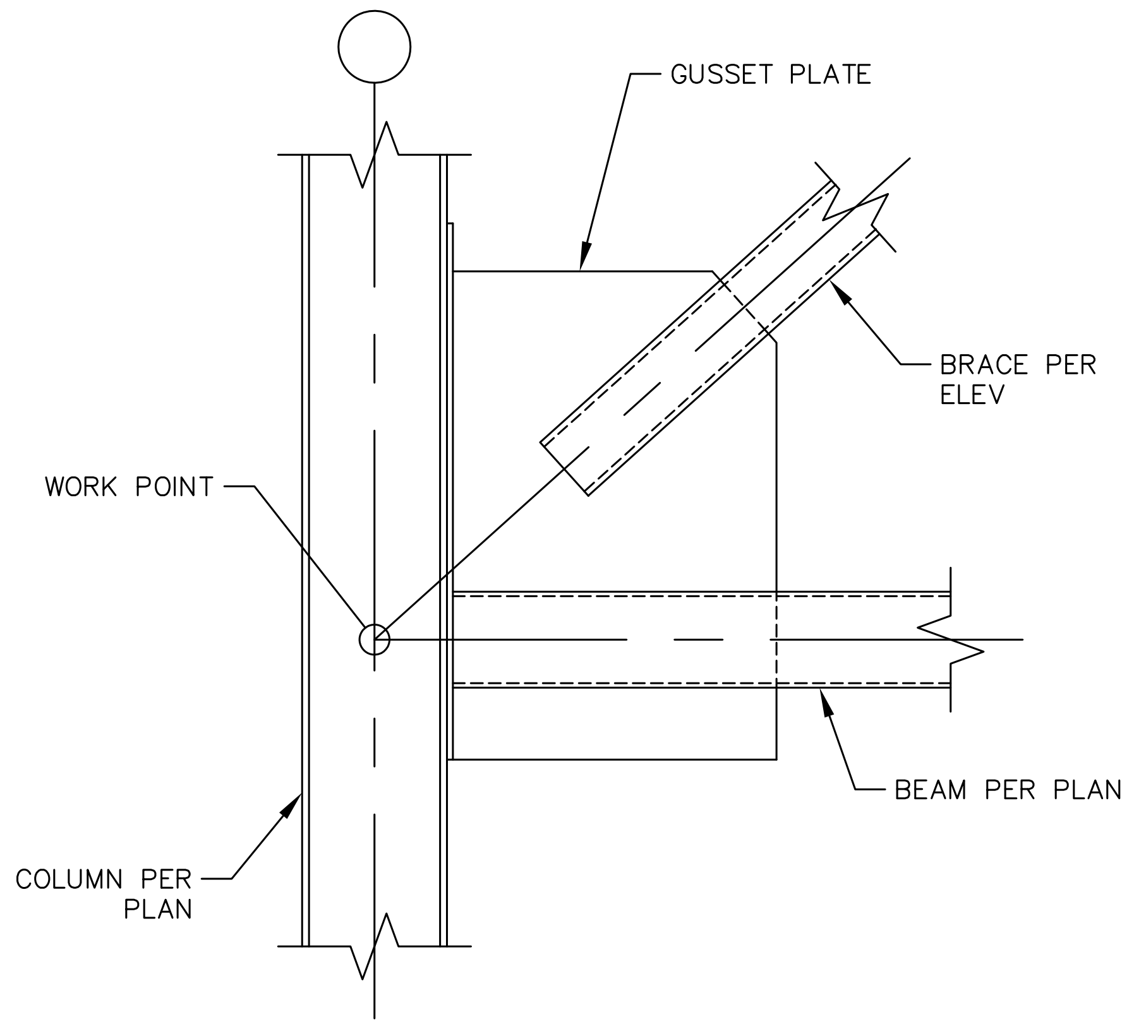
CHEVRON BRACE ELEVATION 1 S5.1

N.T.S.



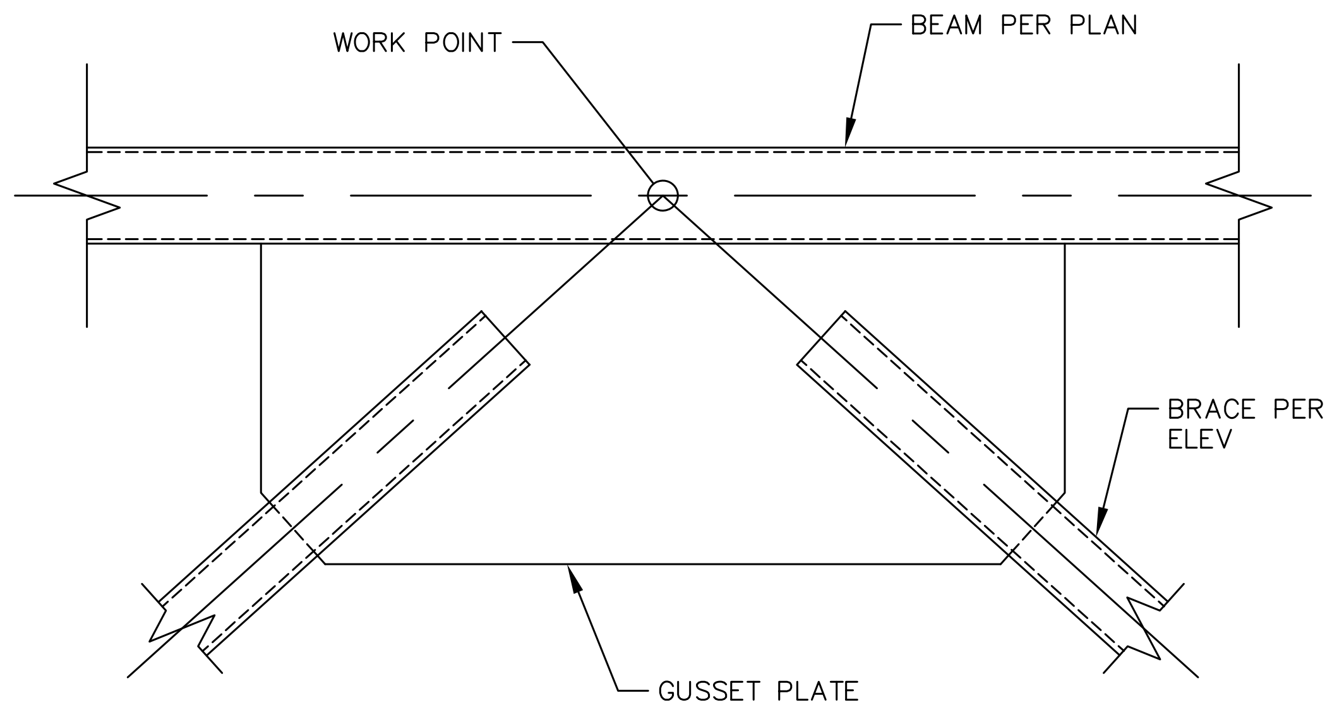
DETAIL 2 S5.1

1 1/2" = 1'–0"



DETAIL 3 S5.1

1 1/2" = 1'–0"



DETAIL 4 S5.1

1 1/2" = 1'–0"

NOT USED 5 S5.1

NOT USED 6 S5.1

NOT USED 7 S5.1

NOT USED 8 S5.1

ABC
ARCHITECTS
ASSOCIATES
123456 STREET ADDRESS
SUITE • CITY, STATE ZIP
PHONE NUMBER – FAX NUMBER
ON THE WEB

– ABLE –
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ENGINEERS
654321 ADDRESS STREET
SUITE • CITY, STATE ZIP
PHONE NUMBER – FAX NUMBER
ON THE WEB

BUILDING 3
ANYWHERE

SEAL

REVISIONS

DESCRIPTION	DATE
△	
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△	

DATE:	PROJECT # 123456
DRAWN BY: ME	CHECKED BY: YOU
ISSUE:	

CHEVRON BRACE
ELEVATION &
DETAILS

S5.1

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Part IV
Additional Resources

This part contains additional design aids that are not available in the AISC *Manual*.

DESIGN TABLE DISCUSSION

Table IV-1. Available Strength in Axial Compression—Composite Filled Rectangular HSS

Available strengths in axial compression are given for filled rectangular HSS with F_y = 50 ksi (ASTM A500 Grade C) in Tables IV-1A and IV-1B. The tables reflect HSS filled with 4-ksi and 5-ksi normal weight concrete. The tabulated values are given for the effective length with respect to the *y*-axis (L_{cy}). However, the effective length with respect to the *x*-axis (L_{cx}) must also be investigated. To determine the available strength in axial compression, the table should be entered at the larger of L_{cy} and $L_{cy\ eq}$, where

$$L_{cy\ eq} = \frac{L_{cx}}{\left(\dfrac{r_{mx}}{r_{my}}\right)} \tag{IV-1}$$

Values of the ratio $r_{mx}\,/\,r_{my}$ and other properties useful in the design of composite HSS compression members are listed at the bottom of Tables IV-1A and IV-1B. The values r_{mx} and r_{my} are the radii of gyration for the composite cross section. The ratio r_{mx}/r_{my} is determined as

$$\frac{r_{mx}}{r_{my}} = \sqrt{\frac{P_{ex}\left(L_{cx}\right)^2}{P_{ey}\left(L_{cy}\right)^2}} \tag{IV-2}$$

For compact composite sections, the values of ϕM_n and M_n/Ω were calculated using the nominal flexural strength equations for Point B of the interaction diagram in AISC *Manual* Table 6-4. For noncompact composite sections, the values of ϕM_n and M_n/Ω are calculated using the closed formed equations presented in the AISC *Specification* Commentary Figure C-I3.7.

The available strengths tabulated in Tables IV-1 through IV-4 are given for the indicated shape with the associated concrete fill. AISC *Specification* Section I2.2b stipulates that the available compressive strength of a filled composite member need not be less than that specified for the bare steel member, as required by AISC *Specification* Chapter E. In these tables, available strengths controlled by the bare steel acting alone are identified. Additionally, there is no longitudinal reinforcement provided because there is no requirement for minimum reinforcement in the AISC *Specification*. The use of filled shapes without longitudinal reinforcement is a common industry practice.

Table IV-2. Available Strength in Axial Compression—Composite Filled Square HSS

Tables IV-2A and IV-2B are the same as Tables IV-1A and IV-1B, except they provide the available strength for filled square HSS with F_y = 50 ksi (ASTM A500 Grade C) filled with 4-ksi and 5-ksi normal weight concrete.

Table IV-3. Available Strength in Axial Compression—Composite Filled Round HSS

Available strengths in axial compression are given for filled round HSS with F_y = 46 ksi (ASTM A500 Grade C) in Tables IV-3A and IV-3B. The tables reflect HSS filled with 4-ksi and 5-ksi normal weight concrete. To determine the available strength in axial compression, the table should be entered at the largest effective length, L_c. Other properties useful in the design of compression members are listed at the bottom of Tables IV-3A and IV-3B.

The values of ϕM_n and M_n/Ω were calculated using the nominal flexural strength equations for Point B of the interaction diagram in AISC *Manual* Table 6-5.

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Table IV-4. Available Strength in Axial Compression—Composite Filled Pipe

Tables IV-4A and IV-4B are the same as Tables IV-3A and IV-3B, except they provide the available strength for filled pipe with F_y = 35 ksi (ASTM A53) filled with 4-ksi and 5-ksi normal weight concrete.

Table IV-5. Combined Flexure and Axial Force—W-Shapes

W-shapes with F_y = 50 ksi (ASTM A992) and subject to combined axial force (tension or compression) and flexure may be checked for compliance with the provisions of AISC *Specification* Sections H1.1 and H1.2 using values listed in Table IV-5 and the appropriate interaction equations provided in the following sections.

Values p, b_x, b_y, t_y and t_r presented in Table IV-5 are defined in Table IV-A.

Table IV-A
Variables in Table IV-5

	LRFD	ASD
Axial Compression	$p = \dfrac{1}{\phi_c P_n}$, (kips)$^{-1}$ (IV-3a)	$p = \dfrac{\Omega_c}{P_n}$, (kips)$^{-1}$ (IV-3b)
Major-Axis Bending	$b_x = \dfrac{8}{9\phi_b M_{nx}}$, (kip-ft)$^{-1}$ (IV-4a)	$b_x = \dfrac{8\Omega_b}{9M_{nx}}$, (kip-ft)$^{-1}$ (IV-4b)
Minor-Axis Bending	$b_y = \dfrac{8}{9\phi_b M_{ny}}$, (kip-ft)$^{-1}$ (IV-5a)	$b_y = \dfrac{8\Omega_b}{9M_{ny}}$, (kip-ft)$^{-1}$ (IV-5b)
Tension Yielding	$t_y = \dfrac{1}{\phi_t F_y A_g}$, (kips)$^{-1}$ (IV-6a)	$t_y = \dfrac{\Omega_t}{F_y A_g}$, (kips)$^{-1}$ (IV-6b)
Tension Rupture	$t_r = \dfrac{1}{\phi_t F_u \left(0.75A_g\right)}$, (kips)$^{-1}$ (IV-7a)	$t_r = \dfrac{\Omega_t}{F_u \left(0.75A_g\right)}$, (kips)$^{-1}$ (IV-7b)

Values of p, b_x and b_y already account for section compactness and can be used directly. Given that the limit state of lateral-torsional buckling does not apply to W-shapes bent about their minor axis, values of b_y are independent of unbraced length and C_b. Values of b_x equally apply to combined flexure and compression, as well as combined flexure and tension. Smaller values of variable p for a given L_c and smaller values of b_x for a given L_b indicate higher strength for the type of load in question. For example, a section with a smaller p at a certain L_c is more effective in carrying axial compression than another section with a larger value of p at the same L_c. Similarly, a section with a smaller b_x is more effective for flexure at a given L_b than another section with a larger b_x for the same L_b. This information may be used to select more efficient shapes when relatively large amounts of axial load or bending are present.

The tabulated values of b_x assume that C_b = 1.0. These values may be modified in accordance with AISC *Specification* Sections F1 and H1.2. The following procedure may be used to account for C_b >1.0.

$$b_{x(C_b>1.0)} = \frac{b_{x(C_b=1.0)}}{C_b} \geq b_{x\,min} \tag{IV-8}$$

Combined Flexure and Compression

Equations H1-1a and H1-1b of the AISC *Specification* may be written as follows using the coefficients listed in Table IV-5 and defined in Table IV-A.

When $pP_r \geq 0.2$:

$$pP_r + b_x M_{rx} + b_y M_{ry} \leq 1.0 \tag{IV-9}$$

When $pP_r < 0.2$:

$$\frac{pP_r}{2}+\frac{9}{8}\left(b_x M_{rx}+b_y M_{ry}\right)\le 1.0 \tag{IV-10}$$

The designer may check acceptability of a given shape using the appropriate interaction Equation IV-9 or IV-10. See Aminmansour (2000) for more information on this method, including an alternative approach for selection of a trial shape.

Combined Flexure and Tension

Equations H1-1a and H1-1b of the AISC *Specification* may be written as follows using the coefficients listed in Table IV-5 and defined in Table IV-A.

When $pP_r \ge 0.2$:

$$\left(t_y \text{ or } t_r\right)P_r + b_x M_{rx} + b_y M_{ry} \le 1.0 \tag{IV-11}$$

When $pP_r < 0.2$:

$$\frac{\left(t_y \text{ or } t_r\right)P_r}{2}+\frac{9}{8}\left(b_x M_{rx}+b_y M_{ry}\right)\le 1.0 \tag{IV-12}$$

The larger value of t_y and t_r should be used in the above equations.

The designer may check acceptability of a given shape using the approximate interaction Equation IV-11 or IV-12 along with variables t_r, t_y, b_x and b_y. See Aminmansour (2006) for more information on this method.

It is noted that the values for t_r listed in Table IV-5 are based on the assumption that $A_e = 0.75A_g$. See Part 5 of the AISC *Manual* for more information on this assumption. When $A_e > 0.75A_g$, the tabulated values for t_r are conservative. When $A_e < 0.75A_g$, t_r must be calculated based upon the actual value of A_e.

Values of $b_{x\,min}$ are listed in Table IV-5 at $L_b = 0$. See Aminmansour (2009) for more information on this method. Values for p, b_x, b_y, t_y and t_r presented in Table IV-5 have been multiplied by 10^3. Thus, when used in the appropriate interaction equation they must be multiplied by 10^{-3} (0.001).

Table IV-6. Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces—W-Shapes

Tables IV-6A and IV-6B are the same as AISC *Manual* Table 6-2, except they provide the available strength for F_y = 65 ksi (ASTM A913 Grade 65) and F_y = 70 ksi (ASTM A913 Grade 70). Discussion on the use of these tables can be found in Part 6 of the AISC *Manual*.

Table IV-7. Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces—Rectangular HSS

The available strengths of rectangular HSS are given in Table IV-7A for F_y = 50 ksi (ASTM A1085 Grade A) and in Table IV-7B for F_y = 50 ksi (ASTM A500 Grade C). These tables may be used to design members with only compression, tension, flexure and shear forces or may be used to design members subject to combined effects. All the information presented here in the following is presented in Parts 3, 4 and 5 of the AISC *Manual*, but has been grouped here for ease of use.

HSS Subject to Flexure

The available flexural strengths of rectangular HSS bent about their major (X-X) and minor (Y-Y) principal axis are given in the lower portion of Tables IV-7A and IV-7B.

The available strength for bending about the major and minor axes is a single value based on the limit states of yielding or flange local buckling. The limit state of lateral-torsional buckling is not included and must be checked for bending in the major axis. Lateral-torsional buckling does not apply to bending of rectangular HSS about their minor axis.

HSS Subject to Shear

The available shear strengths of rectangular HSS for both the major (X-X) and minor (Y-Y) principal axis are given in the lower portion of Tables IV-7A and IV-7B.

HSS Subject to Compression

The available strengths in axial compression are tabulated for the effective length with respect to the minor axis, L_{cy}. However, the effective length with respect to the major axis, L_{cx}, must also be investigated. To determine the available strength in axial compression the table should be entered at the larger of L_{cy} and $L_{cy\ eq}$, where

$$L_{cy\ eq} = \frac{L_{cy}}{\dfrac{r_x}{r_y}} \qquad \text{(\textit{Manual} Eq. 4-1)}$$

Values for the ratio r_x / r_y and other properties useful in the design of rectangular HSS compression members are listed at the bottom of Tables IV-7A and IV-7B.

HSS Subject to Tension

The available tensile strengths of rectangular HSS are given in the lower portion of Tables IV-7A and IV-7B for the limit states of tensile yielding and tensile rupture.

Strengths given for the limit state of tensile rupture are based on the assumption that $A_e = 0.75A_g$.

HSS Subject to Combined Forces

AISC *Specification* Equation H1-1a or Equation H1-1b governs the design of HSS subject to combined axial force and flexure. The values of the available strength in tension, compression or flexure obtained from Table IV-7A or Table IV-7B may be used to check interaction through these equations or the equations given in AISC *Specification* Section H1.3.

Table IV-8. Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces—Square HSS

Tables IV-8A and IV-8B are the same as Tables IV-7A and IV-7B, except they provide the available strength for square HSS with F_y = 50 ksi and F_u = 65 (ASTM A1085 Grade A) and F_y = 50 ksi and F_u = 62 (ASTM A500 Grade C).

The limit state of lateral-torsional buckling does not apply for a square HSS bending in either the major or minor axis.

Table IV-9. Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces—Round HSS

The available strengths of round HSS are given in Table IV-9A for F_y = 50 ksi (ASTM A1085 Grade A) and Table IV-9B for F_y = 46 ksi (ASTM A500 Grade C). These tables are similar to Tables IV-7A and IV-7B, except the available flexural strength is determined from AISC *Specification* Section F8 and the available strength in axial compression is determined by entering the top of the table with the effective length, L_c.

Table IV-10. Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces—Pipe

Table IV-10 is similar to Tables IV-9A and IV-9B, except it provides the available strengths for pips with F_y = 35 ksi (ASTM A53 Grade B).

Table IV-11. Plastic Section Modulus for Coped W-Shapes

Values are given for the gross and net plastic section modulus for coped W-shapes, as illustrated in the table header.

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PART IV REFERENCES

Aminmansour, A. (2000), “A New Approach for Design of Steel Beam-Columns,” *Engineering Journal*, AISC, Vol. 37, No. 2, pp. 41–72.

Aminmansour, A. (2006), “New Method of Design for Combined Tension and Bending,” *Engineering Journal*, AISC, Vol. 43, No. 4, pp. 247–256.

Aminmansour, A. (2009), “Optimum Flexural Design of Steel Members Utilizing Moment Gradient and C_b,” *Engineering Journal*, AISC, Vol. 46, No. 1, pp. 47–55.

4

COMPOSITE
HSS20–HSS16

Table IV-1A

Available Strength in Axial Compression, kips

Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS20x12x								HSS16x12x			
		⅝		½		⅜		5⁄16		⅝		½	
t_{des}, in,		**0.581**		**0.465**		**0.349**		**0.291**		**0.581**		**0.465**	
Steel, lb/ft		**127**		**103**		**78.5**		**65.9**		**110**		**89.7**	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	1220	1830	1070	1600	908	1360	803	1200	1030	1550	898	1350
	1	1220	1830	1070	1600	908	1360	803	1200	1030	1540	898	1350
	2	1220	1830	1060	1600	906	1360	802	1200	1030	1540	896	1340
	3	1220	1820	1060	1590	904	1360	799	1200	1030	1540	894	1340
	4	1210	1820	1060	1590	901	1350	797	1190	1020	1530	891	1340
	5	1210	1810	1050	1580	897	1340	793	1190	1020	1530	887	1330
	6	1200	1800	1050	1570	892	1340	788	1180	1010	1520	882	1320
	7	1190	1790	1040	1560	886	1330	783	1170	1010	1510	876	1310
	8	1180	1780	1030	1550	879	1320	777	1170	998	1500	870	1300
	9	1170	1760	1020	1540	871	1310	771	1160	990	1480	862	1290
	10	1160	1750	1010	1520	863	1290	763	1140	980	1470	854	1280
	11	1150	1730	1000	1510	854	1280	755	1130	970	1460	845	1270
	12	1140	1710	993	1490	844	1270	746	1120	959	1440	836	1250
	13	1130	1690	981	1470	833	1250	737	1100	947	1420	825	1240
	14	1110	1670	968	1450	822	1230	726	1090	934	1400	814	1220
	15	1100	1640	954	1430	809	1210	716	1070	921	1380	802	1200
	16	1080	1620	940	1410	797	1200	704	1060	907	1360	790	1180
	17	1060	1590	925	1390	783	1180	693	1040	892	1340	777	1170
	18	1040	1570	909	1360	769	1150	680	1020	877	1310	763	1140
	19	1030	1540	893	1340	755	1130	668	1000	861	1290	749	1120
	20	1010	1510	876	1310	740	1110	654	981	844	1270	735	1100
	21	987	1480	858	1290	725	1090	641	961	827	1240	720	1080
	22	967	1450	840	1260	709	1060	627	940	809	1210	704	1060
	23	946	1420	822	1230	693	1040	612	919	791	1190	688	1030
	24	925	1390	803	1200	676	1010	598	897	773	1160	672	1010
	25	903	1350	784	1180	660	990	583	875	754	1130	656	984
	26	881	1320	765	1150	643	964	568	852	735	1100	639	959
	27	859	1290	745	1120	626	938	553	829	716	1070	622	934
	28	836	1250	725	1090	608	912	537	806	697	1050	605	908
	29	813	1220	705	1060	591	886	522	783	677	1020	588	883
	30	791	1190	685	1030	573	860	506	760	658	986	571	857
	32	745	1120	644	967	538	807	475	713	618	927	537	805
	34	699	1050	604	906	503	754	444	666	579	868	502	753
	36	653	979	564	846	468	702	413	620	540	810	468	702
	38	608	912	524	786	434	651	383	575	501	752	434	652
	40	564	845	486	728	401	601	354	531	464	696	402	602
Properties													
M_{nx}/Ω_b / $\phi_b M_{nx}$	**kip-ft**	636	956	530	796	417	627	359	540	450	677	375	563
M_{ny}/Ω_b / $\phi_b M_{ny}$	**kip-ft**	434	653	359	539	281	423	232	348	363	545	302	454
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		72200		62100		51100		45100		40300		34900	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		30500		26100		21400		18900		24900		21500	
r_{mx}/r_{my}		1.54		1.54		1.55		1.54		1.27		1.27	
r_{my}, in.		4.93		4.99		5.04		5.07		4.80		4.86	

ASD	**LRFD**
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

4

COMPOSITE
HSS16

Table IV-1A (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS16x12x				HSS16x8x							
		3/8		5/16		5/8		1/2		3/8		5/16	
t_{des}, in.		0.349		0.291		0.581		0.465		0.349		0.291	
Steel, lb/ft		68.3		57.4		93.3		76.1		58.1		48.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	761	1140	692	1040	814	1220	703	1050	590	885	529	794
	1	761	1140	691	1040	814	1220	703	1050	589	884	529	793
	2	760	1140	690	1040	811	1220	700	1050	587	881	527	790
	3	758	1140	688	1030	807	1210	697	1040	584	876	524	786
	4	755	1130	686	1030	800	1200	691	1040	580	870	520	780
	5	751	1130	682	1020	793	1190	685	1030	574	861	515	772
	6	747	1120	678	1020	783	1170	677	1020	567	851	509	763
	7	742	1110	674	1010	772	1160	668	1000	559	839	502	752
	8	736	1100	668	1000	760	1140	657	985	550	826	493	740
	9	730	1090	662	993	746	1120	645	968	540	811	484	726
	10	723	1080	656	983	731	1100	632	948	529	794	474	711
	11	715	1070	648	972	714	1070	618	927	518	776	464	695
	12	706	1060	640	960	697	1050	603	905	505	757	452	678
	13	697	1050	632	948	678	1020	587	881	491	737	440	660
	14	688	1030	623	934	659	988	571	856	477	716	427	640
	15	677	1020	613	920	638	957	553	830	463	694	414	620
	16	666	1000	603	905	617	926	535	803	447	671	400	600
	17	655	983	592	889	595	893	517	775	432	648	385	578
	18	643	965	581	872	573	860	498	747	416	624	371	556
	19	631	946	570	855	551	826	479	718	400	599	356	534
	20	618	927	558	837	528	792	459	689	383	575	341	512
	21	605	908	546	819	505	758	440	659	367	550	326	489
	22	592	888	534	801	482	723	420	630	350	525	311	467
	23	578	867	521	782	459	689	400	600	333	500	296	444
	24	564	846	508	762	436	656	381	571	317	475	281	422
	25	550	825	495	742	416	625	361	542	301	451	267	400
	26	536	803	482	722	395	594	342	513	284	427	252	378
	27	521	782	468	702	375	564	323	485	269	403	238	357
	28	506	760	455	682	356	534	305	457	253	380	224	336
	29	492	737	441	661	336	505	287	430	238	357	210	316
	30	477	715	427	641	317	477	269	404	223	334	197	295
	32	447	670	400	600	280	421	236	355	196	294	173	259
	34	417	626	372	559	248	373	209	314	174	260	153	230
	36	388	582	346	518	221	333	187	280	155	232	137	205
	38	359	539	319	479	199	299	168	252	139	208	123	184
	40	331	497	294	440	179	269	151	227	125	188	111	166
Properties													
M_{nx}/Ω_b | $\phi_b M_{nx}$	kip-ft	296	444	253	381	348	524	292	438	232	348	199	299
M_{ny}/Ω_b | $\phi_b M_{ny}$	kip-ft	237	356	203	304	208	312	174	261	137	205	117	175
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		28700		25400		29100		25700		21400		18900	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		17600		15600		9060		7950		6590		5820	
r_{mx}/r_{my}		1.28		1.28		1.79		1.80		1.80		1.80	
r_{my}, in.		4.91		4.94		3.27		3.32		3.37		3.40	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

COMPOSITE
HSS16–HSS14

Table IV-1A (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS16x8x		HSS14x10x									
		¼		⅝		½		⅜		5⁄16		¼	
t_{des}, in.		0.233		0.581		0.465		0.349		0.291		0.233	
Steel, lb/ft		39.4		93.3		76.1		58.1		48.9		39.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	455	682	835	1250	724	1090	610	915	550	825	488	733
	1	454	682	834	1250	723	1080	610	915	549	824	488	732
	2	453	679	832	1250	722	1080	608	913	548	822	487	730
	3	450	676	829	1240	719	1080	606	909	546	819	485	728
	4	447	670	825	1240	716	1070	603	905	543	815	482	724
	5	442	664	820	1230	711	1070	599	899	540	809	479	719
	6	437	656	813	1220	705	1060	594	892	535	803	475	713
	7	431	646	806	1210	699	1050	589	883	530	795	470	705
	8	424	636	797	1200	692	1040	582	873	524	786	465	697
	9	416	624	787	1180	683	1020	575	863	517	776	459	688
	10	407	611	777	1170	674	1010	567	851	510	765	452	678
	11	398	597	765	1150	664	996	559	838	502	753	445	667
	12	388	582	752	1130	653	980	549	824	494	740	437	655
	13	378	566	739	1110	642	963	539	809	484	727	429	643
	14	366	550	725	1090	630	944	529	793	475	712	420	630
	15	355	532	710	1060	617	925	518	777	465	697	410	616
	16	343	515	694	1040	603	905	506	759	454	681	401	601
	17	331	496	678	1020	589	884	494	741	443	664	391	586
	18	318	477	661	991	575	862	482	722	432	647	380	570
	19	306	458	643	965	560	840	469	703	420	630	370	554
	20	293	439	625	938	545	817	456	684	408	612	359	538
	21	280	420	607	911	529	793	442	663	395	593	347	521
	22	267	400	588	883	513	769	429	643	383	574	336	504
	23	254	381	570	854	497	745	415	622	370	555	325	487
	24	241	362	551	826	481	721	401	601	357	536	313	469
	25	228	343	531	797	464	696	387	580	345	517	301	452
	26	216	324	512	768	448	671	373	559	332	498	290	434
	27	204	306	493	739	431	646	358	538	319	478	278	417
	28	192	288	474	711	414	622	344	516	306	459	266	400
	29	180	270	455	682	398	597	330	495	293	440	255	382
	30	168	253	436	653	382	572	316	475	281	421	244	365
	32	148	222	398	597	349	524	289	433	256	384	221	332
	34	131	197	362	543	318	477	262	394	232	348	200	300
	36	117	175	327	491	288	432	237	355	208	313	179	268
	38	105	157	294	442	259	388	213	319	187	281	161	241
	40	94.7	142	266	399	234	350	192	288	169	253	145	217
Properties													
M_{nx}/Ω_b | $\phi_b M_{nx}$	kip-ft	166	249	324	487	271	408	214	322	184	277	153	229
M_{ny}/Ω_b | $\phi_b M_{ny}$	kip-ft	93.6	141	253	380	211	318	166	250	142	214	116	175
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		16200		24500		21600		17800		15700		13500	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		4980		13900		12300		10100		8870		7620	
r_{mx}/r_{my}		1.80		1.33		1.33		1.33		1.33		1.33	
r_{my}, in.		3.42		3.98		4.04		4.09		4.12		4.14	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

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4
COMPOSITE
HSS12

Table IV-1A (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS12x10x								HSS12x8x			
		½		⅜		5⁄16		¼		⅝		½	
t_{des}, in.		0.465		0.349		0.291		0.233		0.581		0.465	
Steel, lb/ft		69.3		53.0		44.6		36.0		76.3		62.5	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	645	968	544	815	488	732	434	652	650	976	563	844
	1	645	967	543	815	487	731	434	651	650	975	562	843
	2	643	965	542	813	486	729	433	650	648	971	560	840
	3	641	962	540	810	484	727	431	647	644	966	557	836
	4	638	957	537	806	482	723	429	643	639	958	553	829
	5	634	951	534	800	479	718	426	639	632	948	547	821
	6	629	943	529	794	475	712	422	633	624	937	541	811
	7	623	934	524	786	470	705	418	627	615	923	533	799
	8	616	924	518	777	465	697	413	620	605	907	524	786
	9	608	912	512	768	459	688	408	612	593	890	514	771
	10	600	900	505	757	452	678	402	603	581	871	504	755
	11	591	886	497	745	445	668	395	593	567	850	492	738
	12	581	871	488	733	437	656	388	582	552	828	479	719
	13	570	856	479	719	429	644	381	571	537	805	466	699
	14	559	839	470	705	421	631	373	559	521	781	453	679
	15	548	821	460	690	412	617	364	546	504	755	438	657
	16	535	803	449	674	402	603	356	533	486	729	423	635
	17	523	784	439	658	392	588	346	520	468	702	408	612
	18	509	764	427	641	382	573	337	506	450	675	392	589
	19	496	744	416	624	371	557	328	491	432	647	377	565
	20	482	723	404	606	361	541	318	477	413	620	361	541
	21	468	702	392	588	350	524	308	461	395	594	345	517
	22	453	680	379	569	338	508	297	446	377	567	328	493
	23	439	658	367	550	327	491	287	431	360	541	312	469
	24	424	636	354	532	316	474	277	415	343	515	297	445
	25	409	613	342	513	304	456	266	400	325	489	281	421
	26	394	591	329	494	293	439	256	384	308	463	265	398
	27	379	568	316	474	281	422	246	368	292	438	250	375
	28	364	546	304	456	270	405	235	353	275	413	235	353
	29	349	524	291	437	259	388	225	338	259	389	221	331
	30	334	502	279	418	247	371	215	322	243	366	206	310
	32	306	458	254	381	225	338	195	293	214	321	181	272
	34	277	416	230	346	204	306	176	264	189	285	161	241
	36	250	375	207	311	183	275	157	236	169	254	143	215
	38	225	337	186	279	164	246	141	212	152	228	129	193
	40	203	304	168	252	148	222	127	191	137	206	116	174
Properties													
M_{nx}/Ω_b $\phi_b M_{nx}$	kip-ft	214	322	170	255	146	219	121	182	219	329	185	277
M_{ny}/Ω_b $\phi_b M_{ny}$	kip-ft	187	282	148	223	127	191	105	158	163	244	137	206
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		14500		12000		10600		9110		13600		12000	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		10700		8820		7790		6690		6900		6100	
r_{mx}/r_{my}		1.16		1.17		1.17		1.17		1.40		1.40	
r_{my}, in.		3.96		4.01		4.04		4.07		3.16		3.21	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE HSS12

Table IV-1A (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS12x8x				HSS12x6x							
		⅜		¼		⅝		½		⅜		5⁄16	
t_{des}, in,		0.349		0.233		0.581		0.465		0.349		0.291	
Steel, lb/ft		47.9		32.6		67.8		55.7		42.8		36.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	470	705	372	557	560	841	478	716	397	595	353	530
	1	470	704	371	557	559	840	477	715	396	594	352	529
	2	468	702	370	555	556	835	474	711	394	591	350	526
	3	465	698	368	552	551	828	469	704	390	585	347	521
	4	462	693	365	547	544	817	463	695	385	577	343	514
	5	457	686	361	542	535	804	455	683	378	567	337	505
	6	452	678	356	535	524	787	445	668	370	556	330	495
	7	445	668	351	527	512	769	434	652	361	542	322	483
	8	438	657	345	518	498	748	422	633	351	527	313	469
	9	430	644	338	507	482	725	408	613	340	510	303	454
	10	421	631	331	496	466	700	394	590	328	492	292	438
	11	411	616	323	484	448	673	378	567	315	473	281	421
	12	401	601	314	472	429	645	362	542	302	453	269	403
	13	390	584	305	458	410	616	344	517	288	432	256	385
	14	378	567	296	444	390	586	327	490	273	410	244	365
	15	366	549	286	429	370	556	309	464	259	388	231	346
	16	354	530	276	414	349	525	291	438	244	366	217	326
	17	341	511	266	399	329	494	275	413	229	344	204	306
	18	328	492	255	383	308	463	258	388	214	322	191	287
	19	315	472	244	367	288	433	242	364	200	300	178	267
	20	301	452	234	350	268	403	226	339	186	279	166	248
	21	288	432	223	334	248	373	210	316	172	258	153	230
	22	274	412	212	318	229	345	195	293	158	237	141	212
	23	261	392	201	302	211	317	180	270	145	218	129	194
	24	248	372	190	286	194	291	165	248	133	200	119	178
	25	235	352	180	270	178	268	152	229	123	184	109	164
	26	222	332	170	254	165	248	141	211	114	170	101	152
	27	209	313	160	239	153	230	130	196	105	158	93.9	141
	28	197	295	150	224	142	214	121	182	97.9	147	87.3	131
	29	184	277	140	210	133	199	113	170	91.2	137	81.4	122
	30	172	259	131	196	124	186	106	159	85.3	128	76.0	114
	32	152	227	115	172	109	164	92.9	140	74.9	112	66.8	100
	34	134	201	102	153	96.4	145	82.2	124	66.4	99.6	59.2	88.8
	36	120	180	90.7	136	86.0	129	73.4	110	59.2	88.8	52.8	79.2
	38	107	161	81.4	122	77.2	116	65.8	99.0	53.1	79.7	47.4	71.1
	40	97.0	145	73.5	110			59.4	89.3	48.0	71.9	42.8	64.2

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	147	221	105	158	182	274	154	232	123	185	106	160
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	108	163	76.9	116	109	164	92.1	138	73.5	110	63.2	94.9

	HSS12x8x⅜	HSS12x8x¼	HSS12x6x⅝	HSS12x6x½	HSS12x6x⅜	HSS12x6x5⁄16
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}	10100	7690	10800	9520	8120	7260
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}	5100	3860	3380	2980	2520	2250
r_{mx}/r_{my}	1.41	1.41	1.79	1.79	1.80	1.80
r_{my}, in.	3.27	3.32	2.39	2.44	2.49	2.52

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.

4

COMPOSITE
HSS12–HSS10

Table IV-1A (continued)

Available Strength in Axial Compression, kips

Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS12x6x				HSS10x8x							
		¼		3⁄16		5⁄8		½		3⁄8		5⁄16	
t_{des}, in.		0.233		0.174		0.581		0.465		0.349		0.291	
Steel, lb/ft		29.2		22.2		67.8		55.7		42.8		36.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	309	464	255	382	570	855	491	737	410	615	367	550
	1	309	463	254	382	569	854	491	736	410	615	366	549
	2	307	460	253	379	567	851	489	733	408	613	365	547
	3	304	456	250	376	564	846	486	729	406	609	363	544
	4	300	450	247	371	559	839	482	723	403	604	360	540
	5	295	442	243	364	553	830	477	716	399	598	356	534
	6	288	433	238	356	546	819	471	707	394	591	352	528
	7	281	422	232	347	538	807	464	696	388	582	347	520
	8	273	410	225	337	528	792	456	684	382	572	341	511
	9	264	396	218	326	518	777	447	671	374	561	334	501
	10	255	382	210	314	506	759	438	657	366	549	327	490
	11	245	367	201	302	494	741	427	641	358	537	319	479
	12	234	351	192	288	481	721	416	624	348	523	311	466
	13	223	334	183	275	467	700	404	607	339	508	302	453
	14	212	317	174	261	452	678	392	588	329	493	293	439
	15	200	300	164	246	437	657	379	569	318	477	283	425
	16	188	283	154	232	422	635	366	549	307	460	273	410
	17	177	265	145	217	407	612	352	528	296	443	263	395
	18	165	248	135	203	392	589	338	507	284	426	253	379
	19	154	231	126	189	376	565	324	486	272	409	242	364
	20	143	214	116	175	360	541	310	465	261	391	232	348
	21	132	198	107	161	344	517	296	443	249	373	221	332
	22	121	182	98.7	148	328	493	281	422	237	355	210	316
	23	111	166	90.3	135	312	470	267	401	225	338	200	300
	24	102	153	83.0	124	297	446	253	380	214	320	189	284
	25	93.8	141	76.4	115	281	422	239	359	202	303	179	269
	26	86.7	130	70.7	106	266	399	226	338	191	286	169	253
	27	80.4	121	65.5	98.3	251	377	212	318	179	269	159	238
	28	74.8	112	60.9	91.4	236	354	199	299	169	253	149	224
	29	69.7	105	56.8	85.2	221	333	187	280	158	237	140	209
	30	65.1	97.7	53.1	79.6	207	311	175	263	148	221	130	196
	32	57.3	85.9	46.7	70.0	182	274	154	231	130	195	115	172
	34	50.7	76.1	41.3	62.0	161	242	136	205	115	172	102	152
	36	45.2	67.9	36.9	55.3	144	216	121	183	102	154	90.6	136
	38	40.6	60.9	33.1	49.6	129	194	109	164	92.0	138	81.3	122
	40	36.6	55.0	29.9	44.8	116	175	98.4	148	83.0	125	73.4	110
Properties													
M_{nx}/Ω_b $\phi_b M_{nx}$	kip-ft	88.7	133	69.6	105	165	247	139	209	111	167	95.6	144
M_{ny}/Ω_b $\phi_b M_{ny}$	kip-ft	52.2	78.5	39.3	59.1	140	210	118	178	94.1	141	80.9	122
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		6230		5120		8440		7480		6340		5610	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		1930		1570		5820		5140		4360		3850	
r_{mx}/r_{my}		1.80		1.81		1.20		1.21		1.21		1.21	
r_{my}, in.		2.54		2.57		3.09		3.14		3.19		3.22	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE HSS10

Table IV-1A (continued)

Available Strength in Axial Compression, kips

Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS10x8x				HSS10x6x							
		¼		3⁄16		⅝		½		⅜		5⁄16	
t_{des}, in,		0.233		0.174		0.581		0.465		0.349		0.291	
Steel, lb/ft		29.2		22.2		59.3		48.9		37.7		31.8	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	323	484	277	416	491	738	415	623	344	515	306	458
	1	322	484	277	415	490	737	415	622	343	514	305	458
	2	321	482	276	413	487	732	412	618	341	511	303	455
	3	319	479	274	411	483	725	408	612	338	507	300	451
	4	317	475	272	407	476	716	402	603	333	500	296	444
	5	313	470	269	403	468	703	395	593	327	491	291	437
	6	309	464	265	397	458	689	386	580	320	481	285	428
	7	305	457	261	391	447	672	377	565	312	469	278	417
	8	299	449	256	384	434	653	365	548	303	455	270	405
	9	293	440	251	376	420	632	353	530	293	440	261	392
	10	287	430	245	367	405	609	340	510	283	424	252	378
	11	280	420	239	358	389	585	326	489	271	407	242	363
	12	272	409	232	348	372	560	311	467	260	389	231	347
	13	265	397	225	337	355	533	296	445	247	371	220	331
	14	256	384	217	326	337	506	282	423	235	352	209	314
	15	248	371	210	315	319	479	267	401	222	332	198	297
	16	239	358	202	303	300	451	252	379	209	313	186	280
	17	230	344	194	291	282	423	237	357	196	293	175	262
	18	220	331	186	278	263	396	222	334	183	274	163	245
	19	211	316	177	266	245	369	208	312	170	255	152	228
	20	201	302	169	254	228	342	193	291	158	236	141	212
	21	192	288	161	241	210	316	179	269	145	218	130	196
	22	183	274	152	229	194	291	166	249	134	201	120	180
	23	173	260	144	216	177	266	152	229	122	184	110	165
	24	164	246	136	204	163	245	140	210	112	169	101	151
	25	155	232	128	192	150	225	129	194	104	155	92.9	139
	26	146	218	120	180	139	208	119	179	95.7	144	85.9	129
	27	137	205	113	169	129	193	110	166	88.8	133	79.7	119
	28	128	192	105	158	120	180	103	154	82.5	124	74.1	111
	29	120	179	97.9	147	111	168	95.7	144	76.9	116	69.0	104
	30	112	168	91.5	137	104	157	89.4	134	71.9	108	64.5	96.8
	32	98.2	147	80.4	121	91.5	138	78.6	118	63.2	94.9	56.7	85.1
	34	87.0	131	71.2	107	81.1	122	69.6	105	56.0	84.0	50.2	75.3
	36	77.6	116	63.5	95.3	72.3	109	62.1	93.3	49.9	75.0	44.8	67.2
	38	69.7	104	57.0	85.5	64.9	97.6	55.7	83.8	44.8	67.3	40.2	60.3
	40	62.9	94.3	51.5	77.2					40.4	60.7	36.3	54.4

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	79.5	119	62.0	93.3	135	203	115	172	92.1	138	79.6	120
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	67.2	101	52.1	78.3	92.8	140	78.8	118	63.0	94.6	54.2	81.4

$P_{ex}(L_c)^2/10^4$, kip-in.^{2}	4810	3950	6600	5860	5020	4530
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}	3300	2700	2810	2500	2130	1910
r_{mx}/r_{my}	1.21	1.21	1.53	1.53	1.54	1.54
r_{my}, in.	3.25	3.28	2.34	2.39	2.44	2.47

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.

4

COMPOSITE
HSS10

Table IV-1A (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape	HSS10x6x				HSS10x5x							
	¼		3/16		⅜		5/16		¼		3/16	
t_{des}, in.	0.233		0.174		0.349		0.291		0.233		0.174	
Steel, lb/ft	25.8		19.6		35.1		29.7		24.1		18.4	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y												
0	267	401	227	340	310	464	275	412	239	359	202	303
1	267	400	226	340	309	463	274	411	239	358	201	302
2	265	398	225	338	306	459	272	408	237	355	200	299
3	262	394	223	334	302	453	268	402	233	350	197	295
4	259	388	220	329	296	444	263	395	229	343	193	289
5	254	381	216	323	289	433	257	385	223	335	188	282
6	249	373	211	316	280	420	249	374	217	325	182	273
7	243	364	205	308	270	406	241	361	209	314	176	263
8	235	353	199	299	259	389	231	346	201	301	168	253
9	228	342	192	288	248	371	220	331	192	287	160	241
10	219	329	185	277	235	352	209	314	182	273	152	228
11	210	316	177	266	222	333	198	297	172	258	143	215
12	201	302	169	253	208	312	186	279	161	242	134	201
13	192	287	161	241	194	291	173	260	151	226	125	188
14	182	272	152	228	180	270	161	242	140	210	116	174
15	172	257	143	215	166	250	149	223	129	194	107	160
16	161	242	134	201	153	229	137	205	119	178	97.8	147
17	151	227	126	188	140	211	125	188	109	163	89.0	134
18	141	212	117	175	129	193	114	171	98.6	148	80.6	121
19	131	197	108	162	117	176	103	154	89.0	133	72.5	109
20	122	182	100	150	106	159	92.6	139	80.3	120	65.4	98.1
21	112	168	92.0	138	96.2	145	84.0	126	72.8	109	59.3	89.0
22	103	154	84.0	126	87.6	132	76.5	115	66.4	99.5	54.1	81.1
23	94.1	141	76.9	115	80.2	121	70.0	105	60.7	91.1	49.5	74.2
24	86.5	130	70.6	106	73.6	111	64.3	96.5	55.8	83.6	45.4	68.1
25	79.7	120	65.1	97.6	67.9	102	59.3	88.9	51.4	77.1	41.9	62.8
26	73.7	111	60.2	90.2	62.7	94.3	54.8	82.2	47.5	71.3	38.7	58.1
27	68.3	102	55.8	83.7	58.2	87.5	50.8	76.2	44.1	66.1	35.9	53.8
28	63.5	95.3	51.9	77.8	54.1	81.3	47.2	70.9	41.0	61.4	33.4	50.1
29	59.2	88.8	48.4	72.5	50.4	75.8	44.0	66.1	38.2	57.3	31.1	46.7
30	55.3	83.0	45.2	67.8	47.1	70.8	41.2	61.7	35.7	53.5	29.1	43.6
32	48.6	72.9	39.7	59.6	41.4	62.3	36.2	54.3	31.4	47.0	25.6	38.3
34	43.1	64.6	35.2	52.8	36.7	55.2	32.0	48.1	27.8	41.7	22.6	33.9
36	38.4	57.6	31.4	47.1								
38	34.5	51.7	28.2	42.2								
40	31.1	46.7	25.4	38.1								
Properties												
M_{nx}/Ω_b / $\phi_b M_{nx}$, kip-ft	66.5	99.9	52.1	78.3	82.4	124	71.5	107	59.7	89.8	47.0	70.6
M_{ny}/Ω_b / $\phi_b M_{ny}$, kip-ft	45.1	67.7	35.0	52.7	49.2	73.9	42.4	63.8	35.4	53.2	27.5	41.3
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}	3880		3180		4320		3930		3410		2800	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}	1640		1340		1350		1220		1050		859	
r_{mx}/r_{my}	1.54		1.54		1.79		1.79		1.80		1.81	
r_{my}, in.	2.49		2.52		2.05		2.07		2.10		2.13	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS9

Table IV-1A (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS9x7x											
		5⁄8		1⁄2		3⁄8		5⁄16		1⁄4		3⁄16	
t_{des}, in,		0.581		0.465		0.349		0.291		0.233		0.174	
Steel, lb/ft		59.3		48.9		37.7		31.8		25.8		19.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	491	738	420	631	349	523	311	466	272	408	232	348
	1	490	737	420	630	348	522	310	465	272	408	232	348
	2	488	734	418	627	347	520	309	463	271	406	231	346
	3	485	728	415	622	344	516	307	460	269	403	229	343
	4	480	721	410	615	341	511	303	455	266	399	226	339
	5	473	711	405	607	336	504	299	449	262	393	223	335
	6	466	700	398	597	331	496	295	442	258	387	219	329
	7	457	687	390	585	324	486	289	434	253	379	215	322
	8	447	672	381	572	317	476	283	424	247	371	210	315
	9	436	655	372	557	309	464	276	413	241	361	204	306
	10	424	637	361	541	301	451	268	402	234	351	198	297
	11	411	618	350	524	291	437	260	390	227	340	192	288
	12	398	598	338	506	282	423	251	377	219	329	185	278
	13	383	576	325	487	271	407	242	363	211	316	178	267
	14	368	554	312	468	261	391	233	349	203	304	170	256
	15	353	531	298	448	250	375	223	334	194	291	163	244
	16	337	507	285	427	239	358	213	319	185	278	155	233
	17	321	483	271	406	227	341	203	304	176	264	147	221
	18	305	459	257	385	216	324	192	289	167	250	139	209
	19	289	435	243	365	204	306	182	273	158	237	132	197
	20	273	411	230	345	193	289	172	258	149	223	124	186
	21	257	387	217	326	181	272	162	243	140	210	116	174
	22	242	363	204	307	170	255	152	228	131	197	108	163
	23	226	340	191	288	159	239	142	213	123	184	101	151
	24	211	317	179	269	148	223	133	199	114	171	93.8	141
	25	196	295	167	251	138	207	123	185	106	159	86.6	130
	26	182	273	155	234	128	192	114	171	98.0	147	80.1	120
	27	169	253	144	217	118	178	106	159	90.9	136	74.3	111
	28	157	236	134	201	110	165	98.4	148	84.5	127	69.1	104
	29	146	220	125	188	103	154	91.8	138	78.8	118	64.4	96.6
	30	137	205	117	175	95.9	144	85.7	129	73.6	110	60.2	90.2
	32	120	180	103	154	84.3	126	75.4	113	64.7	97.0	52.9	79.3
	34	106	160	90.8	137	74.7	112	66.7	100	57.3	86.0	46.8	70.3
	36	94.9	143	81.0	122	66.6	99.9	59.5	89.3	51.1	76.7	41.8	62.7
	38	85.1	128	72.7	109	59.8	89.7	53.4	80.2	45.9	68.8	37.5	56.2
	40	76.8	115	65.6	98.7	54.0	80.9	48.2	72.3	41.4	62.1	33.8	50.8
Properties													
M_{nx}/Ω_b | $\phi_b M_{nx}$	kip-ft	127	191	108	162	86.4	130	74.6	112	62.2	93.5	48.6	73.0
M_{ny}/Ω_b | $\phi_b M_{ny}$	kip-ft	106	159	89.7	135	71.6	108	61.8	92.9	51.4	77.3	40.1	60.2
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		5690		5080		4330		3870		3330		2720	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		3740		3320		2840		2530		2180		1780	
r_{mx}/r_{my}		1.23		1.24		1.23		1.24		1.24		1.24	
r_{my}, in.		2.68		2.73		2.78		2.81		2.84		2.87	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS9

Table IV-1A (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape	HSS9x5x											
	5⁄8		1⁄2		3⁄8		5⁄16		1⁄4		3⁄16	
t_{des}, in.	0.581		0.465		0.349		0.291		0.233		0.174	
Steel, lb/ft	50.8		42.1		32.6		27.6		22.4		17.1	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y: 0	419	630	347	522	285	427	253	379	220	330	185	278
1	418	628	346	521	284	426	252	378	219	329	185	277
2	414	623	344	516	282	422	250	375	218	326	183	275
3	409	614	339	509	278	417	247	370	215	322	180	271
4	400	602	333	500	272	408	242	363	210	316	177	265
5	390	587	325	488	265	398	236	354	205	308	172	259
6	378	568	315	473	257	386	229	343	199	299	167	251
7	364	548	304	457	248	372	221	331	192	288	161	241
8	349	525	292	439	238	357	212	318	184	276	154	231
9	333	500	279	419	227	340	202	303	176	264	147	220
10	315	473	265	398	215	323	192	288	167	250	139	209
11	297	446	250	376	203	304	181	271	157	236	131	197
12	278	418	235	353	190	285	170	255	148	221	123	184
13	259	389	220	330	177	266	158	238	138	207	114	172
14	239	360	204	307	164	246	147	221	128	192	106	159
15	220	331	189	284	151	227	136	204	118	177	97.5	146
16	202	303	173	261	140	210	125	187	108	162	89.2	134
17	184	276	159	238	128	193	114	171	98.8	148	81.2	122
18	166	250	144	217	117	176	103	155	89.7	135	73.5	110
19	149	224	130	196	107	160	93.0	139	80.8	121	66.0	99.0
20	135	202	117	177	96.5	145	83.9	126	72.9	109	59.6	89.4
21	122	184	107	160	87.5	131	76.1	114	66.1	99.2	54.1	81.1
22	111	167	97.1	146	79.7	120	69.4	104	60.3	90.4	49.2	73.9
23	102	153	88.8	134	72.9	110	63.5	95.2	55.1	82.7	45.1	67.6
24	93.5	141	81.6	123	67.0	101	58.3	87.4	50.6	76.0	41.4	62.1
25	86.2	130	75.2	113	61.7	92.8	53.7	80.6	46.7	70.0	38.1	57.2
26	79.7	120	69.5	104	57.1	85.8	49.7	74.5	43.1	64.7	35.3	52.9
27	73.9	111	64.5	96.9	52.9	79.5	46.1	69.1	40.0	60.0	32.7	49.0
28	68.7	103	59.9	90.1	49.2	74.0	42.8	64.2	37.2	55.8	30.4	45.6
29	64.1	96.3	55.9	84.0	45.9	69.0	39.9	59.9	34.7	52.0	28.3	42.5
30	59.9	90.0	52.2	78.5	42.9	64.4	37.3	56.0	32.4	48.6	26.5	39.7
32	52.6	79.1	45.9	69.0	37.7	56.6	32.8	49.2	28.5	42.7	23.3	34.9
34							29.0	43.6	25.2	37.8	20.6	30.9

Properties

			5⁄8		1⁄2		3⁄8		5⁄16		1⁄4		3⁄16	
M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	101	151	86.0	129	69.3	104	60.2	90.4	50.4	75.8	39.5	59.4
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	65.1	97.9	55.8	83.9	45.0	67.6	38.8	58.3	32.2	48.5	25.2	37.9
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}			4270		3840		3280		2960		2590		2120	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}			1600		1430		1220		1100		958		783	
r_{mx}/r_{my}			1.63		1.64		1.64		1.64		1.64		1.65	
r_{my}, in.			1.92		1.97		2.03		2.05		2.08		2.10	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS8

Table IV-1A (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS8x6x											
		⅝		½		⅜		5⁄16		¼		3⁄16	
t_{des}, in.		0.581		0.465		0.349		0.291		0.233		0.174	
Steel, lb/ft		50.8		42.1		32.6		27.6		22.4		17.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	419	630	351	526	290	435	258	387	225	338	190	285
	1	418	629	350	525	289	434	257	386	225	337	190	285
	2	416	625	348	522	288	431	256	384	223	335	189	283
	3	412	619	344	516	285	427	253	380	221	332	187	280
	4	406	610	339	509	281	421	250	375	218	327	184	276
	5	398	599	333	499	276	413	245	368	214	321	181	271
	6	389	585	325	488	270	404	240	360	209	314	176	265
	7	379	570	317	475	263	394	234	351	204	306	172	258
	8	368	553	307	461	255	382	227	341	198	297	166	250
	9	355	534	296	446	246	369	219	329	191	287	161	241
	10	342	514	286	429	237	355	211	317	184	276	154	232
	11	327	492	274	412	227	340	203	304	176	265	148	222
	12	312	469	262	394	217	325	193	290	168	253	141	211
	13	297	446	250	375	206	309	184	276	160	240	134	201
	14	281	422	237	356	195	293	174	262	152	228	126	190
	15	265	398	224	336	184	276	165	247	143	215	119	179
	16	248	373	210	316	173	259	155	232	134	202	112	167
	17	232	349	197	297	162	242	145	217	126	189	104	156
	18	216	325	184	277	151	226	135	203	117	176	96.9	145
	19	200	301	171	258	140	209	125	188	109	163	89.7	135
	20	185	278	159	239	129	194	116	174	101	151	82.7	124
	21	170	256	147	220	119	178	107	160	92.6	139	75.9	114
	22	156	234	135	202	109	164	98.0	147	84.8	127	69.3	104
	23	142	214	123	185	100	151	89.6	134	77.6	116	63.4	95.1
	24	131	196	113	170	92.1	138	82.3	123	71.3	107	58.2	87.3
	25	120	181	104	157	84.9	128	75.9	114	65.7	98.5	53.7	80.5
	26	111	167	96.4	145	78.5	118	70.1	105	60.7	91.1	49.6	74.4
	27	103	155	89.4	134	72.8	109	65.0	97.6	56.3	84.5	46.0	69.0
	28	96.0	144	83.1	125	67.6	102	60.5	90.7	52.4	78.5	42.8	64.2
	29	89.5	135	77.5	116	63.1	94.8	56.4	84.6	48.8	73.2	39.9	59.8
	30	83.7	126	72.4	109	58.9	88.6	52.7	79.0	45.6	68.4	37.3	55.9
	32	73.5	111	63.6	95.7	51.8	77.8	46.3	69.5	40.1	60.1	32.7	49.1
	34	65.1	97.9	56.4	84.7	45.9	69.0	41.0	61.5	35.5	53.3	29.0	43.5
	36	58.1	87.3	50.3	75.6	40.9	61.5	36.6	54.9	31.7	47.5	25.9	38.8
	38			45.1	67.8	36.7	55.2	32.8	49.3	28.4	42.6	23.2	34.8
	40							29.6	44.5	25.7	38.5	21.0	31.4
Properties													
M_{nx}/Ω_b | $\phi_b M_{nx}$	kip-ft	94.3	142	80.6	121	64.9	97.5	56.2	84.5	46.9	70.5	37.0	55.6
M_{ny}/Ω_b | $\phi_b M_{ny}$	kip-ft	76.4	115	65.2	98.0	52.6	79.1	45.4	68.2	37.8	56.9	29.7	44.6
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		3650		3270		2790		2520		2190		1790	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		2260		2020		1730		1560		1350		1100	
r_{mx}/r_{my}		1.27		1.27		1.27		1.27		1.27		1.28	
r_{my}, in.		2.27		2.32		2.38		2.40		2.43		2.46	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS8

Table IV-1A (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape	HSS8x4x											
	⅝		½		⅜		5⁄16		¼		3⁄16	
t_{des}, in,	0.581		0.465		0.349		0.291		0.233		0.174	
Steel, lb/ft	42.3		35.2		27.5		23.3		19.0		14.5	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y												
0	350	526	292	438	230	345	204	306	176	264	147	220
1	349	524	290	436	229	344	203	304	175	263	146	219
2	344	517	287	431	226	339	200	300	173	260	144	217
3	336	505	280	422	221	332	196	294	170	254	141	212
4	325	489	272	409	215	322	190	285	165	247	137	206
5	312	469	262	393	206	309	183	274	158	238	132	198
6	297	446	250	375	196	295	174	262	151	227	126	189
7	279	420	236	355	186	280	165	247	143	214	119	179
8	261	392	221	332	175	263	155	232	134	201	112	167
9	241	362	205	309	163	245	144	215	125	187	104	156
10	221	332	189	284	151	227	132	198	115	172	95.5	143
11	200	301	173	260	139	209	121	181	105	158	87.3	131
12	180	271	156	235	126	190	109	164	95.3	143	79.1	119
13	161	241	140	211	114	172	98.5	148	85.7	128	71.0	106
14	142	213	125	188	102	154	88.5	133	76.3	115	63.2	94.8
15	124	186	110	165	91.0	137	78.9	119	67.4	101	55.7	83.5
16	109	163	96.6	145	80.1	120	69.7	105	59.2	88.8	48.9	73.4
17	96.4	145	85.6	129	71.0	107	61.7	92.7	52.4	78.7	43.4	65.0
18	85.9	129	76.4	115	63.3	95.1	55.0	82.7	46.8	70.2	38.7	58.0
19	77.1	116	68.5	103	56.8	85.4	49.4	74.2	42.0	63.0	34.7	52.1
20	69.6	105	61.9	93.0	51.3	77.1	44.6	67.0	37.9	56.8	31.3	47.0
21	63.1	94.9	56.1	84.3	46.5	69.9	40.4	60.8	34.4	51.5	28.4	42.6
22	57.5	86.5	51.1	76.8	42.4	63.7	36.8	55.4	31.3	47.0	25.9	38.8
23	52.6	79.1	46.8	70.3	38.8	58.3	33.7	50.7	28.6	43.0	23.7	35.5
24	48.3	72.7	43.0	64.6	35.6	53.5	31.0	46.5	26.3	39.5	21.8	32.6
25	44.6	67.0	39.6	59.5	32.8	49.3	28.5	42.9	24.2	36.4	20.0	30.1
26			36.6	55.0	30.3	45.6	26.4	39.6	22.4	33.6	18.5	27.8
27							24.5	36.8	20.8	31.2	17.2	25.8
28											16.0	24.0

Properties

			⅝		½		⅜		5⁄16		¼		3⁄16	
M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	71.0	107	61.6	92.5	50.1	75.3	43.5	65.4	36.6	55.0	28.8	43.3
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	42.4	63.7	36.8	55.3	29.9	45.0	26.0	39.1	21.7	32.7	17.0	25.6
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}			2570		2330		2010		1820		1610		1330	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}			800		727		628		568		498		411	
r_{mx}/r_{my}			1.79		1.79		1.79		1.79		1.80		1.80	
r_{my}, in.			1.51		1.56		1.61		1.63		1.66		1.69	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS8–HSS7

Table IV-1A (continued)

Available Strength in Axial Compression, kips

Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS8x4x		HSS7x5x									
		⅛		½		⅜		5⁄16		¼		3⁄16	
t_{des}, in.		0.116		0.465		0.349		0.291		0.233		0.174	
Steel, lb/ft		9.86		35.2		27.5		23.3		19.0		14.5	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	114	170	292	438	235	353	209	313	181	272	152	228
	1	113	170	291	437	235	352	208	312	181	271	152	227
	2	112	167	288	433	233	349	206	310	179	269	150	225
	3	109	164	284	427	229	344	203	305	177	265	148	222
	4	106	159	278	419	225	337	199	299	173	260	145	218
	5	102	153	271	408	219	328	194	291	169	253	141	212
	6	96.9	145	263	395	212	318	188	282	164	245	137	205
	7	91.5	137	253	380	204	306	181	272	158	236	132	198
	8	85.6	128	242	364	195	293	174	260	151	227	126	189
	9	79.4	119	231	347	186	278	165	248	144	216	120	180
	10	73.0	110	219	328	176	263	156	235	136	204	114	170
	11	66.6	99.8	206	309	165	248	147	221	128	193	107	160
	12	60.1	90.2	192	289	154	231	138	207	120	180	99.8	150
	13	53.8	80.8	179	269	143	216	128	192	112	168	92.8	139
	14	47.8	71.7	166	249	133	200	119	178	104	155	85.7	129
	15	42.0	62.9	152	229	123	185	109	164	95.4	143	78.8	118
	16	36.9	55.3	139	209	113	170	99.7	150	87.4	131	72.0	108
	17	32.7	49.0	127	190	104	156	90.7	136	79.5	119	65.3	98.0
	18	29.1	43.7	114	172	94.2	142	81.9	123	72.0	108	58.9	88.4
	19	26.1	39.2	103	154	85.1	128	73.6	111	64.6	97.0	52.9	79.3
	20	23.6	35.4	92.7	139	76.8	115	66.4	99.9	58.3	87.5	47.7	71.6
	21	21.4	32.1	84.1	126	69.6	105	60.3	90.6	52.9	79.4	43.3	65.0
	22	19.5	29.3	76.6	115	63.4	95.4	54.9	82.5	48.2	72.3	39.5	59.2
	23	17.8	26.8	70.1	105	58.0	87.2	50.2	75.5	44.1	66.2	36.1	54.1
	24	16.4	24.6	64.4	96.8	53.3	80.1	46.1	69.4	40.5	60.8	33.2	49.7
	25	15.1	22.7	59.3	89.2	49.1	73.8	42.5	63.9	37.3	56.0	30.6	45.8
	26	14.0	20.9	54.9	82.5	45.4	68.3	39.3	59.1	34.5	51.8	28.2	42.4
	27	12.9	19.4	50.9	76.5	42.1	63.3	36.5	54.8	32.0	48.0	26.2	39.3
	28	12.0	18.1	47.3	71.1	39.2	58.9	33.9	51.0	29.8	44.6	24.4	36.5
	29			44.1	66.3	36.5	54.9	31.6	47.5	27.7	41.6	22.7	34.1
	30			41.2	61.9	34.1	51.3	29.5	44.4	25.9	38.9	21.2	31.8
	32					30.0	45.1	26.0	39.0	22.8	34.2	18.6	28.0
	34											16.5	24.8
Properties													
M_{nx}/Ω_b | $\phi_b M_{nx}$	kip-ft	20.6	31.0	57.4	86.3	46.7	70.1	40.5	60.9	34.0	51.1	26.8	40.2
M_{ny}/Ω_b | $\phi_b M_{ny}$	kip-ft	11.7	17.5	44.9	67.5	36.3	54.6	31.6	47.5	26.5	39.8	20.7	31.2
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		1010		1960		1690		1530		1350		1110	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		310		1120		967		872		766		627	
r_{mx}/r_{my}		1.81		1.32		1.32		1.32		1.33		1.33	
r_{my}, in.		1.71		1.91		1.97		1.99		2.02		2.05	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS7

Table IV-1A (continued)

Available Strength in Axial Compression, kips

Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS7x5x		HSS7x4x									
		⅛		½		⅜		5⁄16		¼		3⁄16	
t_{des}, in,		0.116		0.465		0.349		0.291		0.233		0.174	
Steel, lb/ft		9.86		31.8		24.9		21.2		17.3		13.3	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	122	183	264	396	207	311	183	275	158	238	132	198
	1	122	183	263	395	206	309	183	274	158	237	131	197
	2	121	181	259	389	203	305	180	270	156	234	130	195
	3	119	178	253	381	199	298	176	264	152	229	127	190
	4	116	174	245	369	193	289	171	256	148	222	123	185
	5	113	169	236	354	185	279	164	246	142	213	118	178
	6	109	164	224	337	177	266	156	235	136	203	113	169
	7	105	157	212	318	168	252	148	222	128	192	107	160
	8	100	150	198	297	157	236	138	207	120	180	99.8	150
	9	95.0	143	183	275	146	220	128	192	112	167	92.7	139
	10	89.6	134	168	253	135	203	118	177	103	154	85.3	128
	11	84.0	126	153	230	124	186	107	161	93.8	141	77.8	117
	12	78.2	117	138	207	112	169	97.6	147	84.9	127	70.4	106
	13	72.4	109	123	185	101	152	88.2	133	76.1	114	63.1	94.6
	14	66.6	99.8	109	164	90.1	135	79.0	119	67.7	102	56.1	84.1
	15	60.8	91.3	95.7	144	79.7	120	70.2	106	59.6	89.4	49.3	73.9
	16	55.3	82.9	84.1	126	70.0	105	61.8	92.9	52.4	78.6	43.3	65.0
	17	49.9	74.9	74.5	112	62.0	93.2	54.8	82.3	46.4	69.6	38.4	57.6
	18	44.7	67.0	66.4	99.9	55.3	83.2	48.9	73.4	41.4	62.1	34.2	51.3
	19	40.1	60.2	59.6	89.6	49.7	74.6	43.8	65.9	37.2	55.8	30.7	46.1
	20	36.2	54.3	53.8	80.9	44.8	67.4	39.6	59.5	33.5	50.3	27.7	41.6
	21	32.8	49.3	48.8	73.4	40.7	61.1	35.9	53.9	30.4	45.6	25.1	37.7
	22	29.9	44.9	44.5	66.8	37.0	55.7	32.7	49.2	27.7	41.6	22.9	34.4
	23	27.4	41.1	40.7	61.2	33.9	50.9	29.9	45.0	25.4	38.0	21.0	31.4
	24	25.1	37.7	37.4	56.2	31.1	46.8	27.5	41.3	23.3	34.9	19.3	28.9
	25	23.2	34.8	34.4	51.8	28.7	43.1	25.3	38.1	21.5	32.2	17.7	26.6
	26	21.4	32.1			26.5	39.9	23.4	35.2	19.8	29.8	16.4	24.6
	27	19.9	29.8							18.4	27.6	15.2	22.8
	28	18.5	27.7										
	29	17.2	25.8										
	30	16.1	24.1										
	32	14.1	21.2										
	34	12.5	18.8										

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	19.0	28.6	49.1	73.7	40.1	60.2	35.2	52.9	29.5	44.3	23.3	35.0
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	14.5	21.8	32.4	48.7	26.6	39.9	23.2	34.9	19.5	29.3	15.3	23.0
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}			842		1620		1410		1280		1130		940	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}			475		637		553		501		440		364	
r_{mx}/r_{my}			1.33		1.59		1.60		1.60		1.60		1.61	
r_{my}, in.			2.07		1.53		1.58		1.61		1.64		1.66	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

Table IV-1A (continued)

Available Strength in Axial Compression, kips

Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape	HSS7x4x		HSS6x5x							
	⅛		½		⅜		5⁄16		¼	
t_{des}, in,	0.116		0.465		0.349		0.291		0.233	
Steel, lb/ft	9.01		31.8		24.9		21.2		17.3	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y										
0	105	157	264	396	211	316	187	280	162	243
1	104	156	263	395	210	315	186	279	161	242
2	103	154	261	392	208	312	185	277	160	240
3	100	151	257	386	205	307	182	273	158	236
4	97.2	146	251	378	201	301	178	267	154	232
5	93.3	140	245	368	195	293	173	260	150	226
6	88.7	133	237	356	189	283	168	252	146	219
7	83.6	125	228	342	182	272	162	242	140	210
8	78.0	117	218	327	173	260	154	232	134	201
9	72.2	108	207	311	165	247	147	220	128	192
10	66.2	99.3	195	293	156	233	139	208	121	181
11	60.1	90.2	183	275	146	219	130	196	114	170
12	54.1	81.2	171	257	137	205	122	183	106	159
13	48.2	72.4	159	238	127	191	113	170	98.8	148
14	42.6	64.0	146	220	118	177	104	157	91.3	137
15	37.3	55.9	134	201	108	163	95.7	144	83.9	126
16	32.8	49.1	122	183	99.2	149	87.3	131	76.6	115
17	29.0	43.5	110	166	90.2	136	79.2	119	69.6	104
18	25.9	38.8	99.3	149	81.6	123	71.4	107	62.8	94.2
19	23.2	34.8	89.1	134	73.3	110	64.3	96.7	56.3	84.5
20	21.0	31.4	80.4	121	66.2	99.5	58.0	87.2	50.8	76.3
21	19.0	28.5	72.9	110	60.0	90.2	52.7	79.1	46.1	69.2
22	17.3	26.0	66.4	99.9	54.7	82.2	48.0	72.1	42.0	63.0
23	15.9	23.8	60.8	91.4	50.0	75.2	43.9	66.0	38.4	57.7
24	14.6	21.8	55.8	83.9	46.0	69.1	40.3	60.6	35.3	53.0
25	13.4	20.1	51.5	77.3	42.4	63.7	37.2	55.8	32.5	48.8
26	12.4	18.6	47.6	71.5	39.2	58.9	34.3	51.6	30.1	45.1
27	11.5	17.3	44.1	66.3	36.3	54.6	31.9	47.9	27.9	41.8
28	10.7	16.0	41.0	61.6	33.8	50.8	29.6	44.5	25.9	38.9
29			38.2	57.5	31.5	47.3	27.6	41.5	24.2	36.3
30			35.7	53.7	29.4	44.2	25.8	38.8	22.6	33.9
32					25.9	38.9	22.7	34.1	19.9	29.8

Properties

			HSS7x4x⅛		HSS6x5x½		HSS6x5x⅜		HSS6x5x5⁄16		HSS6x5x¼	
M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	16.7	25.0	44.8	67.4	36.6	54.9	31.9	47.9	26.8	40.3
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	10.7	16.1	39.4	59.3	32.1	48.2	27.9	41.9	23.5	35.3
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}			714		1310		1130		1030		905	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}			275		968		838		758		668	
r_{mx}/r_{my}			1.61		1.16		1.16		1.17		1.16	
r_{my}, in.			1.69		1.87		1.92		1.95		1.98	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates Lc/rmy equal to or greater than 200.
Dashed line indicates the Lc beyond which the bare steel strength controls.

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COMPOSITE
HSS6

Table IV-1A (continued)

Available Strength in Axial Compression, kips

Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape	HSS6x5x				HSS6x4x							
	3/16		1/8		1/2		3/8		5/16		1/4	
t_{des}, in,	0.174		0.116		0.465		0.349		0.291		0.233	
Steel, lb/ft	13.3		9.01		28.4		22.4		19.1		15.6	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y 0	135	203	108	162	236	355	185	278	163	244	141	211
1	135	203	108	162	235	353	184	277	162	243	140	210
2	134	201	107	160	232	348	182	273	160	240	138	207
3	132	198	105	158	226	340	178	267	156	235	135	203
4	129	194	103	154	219	329	173	259	151	227	131	196
5	126	189	100	150	210	315	166	249	145	218	126	189
6	122	183	96.7	145	199	300	158	238	138	208	120	180
7	117	176	92.8	139	188	282	149	224	130	196	113	170
8	112	168	88.6	133	175	263	140	210	122	183	106	159
9	106	160	84.0	126	161	243	130	195	113	169	98.0	147
10	101	151	79.1	119	148	222	119	179	104	155	90.1	135
11	94.6	142	74.1	111	134	201	109	164	94.5	142	82.0	123
12	88.4	133	69.0	103	120	181	98.4	148	85.8	129	74.0	111
13	82.0	123	63.8	95.7	107	161	88.2	133	77.2	116	66.2	99.3
14	75.7	114	58.6	87.9	94.3	142	78.4	118	68.9	104	58.7	88.1
15	69.5	104	53.5	80.3	82.3	124	68.9	104	60.9	91.6	51.6	77.6
16	63.4	95.1	48.6	72.9	72.3	109	60.5	91.0	53.5	80.5	45.4	68.3
17	57.5	86.2	43.8	65.7	64.0	96.2	53.6	80.6	47.4	71.3	40.3	60.5
18	51.7	77.6	39.2	58.8	57.1	85.9	47.8	71.9	42.3	63.6	35.9	54.0
19	46.4	69.7	35.2	52.7	51.3	77.1	42.9	64.5	38.0	57.1	32.2	48.4
20	41.9	62.9	31.7	47.6	46.3	69.5	38.7	58.2	34.3	51.5	29.1	43.7
21	38.0	57.0	28.8	43.2	42.0	63.1	35.1	52.8	31.1	46.7	26.4	39.7
22	34.6	52.0	26.2	39.3	38.2	57.5	32.0	48.1	28.3	42.6	24.0	36.1
23	31.7	47.5	24.0	36.0	35.0	52.6	29.3	44.0	25.9	38.9	22.0	33.1
24	29.1	43.7	22.0	33.1	32.1	48.3	26.9	40.4	23.8	35.8	20.2	30.4
25	26.8	40.2	20.3	30.5	29.6	44.5	24.8	37.3	21.9	33.0	18.6	28.0
26	24.8	37.2	18.8	28.2					20.3	30.5	17.2	25.9
27	23.0	34.5	17.4	26.1								
28	21.4	32.1	16.2	24.3								
29	19.9	29.9	15.1	22.6								
30	18.6	27.9	14.1	21.2								
32	16.4	24.6	12.4	18.6								
Properties												
M_{nx}/Ω_b | $\phi_b M_{nx}$ | kip-ft	21.2	31.8	15.1	22.6	37.9	57.0	31.4	47.2	27.5	41.3	23.1	34.7
M_{ny}/Ω_b | $\phi_b M_{ny}$ | kip-ft	18.5	27.8	13.1	19.6	28.3	42.5	23.3	35.0	20.4	30.6	17.1	25.8
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}	748		566		1070		935		849		752	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}	551		417		546		475		433		380	
r_{mx}/r_{my}	1.17		1.17		1.40		1.40		1.40		1.41	
r_{my}, in.	2.01		2.03		1.50		1.55		1.58		1.61	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS6

Table IV-1A (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS6x4x				HSS6x3x							
		3⁄16		1⁄8		1⁄2		3⁄8		5⁄16		1⁄4	
t_{des}, in,		0.174		0.116		0.465		0.349		0.291		0.233	
Steel, lb/ft		12.0		8.16		25.0		19.8		17.0		13.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	117	176	92.7	139	208	313	164	247	140	211	120	180
	1	117	175	92.2	138	206	310	163	245	139	209	119	178
	2	115	172	91.0	136	201	302	159	239	136	204	116	174
	3	112	169	88.9	133	193	290	153	230	131	197	112	168
	4	109	164	86.0	129	182	273	145	218	124	187	106	159
	5	105	157	82.5	124	169	254	135	203	116	175	98.7	148
	6	99.8	150	78.4	118	154	231	124	187	107	161	90.6	136
	7	94.2	141	73.8	111	138	207	113	169	97.3	146	81.9	123
	8	88.2	132	68.8	103	122	183	100	151	87.1	131	73.1	110
	9	81.8	123	63.6	95.4	105	158	88.0	132	76.7	115	64.8	97.4
	10	75.2	113	58.2	87.4	89.9	135	76.0	114	66.6	100	56.7	85.2
	11	68.5	103	52.8	79.3	75.2	113	64.7	97.2	57.0	85.7	48.8	73.4
	12	61.9	92.8	47.5	71.2	63.2	95.0	54.4	81.7	48.0	72.2	41.4	62.3
	13	55.4	83.1	42.3	63.4	53.8	80.9	46.3	69.6	40.9	61.5	35.3	53.1
	14	49.1	73.7	37.3	55.9	46.4	69.8	39.9	60.0	35.3	53.0	30.4	45.7
	15	43.1	64.7	32.6	48.8	40.4	60.8	34.8	52.3	30.7	46.2	26.5	39.9
	16	37.9	56.9	28.6	42.9	35.5	53.4	30.6	46.0	27.0	40.6	23.3	35.0
	17	33.6	50.4	25.3	38.0	31.5	47.3	27.1	40.7	23.9	36.0	20.6	31.0
	18	29.9	44.9	22.6	33.9	28.1	42.2	24.2	36.3	21.4	32.1	18.4	27.7
	19	26.9	40.3	20.3	30.4			21.7	32.6	19.2	28.8	16.5	24.8
	20	24.3	36.4	18.3	27.5							14.9	22.4
	21	22.0	33.0	16.6	24.9								
	22	20.0	30.1	15.1	22.7								
	23	18.3	27.5	13.8	20.8								
	24	16.8	25.3	12.7	19.1								
	25	15.5	23.3	11.7	17.6								
	26	14.4	21.5	10.8	16.3								
	27	13.3	20.0	10.0	15.1								
Properties													
M_{nx}/Ω_b | $\phi_b M_{nx}$	kip-ft	18.3	27.5	13.1	19.7	31.2	47.0	25.9	39.0	22.8	34.3	19.3	29.1
M_{ny}/Ω_b | $\phi_b M_{ny}$	kip-ft	13.5	20.3	9.57	14.4	18.6	27.9	15.5	23.3	13.7	20.5	11.5	17.3
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		632		478		833		736		673		597	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		319		241		260		230		210		186	
r_{mx}/r_{my}		1.41		1.41		1.79		1.79		1.79		1.79	
r_{my}, in.		1.63		1.66		1.12		1.17		1.19		1.22	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS6–HSS5

Table IV-1A (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS6x3x				HSS5x4x							
		3⁄16		1⁄8		1⁄2		3⁄8		5⁄16		1⁄4	
t_{des}, in,		0.174		0.116		0.465		0.349		0.291		0.233	
Steel, lb/ft		10.7		7.31		25.0		19.8		17.0		13.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	98.7	148	77.1	116	208	313	164	247	143	214	123	185
	1	97.9	147	76.5	115	207	311	163	245	142	213	123	184
	2	95.7	144	74.8	112	204	307	161	242	140	210	121	181
	3	92.2	138	71.9	108	199	299	157	237	137	205	118	177
	4	87.4	131	68.1	102	192	289	153	229	132	198	114	172
	5	81.7	123	63.5	95.3	184	276	146	220	127	190	110	165
	6	75.2	113	58.3	87.5	174	262	139	209	120	180	104	156
	7	68.1	102	52.8	79.1	163	246	131	197	113	170	98.3	147
	8	60.8	91.2	47.0	70.4	152	228	123	184	105	159	91.7	138
	9	53.5	80.2	41.2	61.7	139	210	113	170	97.8	147	84.8	127
	10	46.3	69.5	35.5	53.3	127	191	104	156	89.9	135	77.7	117
	11	39.5	59.3	30.2	45.3	114	172	94.5	142	81.9	123	70.5	106
	12	33.3	49.9	25.4	38.0	102	154	85.1	128	73.9	111	63.4	95.1
	13	28.4	42.5	21.6	32.4	90.3	136	76.0	114	66.2	99.5	56.5	84.8
	14	24.4	36.7	18.6	28.0	78.9	119	67.2	101	58.7	88.2	49.9	74.8
	15	21.3	31.9	16.2	24.3	68.7	103	58.7	88.3	51.5	77.4	43.9	66.0
	16	18.7	28.1	14.3	21.4	60.4	90.8	51.6	77.6	45.3	68.0	38.6	58.0
	17	16.6	24.9	12.6	19.0	53.5	80.4	45.7	68.7	40.1	60.3	34.2	51.4
	18	14.8	22.2	11.3	16.9	47.7	71.7	40.8	61.3	35.8	53.7	30.5	45.8
	19	13.3	19.9	10.1	15.2	42.8	64.4	36.6	55.0	32.1	48.2	27.4	41.1
	20	12.0	18.0	9.13	13.7	38.7	58.1	33.0	49.7	29.0	43.5	24.7	37.1
	21			8.28	12.4	35.1	52.7	30.0	45.0	26.3	39.5	22.4	33.7
	22					31.9	48.0	27.3	41.0	23.9	36.0	20.4	30.7
	23					29.2	43.9	25.0	37.5	21.9	32.9	18.7	28.1
	24					26.8	40.4	22.9	34.5	20.1	30.2	17.2	25.8
	25							21.1	31.8	18.5	27.9	15.8	23.8
	26											14.6	22.0
Properties													
M_{nx}/Ω_b / $\phi_b M_{nx}$	kip-ft	15.4	23.1	11.0	16.6	28.2	42.3	23.5	35.3	20.6	31.0	17.4	26.2
M_{ny}/Ω_b / $\phi_b M_{ny}$	kip-ft	9.15	13.8	6.50	9.76	24.0	36.1	20.0	30.0	17.5	26.3	14.8	22.2
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		507		389		658		579		527		468	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		157		120		456		400		363		322	
r_{mx}/r_{my}		1.80		1.80		1.20		1.20		1.20		1.21	
r_{my}, in.		1.25		1.27		1.46		1.52		1.54		1.57	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS5

Table IV-1A (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS5x4x				HSS5x3x							
		3⁄16		⅛		½		⅜		5⁄16		¼	
t_{des}, in.		**0.174**		**0.116**		**0.465**		**0.349**		**0.291**		**0.233**	
Steel, lb/ft		**10.7**		**7.31**		**21.6**		**17.3**		**14.8**		**12.2**	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	102	153	80.5	121	180	271	143	215	123	184	104	156
	1	102	152	80.1	120	179	269	142	213	122	183	103	154
	2	100	150	79.0	119	174	261	139	208	119	179	100	151
	3	98.0	147	77.2	116	166	250	133	200	115	172	96.5	145
	4	95.0	142	74.7	112	156	235	126	189	109	163	91.3	137
	5	91.2	137	71.6	107	144	217	117	176	101	152	84.9	127
	6	86.7	130	67.9	102	131	197	107	161	93.1	140	77.8	117
	7	81.8	123	63.9	95.9	117	175	96.2	145	84.2	127	70.1	105
	8	76.4	115	59.5	89.3	102	154	85.2	128	75.0	113	62.7	94.2
	9	70.8	106	54.9	82.4	87.9	132	74.2	112	65.8	99.0	55.2	83.0
	10	64.9	97.4	50.2	75.4	74.3	112	63.7	95.7	56.9	85.5	48.0	72.1
	11	59.0	88.6	45.5	68.2	61.7	92.7	53.6	80.5	48.4	72.7	41.0	61.7
	12	53.2	79.8	40.8	61.2	51.8	77.9	45.0	67.7	40.7	61.1	34.6	52.0
	13	47.5	71.3	36.3	54.4	44.2	66.4	38.4	57.7	34.7	52.1	29.5	44.3
	14	42.0	63.1	31.9	47.9	38.1	57.2	33.1	49.7	29.9	44.9	25.4	38.2
	15	36.8	55.2	27.8	41.8	33.2	49.9	28.8	43.3	26.0	39.1	22.1	33.3
	16	32.3	48.5	24.5	36.7	29.2	43.8	25.3	38.1	22.9	34.4	19.5	29.2
	17	28.6	43.0	21.7	32.5	25.8	38.8	22.4	33.7	20.3	30.5	17.2	25.9
	18	25.6	38.3	19.3	29.0	23.0	34.6	20.0	30.1	18.1	27.2	15.4	23.1
	19	22.9	34.4	17.4	26.0			18.0	27.0	16.2	24.4	13.8	20.7
	20	20.7	31.0	15.7	23.5								
	21	18.8	28.2	14.2	21.3								
	22	17.1	25.7	12.9	19.4								
	23	15.7	23.5	11.8	17.8								
	24	14.4	21.6	10.9	16.3								
	25	13.2	19.9	10.0	15.0								
	26	12.2	18.4	9.27	13.9								
	27			8.59	12.9								
Properties													
M_{nx}/Ω_b **$\phi_b M_{nx}$**	**kip-ft**	13.8	20.8	9.90	14.9	22.7	34.1	19.1	28.7	16.9	25.4	14.4	21.6
M_{ny}/Ω_b **$\phi_b M_{ny}$**	**kip-ft**	11.7	17.6	8.36	12.6	15.5	23.4	13.1	19.7	11.6	17.5	9.87	14.8
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		397		300		503		450		413		367	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		272		206		214		192		176		157	
r_{mx}/r_{my}		1.21		1.21		1.53		1.53		1.53		1.53	
r_{my}, in.		1.60		1.62		1.09		1.14		1.17		1.19	

ASD	**LRFD**
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS5–HSS4

Table IV-1A (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS5x3x				HSS5x2½x						HSS4x3x	
		3⁄16		⅛		¼		3⁄16		⅛		⅜	
t_{des}, in,		0.174		0.116		0.233		0.174		0.116		0.349	
Steel, lb/ft		9.42		6.46		11.4		8.78		6.03		14.7	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	85.4	128	66.7	100	94.1	141	77.2	116	59.6	89.4	122	184
	1	84.8	127	66.1	99.2	93.0	140	76.4	115	59.0	88.4	121	182
	2	82.8	124	64.6	96.9	90.1	135	73.9	111	57.1	85.6	118	178
	3	79.7	119	62.1	93.2	85.5	129	69.9	105	54.0	81.0	113	170
	4	75.4	113	58.8	88.2	79.4	119	64.7	97.1	50.0	75.0	107	161
	5	70.3	106	54.8	82.1	72.2	109	58.6	87.9	45.3	68.0	98.9	149
	6	64.6	96.9	50.2	75.3	64.3	96.6	51.9	77.8	40.2	60.3	90.0	135
	7	58.4	87.6	45.3	68.0	56.1	84.3	45.0	67.4	34.8	52.3	80.6	121
	8	51.9	77.9	40.3	60.4	47.9	71.9	38.1	57.1	29.6	44.4	70.9	107
	9	45.5	68.3	35.2	52.8	40.0	60.1	31.8	47.8	24.5	36.8	61.3	92.1
	10	39.3	58.9	30.3	45.5	32.7	49.2	26.2	39.3	20.0	30.0	52.1	78.3
	11	33.4	50.0	25.7	38.5	27.0	40.6	21.6	32.5	16.5	24.8	43.5	65.3
	12	28.0	42.0	21.6	32.4	22.7	34.1	18.2	27.3	13.9	20.8	36.5	54.9
	13	23.9	35.8	18.4	27.6	19.4	29.1	15.5	23.3	11.8	17.7	31.1	46.8
	14	20.6	30.9	15.9	23.8	16.7	25.1	13.4	20.1	10.2	15.3	26.8	40.3
	15	17.9	26.9	13.8	20.7	14.5	21.9	11.6	17.5	8.88	13.3	23.4	35.1
	16	15.8	23.6	12.1	18.2	12.8	19.2	10.2	15.4	7.80	11.7	20.5	30.9
	17	14.0	20.9	10.8	16.1			9.06	13.6	6.91	10.4	18.2	27.4
	18	12.5	18.7	9.59	14.4							16.2	24.4
	19	11.2	16.8	8.61	12.9								
	20	10.1	15.1	7.77	11.7								

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	11.5	17.2	8.26	12.4	12.8	19.3	10.3	15.4	7.43	11.2	13.3	19.9
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	7.86	11.8	5.62	8.45	7.66	11.5	6.13	9.21	4.40	6.61	10.7	16.2
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}			313		242		317		271		212		248	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}			132		102		99.3		84.3		65.6		153	
r_{mx}/r_{my}			1.54		1.54		1.79		1.79		1.80		1.27	
r_{my}, in.			1.22		1.25		0.999		1.02		1.05		1.11	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS4

Table IV-1A (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS4x3x								HSS4x2½x			
		5⁄16		¼		3⁄16		⅛		⅜		5⁄16	
t_{des}, in,		0.291		0.233		0.174		0.116		0.349		0.291	
Steel, lb/ft		12.7		10.5		8.15		5.61		13.4		11.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	105	158	87.9	132	72.4	109	56.2	84.3	112	168	96.7	145
	1	105	157	87.2	131	71.8	108	55.8	83.6	111	166	95.6	144
	2	102	153	85.0	128	70.1	105	54.4	81.6	107	160	92.3	139
	3	97.9	147	81.5	122	67.3	101	52.3	78.4	100	151	87.0	131
	4	92.4	139	76.9	116	63.6	95.4	49.4	74.1	91.8	138	80.1	120
	5	85.8	129	71.6	108	59.2	88.7	46.0	68.9	82.2	123	72.1	108
	6	78.3	118	65.7	98.8	54.1	81.2	42.1	63.1	71.7	108	63.4	95.2
	7	70.4	106	59.4	89.2	48.7	73.1	37.9	56.8	61.0	91.7	54.4	81.8
	8	62.2	93.4	52.8	79.4	43.2	64.7	33.6	50.3	50.7	76.2	45.6	68.6
	9	54.0	81.2	46.2	69.5	37.6	56.4	29.3	43.9	41.0	61.6	37.3	56.1
	10	46.2	69.4	39.8	59.9	32.3	48.4	25.1	37.7	33.2	49.9	30.2	45.4
	11	38.8	58.3	33.8	50.8	27.3	41.0	21.2	31.7	27.4	41.2	25.0	37.6
	12	32.6	49.0	28.4	42.7	23.0	34.6	17.8	26.7	23.0	34.6	21.0	31.6
	13	27.8	41.7	24.2	36.3	19.6	29.4	15.2	22.7	19.6	29.5	17.9	26.9
	14	23.9	36.0	20.9	31.3	16.9	25.4	13.1	19.6	16.9	25.4	15.4	23.2
	15	20.9	31.3	18.2	27.3	14.7	22.1	11.4	17.1	14.7	22.2	13.4	20.2
	16	18.3	27.5	16.0	24.0	12.9	19.4	10.0	15.0				
	17	16.2	24.4	14.1	21.3	11.5	17.2	8.86	13.3				
	18	14.5	21.8	12.6	19.0	10.2	15.4	7.90	11.9				
	19			11.3	17.0	9.17	13.8	7.09	10.6				
	20							6.40	9.60				

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	11.8	17.7	10.1	15.1	8.08	12.1	5.86	8.80	11.6	17.4	10.3	15.5
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	9.56	14.4	8.17	12.3	6.54	9.83	4.71	7.08	8.18	12.3	7.33	11.0
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}			229		205		174		136		210		195	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}			142		127		108		84.1		95.5		88.8	
r_{mx}/r_{my}			1.27		1.27		1.27		1.27		1.48		1.48	
r_{my}, in.			1.13		1.16		1.19		1.21		0.922		0.947	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS4

Table IV-1A (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape	HSS4x2½x						HSS4x2x					
	¼		$^3\!/_{16}$		⅛		⅜		$^5\!/_{16}$		¼	
t_{des}, in,	0.233		0.174		0.116		0.349		0.291		0.233	
Steel, lb/ft	9.66		7.51		5.18		12.2		10.6		8.81	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y												
0	79.9	120	64.8	97.2	50.0	75.0	101	153	88	132	73.1	110
1	79.1	119	64.1	96.1	49.4	74.2	99.5	150	86.4	130	71.8	108
2	76.5	115	61.9	92.9	47.8	71.7	93.8	141	81.7	123	68.2	102
3	72.3	109	58.5	87.7	45.2	67.8	84.9	128	74.5	112	62.5	93.9
4	66.9	101	54.0	81.0	41.8	62.7	73.9	111	65.5	98.4	55.3	83.2
5	60.5	91.0	48.7	73.0	37.8	56.7	61.9	93.0	55.4	83.3	47.3	71.2
6	53.6	80.5	42.9	64.4	33.4	50.1	49.7	74.8	45.2	67.9	39.1	58.8
7	46.4	69.7	37.0	55.5	28.8	43.3	38.4	57.7	35.5	53.4	31.2	46.9
8	39.2	59.0	31.4	47.2	24.4	36.6	29.4	44.2	27.3	41.0	24.1	36.3
9	32.5	48.8	26.2	39.4	20.1	30.2	23.2	34.9	21.5	32.4	19.1	28.7
10	26.4	39.7	21.5	32.3	16.3	24.5	18.8	28.3	17.4	26.2	15.5	23.2
11	21.8	32.8	17.7	26.7	13.5	20.3	15.5	23.4	14.4	21.7	12.8	19.2
12	18.3	27.5	14.9	22.4	11.4	17.0	13.1	19.6	12.1	18.2	10.7	16.1
13	15.6	23.5	12.7	19.1	9.67	14.5					9.15	13.7
14	13.5	20.2	10.9	16.5	8.34	12.5						
15	11.7	17.6	9.54	14.3	7.27	10.9						
16	10.3	15.5	8.38	12.6	6.39	9.58						
17					5.66	8.49						
Properties												
M_{nx}/Ω_b / $\phi_b M_{nx}$, kip-ft	8.90	13.4	7.17	10.8	5.20	7.82	9.86	14.8	8.88	13.4	7.70	11.6
M_{ny}/Ω_b / $\phi_b M_{ny}$, kip-ft	6.31	9.48	5.08	7.64	3.66	5.50	5.87	8.82	5.31	7.98	4.61	6.92
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}	175		150		119		172		161		146	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}	80.0		68.3		53.7		53.3		50.2		45.6	
r_{mx}/r_{my}	1.48		1.48		1.49		1.80		1.79		1.79	
r_{my}, in.	0.973		0.999		1.03		0.729		0.754		0.779	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS4

Table IV-1A (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS4x2x			
		3⁄16		1⁄8	
t_{des}, in,		**0.174**		**0.116**	
Steel, lb/ft		**6.87**		**4.75**	
Design		**P_n/Ω_c**	**$\phi_c P_n$**	**P_n/Ω_c**	**$\phi_c P_n$**
		ASD	**LRFD**	**ASD**	**LRFD**
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	57.5	86.2	43.8	65.7
	1	56.5	84.7	43.1	64.6
	2	53.5	80.3	40.9	61.4
	3	49.0	73.5	37.6	56.4
	4	43.6	65.5	33.4	50.1
	5	37.7	56.6	28.6	43.0
	6	31.5	47.3	23.8	35.6
	7	25.5	38.3	19.0	28.6
	8	19.9	29.9	14.8	22.2
	9	15.7	23.7	11.7	17.5
	10	12.8	19.2	9.46	14.2
	11	10.5	15.8	7.82	11.7
	12	8.86	13.3	6.57	9.85
	13	7.55	11.3	5.60	8.39

Properties

M_{nx}/Ω_b	**$\phi_b M_{nx}$**	**kip-ft**	6.24	9.38	4.56	6.86
M_{ny}/Ω_b	**$\phi_b M_{ny}$**	**kip-ft**	3.72	5.60	2.71	4.07
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}			125		100	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}			39.1		31.1	
r_{mx}/r_{my}			1.79		1.79	
r_{my}, in.			0.804		0.830	

ASD	**LRFD**
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

5

COMPOSITE
HSS20–HSS16

Table IV-1B
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS20x12x								HSS16x12x			
		5⁄8		1⁄2		3⁄8		5⁄16		5⁄8		1⁄2	
t_{des}, in.		0.581		0.465		0.349		0.291		0.581		0.465	
Steel, lb/ft		127		103		78.5		65.9		110		89.7	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	1310	1960	1160	1730	1000	1500	891	1340	1100	1650	969	1450
	1	1310	1960	1160	1730	1000	1500	890	1340	1100	1650	969	1450
	2	1310	1960	1150	1730	999	1500	889	1330	1100	1640	967	1450
	3	1300	1950	1150	1730	996	1490	886	1330	1090	1640	965	1450
	4	1300	1950	1150	1720	992	1490	883	1320	1090	1630	961	1440
	5	1290	1940	1140	1710	987	1480	879	1320	1080	1630	956	1430
	6	1280	1930	1130	1700	981	1470	873	1310	1080	1620	951	1430
	7	1280	1910	1130	1690	974	1460	867	1300	1070	1610	944	1420
	8	1270	1900	1120	1680	966	1450	860	1290	1060	1590	937	1410
	9	1260	1880	1110	1660	958	1440	852	1280	1050	1580	928	1390
	10	1240	1870	1100	1650	948	1420	843	1260	1040	1560	919	1380
	11	1230	1850	1090	1630	937	1410	834	1250	1030	1550	909	1360
	12	1220	1820	1070	1610	925	1390	823	1230	1020	1530	898	1350
	13	1200	1800	1060	1590	913	1370	812	1220	1010	1510	886	1330
	14	1180	1780	1040	1570	899	1350	800	1200	992	1490	874	1310
	15	1170	1750	1030	1540	885	1330	787	1180	977	1470	860	1290
	16	1150	1720	1010	1520	870	1310	774	1160	962	1440	846	1270
	17	1130	1700	995	1490	855	1280	760	1140	945	1420	832	1250
	18	1110	1670	977	1470	838	1260	746	1120	928	1390	817	1220
	19	1090	1630	958	1440	822	1230	731	1100	911	1370	801	1200
	20	1070	1600	939	1410	804	1210	715	1070	892	1340	784	1180
	21	1050	1570	919	1380	787	1180	699	1050	873	1310	768	1150
	22	1020	1540	899	1350	768	1150	683	1020	854	1280	750	1130
	23	1000	1500	878	1320	750	1120	666	999	834	1250	733	1100
	24	978	1470	857	1290	731	1100	649	974	814	1220	715	1070
	25	954	1430	836	1250	711	1070	632	948	794	1190	696	1040
	26	929	1390	814	1220	692	1040	615	922	773	1160	678	1020
	27	905	1360	792	1190	672	1010	597	895	752	1130	659	989
	28	880	1320	769	1150	652	978	579	869	731	1100	640	960
	29	855	1280	747	1120	632	948	561	842	709	1060	621	932
	30	830	1240	724	1090	612	918	543	815	688	1030	602	903
	32	779	1170	679	1020	572	858	508	761	645	968	564	846
	34	729	1090	634	952	532	798	472	708	602	903	526	789
	36	679	1020	590	885	493	740	437	656	560	840	488	733
	38	630	945	546	819	455	682	403	605	518	778	452	678
	40	582	874	504	756	418	626	370	555	478	717	416	624

Properties

			5⁄8 ASD	5⁄8 LRFD	1⁄2 ASD	1⁄2 LRFD	3⁄8 ASD	3⁄8 LRFD	5⁄16 ASD	5⁄16 LRFD	5⁄8 ASD	5⁄8 LRFD	1⁄2 ASD	1⁄2 LRFD
M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	647	972	540	812	425	639	367	551	457	687	381	573
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	439	660	362	545	285	428	235	353	367	551	306	460

	HSS20x12x5⁄8	HSS20x12x1⁄2	HSS20x12x3⁄8	HSS20x12x5⁄16	HSS16x12x5⁄8	HSS16x12x1⁄2
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}	74400	64200	53100	47100	41400	36000
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}	31200	26800	22100	19600	25500	22100
r_{mx}/r_{my}	1.54	1.55	1.55	1.55	1.27	1.28
r_{my}, in.	4.93	4.99	5.04	5.07	4.80	4.86

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_c = 2.00$	$\phi_c = 0.75$

5

COMPOSITE
HSS16

Table IV-1B (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS16x12x				HSS16x8x							
		⅜		5⁄16		⅝		½		⅜		5⁄16	
t_{des}, in.		**0.349**		**0.291**		**0.581**		**0.465**		**0.349**		**0.291**	
Steel, lb/ft		**68.3**		**57.4**		**93.3**		**76.1**		**58.1**		**48.9**	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	835	1250	766	1150	857	1290	749	1120	637	956	578	867
	1	834	1250	766	1150	857	1280	748	1120	636	955	577	866
	2	833	1250	765	1150	854	1280	745	1120	634	951	575	863
	3	831	1250	762	1140	849	1270	741	1110	631	946	572	858
	4	827	1240	759	1140	842	1260	735	1100	626	939	567	851
	5	823	1230	756	1130	834	1250	728	1090	619	929	561	842
	6	818	1230	751	1130	824	1240	719	1080	612	917	554	831
	7	812	1220	745	1120	812	1220	709	1060	603	904	546	819
	8	806	1210	739	1110	798	1200	697	1050	592	889	536	805
	9	798	1200	732	1100	783	1170	684	1030	581	872	526	789
	10	790	1180	724	1090	766	1150	670	1000	569	853	514	771
	11	781	1170	715	1070	749	1120	654	981	555	833	502	753
	12	771	1160	706	1060	729	1090	638	957	541	811	489	733
	13	760	1140	696	1040	709	1060	620	930	526	789	475	712
	14	749	1120	685	1030	688	1030	602	903	510	765	460	690
	15	737	1110	674	1010	666	999	583	874	493	740	445	667
	16	725	1090	662	993	643	965	563	845	476	714	429	643
	17	711	1070	649	974	620	930	543	814	459	688	413	619
	18	698	1050	636	955	596	894	522	783	441	661	396	594
	19	684	1030	623	935	572	857	501	752	423	634	379	569
	20	669	1000	609	914	547	821	480	720	404	607	363	544
	21	654	981	595	893	523	784	458	688	386	579	346	519
	22	639	958	581	871	498	747	437	655	368	551	329	493
	23	623	935	566	849	473	710	416	623	349	524	312	468
	24	607	911	551	826	449	674	395	592	331	497	295	443
	25	591	886	536	803	425	637	374	560	313	470	279	419
	26	574	862	520	780	401	602	353	529	295	443	263	394
	27	558	837	505	757	378	567	333	499	278	417	247	371
	28	541	812	489	733	356	534	313	470	261	392	232	348
	29	524	786	473	710	336	505	294	441	245	367	216	325
	30	508	761	457	686	317	477	275	412	229	343	202	303
	32	474	711	426	639	280	421	241	362	201	301	178	267
	34	441	661	395	593	248	373	214	321	178	267	158	236
	36	408	612	365	547	221	333	191	286	159	238	140	211
	38	376	563	335	502	199	299	171	257	142	214	126	189
	40	345	517	306	459	179	269	154	232	129	193	114	171
Properties													
M_{nx}/Ω_b | **$\phi_b M_{nx}$**	**kip-ft**	301	453	258	388	354	532	297	446	236	355	203	306
M_{ny}/Ω_b | **$\phi_b M_{ny}$**	**kip-ft**	240	361	205	308	210	315	175	263	138	207	118	177
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		29700		26400		29700		26400		22100		19600	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		18200		16100		9200		8110		6760		5980	
r_{mx}/r_{my}		1.28		1.28		1.80		1.80		1.81		1.81	
r_{my}, in.		4.91		4.94		3.27		3.32		3.37		3.40	

ASD	**LRFD**
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

COMPOSITE
HSS16–HSS14

Table IV-1B (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS16x8x		HSS14x10x									
		¼		⅝		½		⅜		5⁄16		¼	
t_{des}, in,		0.233		0.581		0.465		0.349		0.291		0.233	
Steel, lb/ft		39.4		93.3		76.1		58.1		48.9		39.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	501	752	883	1320	774	1160	663	994	603	905	543	815
	1	500	751	882	1320	773	1160	662	993	603	904	543	814
	2	499	748	880	1320	772	1160	661	991	601	902	541	812
	3	496	744	877	1320	769	1150	658	987	599	899	539	808
	4	492	738	872	1310	765	1150	655	982	596	894	536	804
	5	487	730	867	1300	760	1140	650	975	592	887	532	798
	6	480	720	859	1290	754	1130	645	967	586	880	527	791
	7	473	709	851	1280	746	1120	638	957	580	870	521	782
	8	465	697	842	1260	738	1110	631	946	573	860	515	772
	9	456	683	831	1250	729	1090	623	934	566	849	508	762
	10	445	668	819	1230	719	1080	614	920	557	836	500	750
	11	435	652	806	1210	708	1060	604	906	548	822	491	737
	12	423	634	793	1190	696	1040	593	890	538	807	482	723
	13	411	616	778	1170	683	1020	582	873	528	791	472	708
	14	398	597	762	1140	669	1000	570	855	516	774	462	692
	15	385	577	746	1120	655	982	557	836	505	757	451	676
	16	371	556	729	1090	640	960	544	816	492	738	439	659
	17	357	535	711	1070	625	937	530	796	480	719	427	641
	18	342	513	693	1040	609	913	516	774	466	700	415	623
	19	328	492	674	1010	592	888	502	753	453	679	402	604
	20	313	469	654	981	575	863	487	730	439	659	390	584
	21	298	447	634	951	558	837	472	708	425	637	377	565
	22	284	425	614	921	540	811	456	685	411	616	363	545
	23	269	403	594	891	523	784	441	661	396	594	350	525
	24	254	382	573	860	505	757	425	638	382	573	337	505
	25	240	360	553	829	487	730	409	614	367	551	323	485
	26	226	339	532	798	469	703	394	590	353	529	310	465
	27	213	319	511	767	450	676	378	567	338	507	297	445
	28	199	299	490	736	432	649	362	543	324	485	283	425
	29	186	279	470	705	415	622	347	520	309	464	270	405
	30	174	261	450	674	397	595	331	497	295	443	257	386
	32	153	229	410	614	362	543	301	451	267	401	232	348
	34	135	203	371	557	328	492	272	408	241	361	208	312
	36	121	181	333	500	295	443	244	365	215	322	185	278
	38	108	162	299	449	265	397	219	328	193	289	166	250
	40	97.7	147	270	405	239	359	197	296	174	261	150	225
Properties													
M_{nx}/Ω_b | $\phi_b M_{nx}$	kip-ft	169	254	329	494	276	414	218	328	188	282	156	234
M_{ny}/Ω_b | $\phi_b M_{ny}$	kip-ft	94.8	142	255	384	214	321	168	253	144	217	118	177
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		16900		25000		22200		18400		16300		14000	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		5130		14200		12600		10400		9150		7890	
r_{mx}/r_{my}		1.82		1.33		1.33		1.33		1.33		1.33	
r_{my}, in.		3.42		3.98		4.04		4.09		4.12		4.14	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

COMPOSITE
HSS12

Table IV-1B (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS12x10x								HSS12x8x			
		½		⅜		5⁄16		¼		⅝		½	
t_{des}, in.		0.465		0.349		0.291		0.233		0.581		0.465	
Steel, lb/ft		69.3		53.0		44.6		36.0		76.3		62.5	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	688	1030	588	882	533	800	481	722	682	1020	596	894
	1	687	1030	588	882	533	799	481	721	681	1020	595	893
	2	686	1030	586	880	532	797	479	719	679	1020	593	890
	3	683	1020	584	876	530	794	477	716	675	1010	590	885
	4	680	1020	581	871	527	790	475	712	669	1000	585	878
	5	675	1010	577	865	523	784	471	707	662	993	579	868
	6	669	1000	572	858	518	777	467	700	654	980	572	857
	7	663	994	566	849	513	769	462	693	644	966	563	845
	8	655	983	559	839	507	760	456	684	633	949	553	830
	9	647	970	552	828	500	750	450	674	620	930	543	814
	10	638	956	544	816	492	739	442	664	606	910	531	796
	11	627	941	535	803	484	726	435	652	592	887	518	777
	12	617	925	526	788	475	713	426	640	576	864	505	757
	13	605	907	515	773	466	699	418	626	559	839	490	735
	14	593	889	505	757	456	684	408	612	542	813	475	713
	15	580	870	493	740	445	668	399	598	524	785	460	689
	16	566	849	482	722	435	652	388	582	505	757	443	665
	17	552	828	469	704	423	635	378	567	486	729	427	640
	18	538	807	457	685	412	617	367	550	466	699	410	615
	19	523	784	444	665	400	599	356	533	446	669	393	589
	20	508	761	430	645	387	581	344	516	426	640	375	563
	21	492	738	417	625	375	562	333	499	406	609	358	537
	22	476	714	403	604	362	543	321	481	386	579	341	511
	23	460	690	389	584	349	524	309	464	366	550	323	485
	24	444	666	375	563	336	505	297	446	347	520	306	459
	25	428	642	361	541	323	485	285	428	327	491	289	434
	26	411	617	347	520	311	466	273	410	308	463	273	409
	27	395	593	333	499	298	446	261	392	292	438	257	385
	28	379	568	319	478	285	427	250	374	275	413	241	361
	29	363	544	305	457	272	408	238	357	259	389	225	338
	30	347	520	291	437	260	389	227	340	243	366	210	316
	32	316	474	264	396	235	353	204	306	214	321	185	277
	34	286	428	238	358	211	317	183	274	189	285	164	246
	36	256	384	213	320	189	283	163	244	169	254	146	219
	38	230	345	191	287	169	254	146	219	152	228	131	197
	40	208	311	173	259	153	229	132	198	137	206	118	178

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	217	327	173	259	148	223	123	185	222	334	187	282
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	190	285	150	226	129	193	106	160	164	247	138	208
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}			14800		12400		11000		9460		13900		12300	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}			10900		9070		8030		6930		7000		6220	
r_{mx}/r_{my}			1.17		1.17		1.17		1.17		1.41		1.41	
r_{my}, in.			3.96		4.01		4.04		4.07		3.16		3.21	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS12

Table IV-1B (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS12x8x				HSS12x6x							
		⅜		¼		⅝		½		⅜		5⁄16	
t_{des}, in,		**0.349**		**0.233**		**0.581**		**0.465**		**0.349**		**0.291**	
Steel, lb/ft		**47.9**		**32.6**		**67.8**		**55.7**		**42.8**		**36.1**	
Design		**P_n/Ω_c**	**$\phi_c P_n$**	**P_n/Ω_c**	**$\phi_c P_n$**	**P_n/Ω_c**	**$\phi_c P_n$**	**P_n/Ω_c**	**$\phi_c P_n$**	**P_n/Ω_c**	**$\phi_c P_n$**	**P_n/Ω_c**	**$\phi_c P_n$**
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	505	758	409	613	578	867	501	752	422	633	379	569
	1	505	757	408	612	577	866	500	751	421	632	379	568
	2	503	754	407	610	574	860	497	746	419	628	376	564
	3	500	750	404	606	568	852	492	739	415	622	373	559
	4	496	744	401	601	560	840	486	728	409	613	367	551
	5	491	736	396	594	550	824	477	715	402	603	361	541
	6	484	727	391	586	537	806	467	700	393	590	353	530
	7	477	716	385	577	523	785	455	682	383	575	344	516
	8	469	703	377	566	507	761	441	662	372	558	334	501
	9	460	689	370	554	490	735	426	640	360	539	323	484
	10	450	674	361	542	471	707	410	616	346	519	311	466
	11	439	658	352	528	452	677	394	590	332	498	298	447
	12	427	641	342	513	431	646	376	564	317	476	285	427
	13	415	622	332	497	410	616	358	536	302	453	271	407
	14	402	603	321	481	390	586	339	508	286	430	257	385
	15	389	583	309	464	370	556	320	479	270	406	243	364
	16	375	562	298	447	349	525	300	451	254	381	228	342
	17	361	541	286	429	329	494	281	422	238	357	214	320
	18	346	519	274	411	308	463	262	393	222	333	199	299
	19	332	497	261	392	288	433	243	365	207	310	185	278
	20	317	475	249	374	268	403	226	339	191	287	171	257
	21	302	453	237	355	248	373	210	316	176	265	158	237
	22	287	431	225	337	229	345	195	293	162	243	145	217
	23	273	409	212	319	211	317	180	270	148	222	132	199
	24	258	387	200	301	194	291	165	248	136	204	122	182
	25	244	366	189	283	178	268	152	229	125	188	112	168
	26	230	345	177	266	165	248	141	211	116	174	104	155
	27	216	324	166	249	153	230	130	196	107	161	961	144
	28	203	304	155	232	142	214	121	182	99.9	150	89.3	134
	29	189	284	144	216	133	199	113	170	93.1	140	83.3	125
	30	177	265	135	202	124	186	106	159	87.0	130	77.8	117
	32	155	233	118	178	109	164	92.9	140	76.5	115	68.4	103
	34	138	206	105	157	96.4	145	82.2	124	67.7	102	60.6	90.9
	36	123	184	93.6	140	86.0	129	73.4	110	60.4	90.6	54.0	81.1
	38	110	165	84.0	126	77.2	116	65.8	99.0	54.2	81.3	48.5	72.8
	40	99.4	149	75.8	114			59.4	89.3	48.9	73.4	43.8	65.7
Properties													
M_{nx}/Ω_b **$\phi_b M_{nx}$**	**kip-ft**	149	225	107	161	184	277	157	235	125	188	108	163
M_{ny}/Ω_b **$\phi_b M_{ny}$**	**kip-ft**	110	165	77.8	117	110	165	92.8	140	74.1	111	63.7	95.8
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		10400		7970		10900		9730		8350		7500	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		5220		3980		3410		3020		2570		2300	
r_{mx}/r_{my}		1.41		1.42		1.79		1.79		1.80		1.81	
r_{my}, in.		3.27		3.32		2.39		2.44		2.49		2.52	

ASD	**LRFD**
Ω_b = 1.67	**ϕ_b = 0.90**
Ω_c = 2.00	**ϕ_c = 0.75**

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS12–HSS10

Table IV-1B (continued)

Available Strength in Axial Compression, kips

Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS12x6x				HSS10x8x							
		¼		3⁄16		⅝		½		⅜		5⁄16	
t_{des}, in,		0.233		0.174		0.581		0.465		0.349		0.291	
Steel, lb/ft		29.2		22.2		67.8		55.7		42.8		36.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	336	504	281	421	595	893	518	778	439	659	396	594
	1	336	503	280	420	595	892	518	777	439	658	396	594
	2	334	500	278	418	592	889	516	774	437	656	394	592
	3	330	495	276	413	589	883	513	769	434	652	392	588
	4	325	488	272	407	584	876	509	763	431	646	389	583
	5	319	479	267	400	577	866	503	755	426	639	385	577
	6	312	469	260	391	570	855	497	745	421	631	379	569
	7	304	456	253	380	561	841	489	733	414	622	374	560
	8	295	442	246	368	551	826	480	720	407	611	367	550
	9	285	427	237	356	539	809	471	706	399	598	359	539
	10	274	411	228	342	527	791	460	690	390	585	351	527
	11	262	394	218	327	514	771	449	673	381	571	342	514
	12	250	375	208	312	500	750	437	655	370	555	333	500
	13	238	357	197	296	485	727	424	636	359	539	323	485
	14	225	338	186	280	469	704	410	616	348	522	313	469
	15	212	318	175	263	453	679	396	595	336	505	302	453
	16	199	298	164	247	436	654	382	573	324	486	291	437
	17	186	279	153	230	419	628	367	551	312	468	280	420
	18	173	260	143	214	401	602	352	528	299	449	268	402
	19	160	241	132	198	384	576	337	506	286	430	256	385
	20	148	222	122	183	366	549	322	483	273	410	245	367
	21	136	204	112	168	348	522	306	460	260	391	233	349
	22	124	187	102	153	330	496	291	437	248	371	221	332
	23	114	171	93.2	140	313	470	276	414	235	352	209	314
	24	105	157	85.6	128	297	446	261	391	222	333	198	297
	25	96.3	145	78.9	118	281	422	246	369	210	314	187	280
	26	89.1	134	72.9	109	266	399	232	347	197	296	175	263
	27	82.6	124	67.6	101	251	377	217	326	185	278	165	247
	28	76.8	115	62.9	94.3	236	354	204	305	174	260	154	231
	29	71.6	107	58.6	87.9	221	333	190	285	162	243	143	215
	30	66.9	100	54.8	82.2	207	311	177	266	151	227	134	201
	32	58.8	88.2	48.2	72.2	182	274	156	234	133	199	118	177
	34	52.1	78.1	42.7	64.0	161	242	138	207	118	177	104	157
	36	46.5	69.7	38.0	57.1	144	216	123	185	105	158	93.1	140
	38	41.7	62.6	34.1	51.2	129	194	111	166	94.3	141	83.5	125
	40	37.6	56.5	30.8	46.2	116	175	99.7	150	85.1	128	75.4	113
Properties													
M_{nx}/Ω_b $\phi_b M_{nx}$	kip-ft	90.6	136	71.2	107	167	250	141	212	113	169	97.2	146
M_{ny}/Ω_b $\phi_b M_{ny}$	kip-ft	52.7	79.2	39.8	59.8	141	212	120	180	95.2	143	82.0	123
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		6460		5340		8590		7640		6520		5780	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		1980		1620		5900		5240		4470		3960	
r_{mx}/r_{my}		1.81		1.82		1.21		1.21		1.21		1.21	
r_{my}, in.		2.54		2.57		3.09		3.14		3.19		3.22	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

5

COMPOSITE
HSS10

Table IV-1B (continued)

Available Strength in Axial Compression, kips

Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape	HSS10x8x				HSS10x6x							
	¼		3⁄16		5⁄8		½		3⁄8		5⁄16	
t_{des}, in.	0.233		0.174		0.581		0.465		0.349		0.291	
Steel, lb/ft	29.2		22.2		59.3		48.9		37.7		31.8	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	353	530	308	463	500	750	435	652	365	547	327	491
1	353	529	308	462	499	749	434	651	364	546	327	490
2	352	527	307	460	496	744	431	647	362	542	325	487
3	349	524	305	457	491	736	427	640	358	537	321	482
4	346	519	302	453	484	725	421	631	353	529	317	475
5	342	514	298	448	474	712	413	619	347	520	311	467
6	338	507	294	441	463	695	404	605	339	508	304	456
7	332	498	289	433	451	676	393	589	330	495	296	445
8	326	489	283	425	437	655	381	571	320	480	288	431
9	319	479	277	415	421	632	368	552	309	464	278	417
10	312	468	270	405	405	609	354	530	298	446	267	401
11	304	456	263	394	389	585	339	508	285	428	256	384
12	295	443	255	382	372	560	323	484	272	408	245	367
13	286	429	246	370	355	533	307	460	259	388	233	349
14	277	415	238	357	337	506	290	435	245	368	220	330
15	267	400	229	343	319	479	273	410	231	347	208	312
16	257	385	220	329	300	451	256	384	217	325	195	293
17	246	369	210	315	282	423	239	359	203	304	182	274
18	236	353	201	301	263	396	223	334	189	283	170	255
19	225	337	191	287	245	369	208	312	175	263	158	237
20	214	321	181	272	228	342	193	291	162	243	146	219
21	204	305	172	258	210	316	179	269	149	224	134	201
22	193	289	162	243	194	291	166	249	136	205	123	184
23	182	274	153	229	177	266	152	229	125	187	112	169
24	172	258	144	216	163	245	140	210	115	172	103	155
25	162	243	135	202	150	225	129	194	106	158	95.1	143
26	152	228	126	189	139	208	119	179	97.6	146	87.9	132
27	142	213	117	176	129	193	110	166	90.5	136	81.5	122
28	132	198	109	163	120	180	103	154	84.2	126	75.8	114
29	123	185	102	152	111	168	95.7	144	78.5	118	70.7	106
30	115	173	94.8	142	104	157	89.4	134	73.3	110	66.0	99.1
32	101	152	83.4	125	91.5	138	78.6	118	64.4	96.7	58.0	87.1
34	89.7	135	73.8	111	81.1	122	69.6	105	57.1	85.6	51.4	77.1
36	80.0	120	65.9	98.8	72.3	109	62.1	93.3	50.9	76.4	45.9	68.8
38	71.8	108	59.1	88.7	64.9	97.6	55.7	83.8	45.7	68.6	41.2	61.7
40	64.8	97.3	53.4	80.0					41.2	61.9	37.1	55.7
Properties												
M_{nx}/Ω_b / $\phi_b M_{nx}$, kip-ft	80.9	122	63.2	94.9	137	205	116	175	93.5	141	81.0	122
M_{ny}/Ω_b / $\phi_b M_{ny}$, kip-ft	68.1	102	52.8	79.3	93.5	140	79.5	119	63.6	95.5	54.7	82.3
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}	4980		4110		6700		5970		5150		4670	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}	3410		2800		2840		2540		2170		1950	
r_{mx}/r_{my}	1.21		1.21		1.54		1.53		1.54		1.55	
r_{my}, in.	3.25		3.28		2.34		2.39		2.44		2.47	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

5

COMPOSITE
HSS10

Table IV-1B (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F$_y$ = 50 ksi
f′$_c$ = 5 ksi

Shape		HSS10x6x				HSS10x5x							
		¼		3/16		3/8		5/16		¼		3/16	
t_{des}, in.		0.233		0.174		0.349		0.291		0.233		0.174	
Steel, lb/ft		25.8		19.6		35.1		29.7		24.1		18.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	290	434	250	375	327	490	293	439	258	386	221	331
	1	289	433	250	374	326	488	292	438	257	385	220	330
	2	287	431	248	372	323	484	289	434	255	382	218	327
	3	284	426	245	368	318	477	285	428	251	376	215	322
	4	280	420	242	362	312	468	280	419	246	369	210	316
	5	275	412	237	355	304	456	272	409	240	359	205	307
	6	269	403	231	347	295	442	264	396	232	348	198	297
	7	261	392	225	337	284	426	254	382	224	336	191	286
	8	253	380	217	326	272	408	244	366	214	321	182	273
	9	245	367	209	314	259	388	232	349	204	306	173	260
	10	235	353	201	301	245	368	220	330	193	290	163	245
	11	225	338	192	288	231	346	207	311	182	273	153	230
	12	215	322	183	274	216	324	194	291	170	255	143	215
	13	204	306	173	259	201	302	181	271	158	238	133	199
	14	193	289	163	244	186	279	168	251	147	220	122	184
	15	181	272	153	229	171	257	154	232	135	202	112	168
	16	170	255	143	214	157	235	141	212	123	185	102	153
	17	159	238	133	199	143	214	129	193	112	168	92.5	139
	18	148	221	123	185	129	194	117	175	101	152	83.1	125
	19	137	205	114	170	117	176	105	157	91.1	137	74.6	112
	20	126	189	104	156	106	159	94.5	142	82.2	123	67.3	101
	21	116	174	95.1	143	96.2	145	85.7	129	74.6	112	61.0	91.6
	22	106	158	86.7	130	87.6	132	78.1	117	68.0	102	55.6	83.4
	23	96.7	145	79.3	119	80.2	121	71.4	107	62.2	93.3	50.9	76.3
	24	88.8	133	72.8	109	73.6	111	65.6	98.4	57.1	85.7	46.7	70.1
	25	81.8	123	67.1	101	67.9	102	60.5	90.7	52.6	78.9	43.1	64.6
	26	75.7	113	62.1	93.1	62.7	94.3	55.9	83.9	48.7	73.0	39.8	59.7
	27	70.2	105	57.6	86.3	58.2	87.5	51.8	77.8	45.1	67.7	36.9	55.4
	28	65.2	97.8	53.5	80.3	54.1	81.3	48.2	72.3	42.0	62.9	34.3	51.5
	29	60.8	91.2	49.9	74.8	50.4	75.8	44.9	67.4	39.1	58.7	32.0	48.0
	30	56.8	85.2	46.6	69.9	47.1	70.8	42.0	63.0	36.5	54.8	29.9	44.9
	32	49.9	74.9	41.0	61.5	41.4	62.3	36.9	55.4	32.1	48.2	26.3	39.4
	34	44.2	66.4	36.3	54.4	36.7	55.2	32.7	49.0	28.5	42.7	23.3	34.9
	36	39.5	59.2	32.4	48.6								
	38	35.4	53.1	29.1	43.6								
	40	32.0	47.9	26.2	39.3								
Properties													
M_{nx}/Ω_b | $\phi_b M_{nx}$	kip-ft	67.8	102	53.2	79.9	83.7	126	72.7	109	60.9	91.5	48.0	72.1
M_{ny}/Ω_b | $\phi_b M_{ny}$	kip-ft	45.6	68.5	35.4	53.3	49.6	74.5	42.8	64.3	35.7	53.7	27.7	41.7
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		4010		3310		4430		4040		3520		2900	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		1680		1380		1370		1240		1080		884	
r_{mx}/r_{my}		1.54		1.55		1.80		1.81		1.81		1.81	
r_{my}, in.		2.49		2.52		2.05		2.07		2.10		2.13	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

5
COMPOSITE
HSS9

Table IV-1B (continued)

Available Strength in Axial Compression, kips

Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS9x7x											
		⅝		½		⅜		5⁄16		¼		3⁄16	
t_{des}, in.		0.581		0.465		0.349		0.291		0.233		0.174	
Steel, lb/ft		59.3		48.9		37.7		31.8		25.8		19.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	507	760	441	662	371	556	334	500	296	444	256	385
	1	506	759	441	661	370	556	333	500	295	443	256	384
	2	503	755	438	658	369	553	332	497	294	441	255	382
	3	499	749	435	652	366	549	329	494	292	438	253	379
	4	494	741	430	645	362	543	325	488	288	433	250	375
	5	487	730	424	636	357	535	321	481	284	427	246	369
	6	478	717	417	625	351	526	316	473	279	419	241	362
	7	468	702	408	613	344	516	309	464	274	411	236	354
	8	457	685	399	598	336	504	302	453	267	401	230	345
	9	445	667	388	583	327	491	294	442	260	390	224	336
	10	431	647	377	565	318	477	286	429	252	378	217	325
	11	417	625	365	547	308	462	277	415	244	366	209	314
	12	402	603	352	528	297	446	267	401	235	353	201	302
	13	386	579	338	507	286	429	257	385	226	339	193	290
	14	369	554	324	486	274	411	246	370	217	325	184	277
	15	353	531	310	464	262	393	236	353	207	310	176	264
	16	337	507	295	442	250	375	225	337	197	295	167	250
	17	321	483	280	420	238	356	213	320	187	280	158	237
	18	305	459	265	397	225	338	202	303	177	265	149	223
	19	289	435	250	375	213	319	191	286	166	250	140	210
	20	273	411	235	353	200	300	180	270	156	235	131	196
	21	257	387	220	331	188	282	169	253	147	220	122	183
	22	242	363	206	309	176	264	158	237	137	205	114	171
	23	226	340	192	288	164	246	147	221	127	191	105	158
	24	211	317	179	269	153	229	137	205	118	177	97.2	146
	25	196	295	167	251	141	212	127	190	109	164	89.6	134
	26	182	273	155	234	131	196	117	176	101	151	82.8	124
	27	169	253	144	217	121	182	109	163	93.5	140	76.8	115
	28	157	236	134	201	113	169	101	151	87.0	130	71.4	107
	29	146	220	125	188	105	157	94.1	141	81.1	122	66.6	99.9
	30	137	205	117	175	98.1	147	87.9	132	75.8	114	62.2	93.3
	32	120	180	103	154	86.2	129	77.3	116	66.6	99.9	54.7	82.0
	34	106	160	90.8	137	76.3	115	68.5	103	59.0	88.5	48.4	72.7
	36	94.9	143	81.0	122	68.1	102	61.1	91.6	52.6	78.9	43.2	64.8
	38	85.1	128	72.7	109	61.1	91.7	54.8	82.2	47.2	70.8	38.8	58.2
	40	76.8	115	65.6	98.7	55.2	82.7	49.5	74.2	42.6	63.9	35.0	52.5

Properties

			⅝		½		⅜		5⁄16		¼		3⁄16	
M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	128	193	109	164	87.7	132	75.8	114	63.3	95.1	49.5	74.3
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	107	160	90.6	136	72.4	109	62.6	94.1	52.1	78.3	40.6	61.0

	⅝	½	⅜	5⁄16	¼	3⁄16
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}	5780	5180	4440	3980	3430	2830
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}	3790	3380	2900	2600	2240	1840
r_{mx}/r_{my}	1.23	1.24	1.24	1.24	1.24	1.24
r_{my}, in.	2.68	2.73	2.78	2.81	2.84	2.87

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

5

COMPOSITE
HSS9

Table IV-1B (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS9x5x											
		⅝		½		⅜		5⁄16		¼		3⁄16	
t_{des}, in,		0.581		0.465		0.349		0.291		0.233		0.174	
Steel, lb/ft		50.8		42.1		32.6		27.6		22.4		17.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	419	630	359	539	300	450	269	403	236	355	202	303
	1	418	628	358	537	299	449	268	402	236	354	202	302
	2	414	623	355	533	296	445	266	398	234	350	200	300
	3	409	614	350	525	292	438	262	393	230	345	197	295
	4	400	602	343	514	286	429	257	385	226	338	193	289
	5	390	587	334	500	279	418	250	375	220	330	187	281
	6	378	568	323	484	270	405	242	363	213	319	181	272
	7	364	548	310	466	260	390	233	350	205	308	174	262
	8	349	525	297	445	249	373	223	335	196	294	167	250
	9	333	500	282	423	237	355	213	319	187	280	158	237
	10	315	473	266	400	224	336	201	302	177	265	149	224
	11	297	446	250	376	211	316	190	284	166	249	140	210
	12	278	418	235	353	197	296	177	266	156	233	131	196
	13	259	389	220	330	183	275	165	248	145	217	121	182
	14	239	360	204	307	170	254	153	229	134	201	112	168
	15	220	331	189	284	156	234	141	211	123	184	102	153
	16	202	303	173	261	142	214	129	193	112	169	93.2	140
	17	184	276	159	238	129	194	117	175	102	153	84.3	126
	18	166	250	144	217	117	176	106	159	92.2	138	75.7	114
	19	149	224	130	196	107	160	94.9	142	82.8	124	67.9	102
	20	135	202	117	177	96.5	145	85.6	128	74.7	112	61.3	92.0
	21	122	184	107	160	87.5	131	77.6	116	67.7	102	55.6	83.4
	22	111	167	97.1	146	79.7	120	70.8	106	61.7	92.6	50.7	76.0
	23	102	153	88.8	134	72.9	110	64.7	97.1	56.5	84.7	46.4	69.5
	24	93.5	141	81.6	123	67.0	101	59.5	89.2	51.9	77.8	42.6	63.9
	25	86.2	130	75.2	113	61.7	92.8	54.8	82.2	47.8	71.7	39.2	58.8
	26	79.7	120	69.5	104	57.1	85.8	50.7	76.0	44.2	66.3	36.3	54.4
	27	73.9	111	64.5	96.9	52.9	79.5	47.0	70.5	41.0	61.5	33.6	50.5
	28	68.7	103	59.9	90.1	49.2	74.0	43.7	65.5	38.1	57.2	31.3	46.9
	29	64.1	96.3	55.9	84.0	45.9	69.0	40.7	61.1	35.5	53.3	29.2	43.7
	30	59.9	90.0	52.2	78.5	42.9	64.4	38.0	57.1	33.2	49.8	27.2	40.9
	32	52.6	79.1	45.9	69.0	37.7	56.6	33.4	50.2	29.2	43.8	23.9	35.9
	34							29.6	44.4	25.8	38.8	21.2	31.8

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	102	153	87.0	131	70.4	106	61.2	91.9	51.4	77.2	40.3	60.6
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	65.5	98.5	56.2	84.5	45.4	68.2	39.1	58.8	32.6	48.9	25.5	38.3

	⅝	½	⅜	5⁄16	¼	3⁄16
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}	4330	3900	3350	3040	2670	2200
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}	1610	1450	1240	1120	981	805
r_{mx}/r_{my}	1.64	1.64	1.64	1.65	1.65	1.65
r_{my}, in.	1.92	1.97	2.03	2.05	2.08	2.10

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.

Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS8

Table IV-1B (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape	HSS8x6x											
	5⁄8		1⁄2		3⁄8		5⁄16		1⁄4		3⁄16	
t_{des}, in.	0.581		0.465		0.349		0.291		0.233		0.174	
Steel, lb/ft	50.8		42.1		32.6		27.6		22.4		17.1	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y												
0	420	630	366	549	306	459	275	413	243	364	209	313
1	419	629	365	547	306	458	274	412	242	363	208	312
2	416	625	363	544	304	456	273	409	241	361	207	310
3	412	619	359	538	301	451	270	405	238	357	204	307
4	406	610	353	530	296	444	266	399	235	352	201	302
5	398	599	347	520	291	436	261	392	230	345	197	296
6	389	585	339	508	284	426	255	383	225	337	193	289
7	379	570	329	494	276	415	248	373	219	328	187	281
8	368	553	319	478	268	402	241	361	212	318	181	272
9	355	534	307	461	258	388	232	349	205	307	174	262
10	342	514	295	443	248	373	223	335	196	295	167	251
11	327	492	282	423	238	356	214	321	188	282	160	239
12	312	469	268	403	226	340	204	306	179	269	152	227
13	297	446	254	382	215	322	193	290	170	255	143	215
14	281	422	240	360	203	305	183	274	160	241	135	203
15	265	398	226	338	191	287	172	258	151	226	127	190
16	248	373	211	316	179	269	161	242	141	212	118	178
17	232	349	197	297	167	251	151	226	132	198	110	165
18	216	325	184	277	155	233	140	210	122	184	102	153
19	200	301	171	258	144	215	130	195	113	170	93.8	141
20	185	278	159	239	132	199	120	179	104	156	86.1	129
21	170	256	147	220	121	182	110	165	95.5	143	78.4	118
22	156	234	135	202	111	166	100	150	87.1	131	71.5	107
23	142	214	123	185	101	152	91.7	138	79.7	119	65.4	98.1
24	131	196	113	170	93.1	140	84.2	126	73.2	110	60.1	90.1
25	120	181	104	157	85.8	129	77.6	116	67.4	101	55.3	83.0
26	111	167	96.4	145	79.3	119	71.8	108	62.3	93.5	51.2	76.8
27	103	155	89.4	134	73.5	110	66.5	99.8	57.8	86.7	47.5	71.2
28	96.0	144	83.1	125	68.4	103	61.9	92.8	53.8	80.6	44.1	66.2
29	89.5	135	77.5	116	63.7	95.6	57.7	86.5	50.1	75.2	41.1	61.7
30	83.7	126	72.4	109	59.6	89.3	53.9	80.8	46.8	70.2	38.4	57.7
32	73.5	111	63.6	95.7	52.3	78.5	47.4	71.1	41.2	61.7	33.8	50.7
34	65.1	97.9	56.4	84.7	46.4	69.6	42.0	62.9	36.5	54.7	29.9	44.9
36	58.1	87.3	50.3	75.6	41.4	62.0	37.4	56.1	32.5	48.8	26.7	40.0
38			45.1	67.8	37.1	55.7	33.6	50.4	29.2	43.8	24.0	35.9
40							30.3	45.5	26.3	39.5	21.6	32.4

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	95.2	143	81.6	123	65.8	98.9	57.1	85.8	47.7	71.7	37.6	56.6
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	77.0	116	65.8	98.9	53.2	79.9	45.9	69.0	38.3	57.6	30.0	45.2

$P_{ex}(L_c)^2/10^4$, kip-in.^{2}	3700	3320	2860	2590	2250	1850
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}	2290	2050	1760	1590	1380	1140
r_{mx}/r_{my}	1.27	1.27	1.27	1.28	1.28	1.27
r_{my}, in.	2.27	2.32	2.38	2.40	2.43	2.46

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.

Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS8

Table IV-1B (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS8x4x											
		⅝		½		⅜		5⁄16		¼		3⁄16	
t_{des}, in,		0.581		0.465		0.349		0.291		0.233		0.174	
Steel, lb/ft		42.3		35.2		27.5		23.3		19.0		14.5	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	350	526	292	438	241	361	214	322	187	281	159	238
	1	349	524	290	436	239	359	214	320	187	280	158	237
	2	344	517	287	431	236	354	211	316	184	276	156	234
	3	336	505	280	422	231	346	206	309	180	270	152	229
	4	325	489	272	409	224	335	200	299	175	262	148	222
	5	312	469	262	393	215	322	192	288	168	252	142	213
	6	297	446	250	375	204	306	183	274	160	240	135	202
	7	279	420	236	355	192	289	172	258	151	226	127	191
	8	261	392	221	332	180	269	161	242	141	212	119	178
	9	241	362	205	309	166	249	149	224	131	196	110	165
	10	221	332	189	284	152	229	137	206	120	180	101	151
	11	200	301	173	260	139	209	125	187	109	164	91.7	138
	12	180	271	156	235	126	190	113	169	98.8	148	82.6	124
	13	161	241	140	211	114	172	101	151	88.4	133	73.8	111
	14	142	213	125	188	102	154	89.1	134	78.4	118	65.2	97.9
	15	124	186	110	165	91.0	137	78.9	119	68.7	103	57.1	85.6
	16	109	163	96.6	145	80.1	120	69.7	105	60.4	90.6	50.2	75.3
	17	96.4	145	85.6	129	71.0	107	61.7	92.7	53.5	80.3	44.4	66.7
	18	85.9	129	76.4	115	63.3	95.1	55.0	82.7	47.7	71.6	39.6	59.5
	19	77.1	116	68.5	103	56.8	85.4	49.4	74.2	42.8	64.2	35.6	53.4
	20	69.6	105	61.9	93.0	51.3	77.1	44.6	67.0	38.7	58.0	32.1	48.2
	21	63.1	94.9	56.1	84.3	46.5	69.9	40.4	60.8	35.1	52.6	29.1	43.7
	22	57.5	86.5	51.1	76.8	42.4	63.7	36.8	55.4	31.9	47.9	26.5	39.8
	23	52.6	79.1	46.8	70.3	38.8	58.3	33.7	50.7	29.2	43.8	24.3	36.4
	24	48.3	72.7	43.0	64.6	35.6	53.5	31.0	46.5	26.8	40.3	22.3	33.5
	25	44.6	67.0	39.6	59.5	32.8	49.3	28.5	42.9	24.7	37.1	20.6	30.8
	26			36.6	55.0	30.3	45.6	26.4	39.6	22.9	34.3	19.0	28.5
	27							24.5	36.8	21.2	31.8	17.6	26.4
	28											16.4	24.6

Properties

			⅝		½		⅜		5⁄16		¼		3⁄16	
M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	71.6	108	62.2	93.5	50.8	76.3	44.2	66.4	37.2	55.9	29.4	44.1
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	42.6	64.0	37.0	55.6	30.2	45.3	26.2	39.4	21.9	33.0	17.2	25.8
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}			2600		2360		2050		1860		1650		1380	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}			805		733		636		577		508		422	
r_{mx}/r_{my}			1.80		1.79		1.80		1.80		1.80		1.81	
r_{my}, in.			1.51		1.56		1.61		1.63		1.66		1.69	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS8–HSS7

Table IV-1B (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS8x4x		HSS7x5x									
		⅛		½		⅜		5⁄16		¼		3⁄16	
t_{des}, in.		0.116		0.465		0.349		0.291		0.233		0.174	
Steel, lb/ft		9.9		35.2		27.5		23.3		19.0		14.5	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	125	188	296	443	247	370	221	331	194	291	165	248
	1	125	187	295	442	246	369	220	330	193	290	165	247
	2	123	184	292	438	244	366	218	327	192	287	163	245
	3	120	180	287	431	240	360	215	322	189	283	161	241
	4	116	174	281	422	235	353	211	316	185	277	157	236
	5	111	167	273	410	229	343	205	307	180	270	153	229
	6	106	158	264	396	221	332	198	297	174	261	148	222
	7	99.3	149	253	380	213	319	191	286	168	251	142	213
	8	92.5	139	242	364	203	305	182	273	160	240	136	203
	9	85.4	128	231	347	193	290	173	260	152	228	129	193
	10	78.1	117	219	328	182	274	164	246	144	216	121	182
	11	70.7	106	206	309	171	257	154	231	135	203	114	171
	12	63.5	95.2	192	289	160	239	143	215	126	189	106	159
	13	56.4	84.6	179	269	148	222	133	200	117	176	98.0	147
	14	49.7	74.5	166	249	136	204	123	184	108	162	90.2	135
	15	43.3	64.9	152	229	125	187	113	169	99.2	149	82.5	124
	16	38.0	57.1	139	209	114	170	103	154	90.5	136	75.0	112
	17	33.7	50.6	127	190	104	156	92.9	139	82.0	123	67.7	102
	18	30.1	45.1	114	172	94.2	142	83.5	125	73.8	111	60.6	90.9
	19	27.0	40.5	103	154	85.1	128	75.0	112	66.2	99.3	54.4	81.6
	20	24.4	36.5	92.7	139	76.8	115	67.7	101	59.7	89.6	49.1	73.7
	21	22.1	33.1	84.1	126	69.6	105	61.4	92.0	54.2	81.3	44.5	66.8
	22	20.1	30.2	76.6	115	63.4	95.4	55.9	83.9	49.4	74.1	40.6	60.9
	23	18.4	27.6	70.1	105	58.0	87.2	51.2	76.7	45.2	67.8	37.1	55.7
	24	16.9	25.4	64.4	96.8	53.3	80.1	47.0	70.5	41.5	62.2	34.1	51.2
	25	15.6	23.4	59.3	89.2	49.1	73.8	43.3	64.9	38.2	57.4	31.4	47.1
	26	14.4	21.6	54.9	82.5	45.4	68.3	40.0	60.0	35.4	53.0	29.1	43.6
	27	13.4	20.0	50.9	76.5	42.1	63.3	37.1	55.7	32.8	49.2	26.9	40.4
	28	12.4	18.6	47.3	71.1	39.2	58.9	34.5	51.8	30.5	45.7	25.1	37.6
	29			44.1	66.3	36.5	54.9	32.2	48.3	28.4	42.6	23.4	35.0
	30			41.2	61.9	34.1	51.3	30.1	45.1	26.6	39.8	21.8	32.7
	32					30.0	45.1	26.4	39.6	23.3	35.0	19.2	28.8
	34											17.0	25.5

Properties

			HSS8x4x⅛		HSS7x5x½		HSS7x5x⅜		HSS7x5x5⁄16		HSS7x5x¼		HSS7x5x3⁄16	
M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	21.1	31.7	58.0	87.2	47.3	71.0	41.1	61.7	34.6	52.0	27.2	40.9
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	11.8	17.7	45.3	68.0	36.7	55.1	32.0	48.0	26.8	40.2	21.0	31.6
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}			1050		1990		1720		1570		1390		1150	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}			320		1130		982		889		785		645	
r_{mx}/r_{my}			1.81		1.33		1.32		1.33		1.33		1.34	
r_{my}, in.			1.71		1.91		1.97		1.99		2.02		2.05	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS7

Table IV-1B (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS7x5x		HSS7x4x									
		⅛		½		⅜		5⁄16		¼		3⁄16	
t_{des}, in.		0.116		0.465		0.349		0.291		0.233		0.174	
Steel, lb/ft		9.86		31.8		24.9		21.2		17.3		13.3	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	136	204	264	396	216	324	193	289	168	252	142	213
	1	135	203	263	395	215	322	192	288	167	251	142	212
	2	134	201	259	389	212	318	189	284	165	248	140	210
	3	132	198	253	381	207	311	185	277	162	242	137	205
	4	129	193	245	369	201	301	179	269	156	235	132	198
	5	125	187	236	354	192	288	172	258	150	225	127	190
	6	120	181	224	337	183	274	163	245	143	214	121	181
	7	115	173	212	318	172	258	154	231	135	202	114	170
	8	110	165	198	297	160	241	144	216	126	189	106	159
	9	104	156	183	275	148	222	133	200	117	175	98.1	147
	10	97.4	146	168	253	136	203	122	183	107	161	89.9	135
	11	90.8	136	153	230	124	186	111	166	97.4	146	81.6	122
	12	84.1	126	138	207	112	169	99.8	150	87.8	132	73.4	110
	13	77.4	116	123	185	101	152	89.0	134	78.4	118	65.5	98.2
	14	70.8	106	109	164	90.1	135	79.0	119	69.4	104	57.8	86.7
	15	64.3	96.4	95.7	144	79.7	120	70.2	106	60.8	91.2	50.5	75.8
	16	58.0	87.0	84.1	126	70.0	105	61.8	92.9	53.4	80.2	44.4	66.6
	17	51.9	77.8	74.5	112	62.0	93.2	54.8	82.3	47.3	71.0	39.3	59.0
	18	46.3	69.4	66.4	99.9	55.3	83.2	48.9	73.4	42.2	63.3	35.1	52.6
	19	41.5	62.3	59.6	89.6	49.7	74.6	43.8	65.9	37.9	56.8	31.5	47.2
	20	37.5	56.2	53.8	80.9	44.8	67.4	39.6	59.5	34.2	51.3	28.4	42.6
	21	34.0	51.0	48.8	73.4	40.7	61.1	35.9	53.9	31.0	46.5	25.8	38.7
	22	31.0	46.5	44.5	66.8	37.0	55.7	32.7	49.2	28.3	42.4	23.5	35.2
	23	28.3	42.5	40.7	61.2	33.9	50.9	29.9	45.0	25.9	38.8	21.5	32.2
	24	26.0	39.1	37.4	56.2	31.1	46.8	27.5	41.3	23.7	35.6	19.7	29.6
	25	24.0	36.0	34.4	51.8	28.7	43.1	25.3	38.1	21.9	32.8	18.2	27.3
	26	22.2	33.3			26.5	39.9	23.4	35.2	20.2	30.4	16.8	25.2
	27	20.6	30.9							18.8	28.1	15.6	23.4
	28	19.1	28.7										
	29	17.8	26.7										
	30	16.7	25.0										
	32	14.6	22.0										
	34	13.0	19.5										
Properties													
M_{nx}/Ω_b $\phi_b M_{nx}$	kip-ft	19.4	29.1	49.5	74.5	40.6	61.0	35.7	53.6	30.0	45.1	23.7	35.7
M_{ny}/Ω_b $\phi_b M_{ny}$	kip-ft	14.7	22.0	32.6	49.0	26.8	40.2	23.4	35.2	19.6	29.5	15.4	23.2
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		876		1640		1430		1300		1160		970	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		492		642		560		508		449		373	
r_{mx}/r_{my}		1.33		1.60		1.60		1.60		1.61		1.61	
r_{my}, in.		2.07		1.53		1.58		1.61		1.64		1.66	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

COMPOSITE
HSS7–HSS6

Table IV-1B (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS7x4x		HSS6x5x							
		⅛		½		⅜		5⁄16		¼	
t_{des}, in,		0.116		0.465		0.349		0.291		0.233	
Steel, lb/ft		9.01		31.8		24.9		21.2		17.3	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	115	173	264	396	220	330	197	295	172	259
	1	115	172	263	395	220	329	196	295	172	258
	2	113	170	261	392	218	326	195	292	170	256
	3	110	166	257	386	214	321	192	287	168	252
	4	107	160	251	378	210	314	188	281	164	246
	5	102	153	245	368	204	306	182	274	160	240
	6	96.8	145	237	356	197	295	176	264	155	232
	7	90.8	136	228	342	189	284	169	254	149	223
	8	84.4	127	218	327	180	271	162	243	142	213
	9	77.6	116	207	311	171	257	154	230	135	202
	10	70.7	106	195	293	161	242	145	217	127	191
	11	63.8	95.7	183	275	151	226	136	204	119	179
	12	57.0	85.5	171	257	140	211	126	190	111	167
	13	50.5	75.7	159	238	130	195	117	176	103	155
	14	44.1	66.2	146	220	119	179	108	162	95.0	143
	15	38.5	57.7	134	201	109	164	98.6	148	87.0	130
	16	33.8	50.7	122	183	99.2	149	89.6	134	79.2	119
	17	29.9	44.9	110	166	90.2	136	81.0	121	71.6	107
	18	26.7	40.1	99.3	149	81.6	123	72.6	109	64.2	96.4
	19	24.0	35.9	89.1	134	73.3	110	65.1	97.7	57.7	86.5
	20	21.6	32.4	80.4	121	66.2	99.5	58.8	88.2	52.0	78.1
	21	19.6	29.4	72.9	110	60.0	90.2	53.3	80.0	47.2	70.8
	22	17.9	26.8	66.4	99.9	54.7	82.2	48.6	72.9	43.0	64.5
	23	16.4	24.5	60.8	91.4	50.0	75.2	44.4	66.7	39.3	59.0
	24	15.0	22.5	55.8	83.9	46.0	69.1	40.8	61.2	36.1	54.2
	25	13.8	20.8	51.5	77.3	42.4	63.7	37.6	56.4	33.3	50.0
	26	12.8	19.2	47.6	71.5	39.2	58.9	34.8	52.2	30.8	46.2
	27	11.9	17.8	44.1	66.3	36.3	54.6	32.3	48.4	28.6	42.8
	28	11.0	16.6	41.0	61.6	33.8	50.8	30.0	45.0	26.5	39.8
	29			38.2	57.5	31.5	47.3	28.0	41.9	24.7	37.1
	30			35.7	53.7	29.4	44.2	26.1	39.2	23.1	34.7
	32					25.9	38.9	23.0	34.4	20.3	30.5
Properties											
M_{nx}/Ω_b | $\phi_b M_{nx}$	kip-ft	17.0	25.6	45.3	68.0	37.0	55.6	32.3	48.5	27.2	40.9
M_{ny}/Ω_b | $\phi_b M_{ny}$	kip-ft	10.8	16.3	39.7	59.7	32.4	48.7	28.2	42.4	23.8	35.7
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		743		1330		1150		1050		928	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		284		978		850		772		684	
r_{mx}/r_{my}		1.62		1.17		1.16		1.17		1.16	
r_{my}, in.		1.69		1.87		1.92		1.95		1.98	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.

Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS6

Table IV-1B (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS6x5x				HSS6x4x							
		3⁄16		1⁄8		1⁄2		3⁄8		5⁄16		1⁄4	
t_{des}, in.		0.174		0.116		0.465		0.349		0.291		0.233	
Steel, lb/ft		13.3		9.01		28.4		22.4		19.1		15.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	147	220	120	180	236	355	191	287	171	256	149	223
	1	146	219	120	179	235	353	191	286	170	255	148	222
	2	145	217	118	177	232	348	188	282	168	251	146	219
	3	143	214	116	175	226	340	183	275	164	245	143	214
	4	139	209	114	170	219	329	177	266	158	238	138	207
	5	136	203	110	165	210	315	170	255	152	228	133	199
	6	131	196	106	159	199	300	161	242	144	216	126	189
	7	126	189	102	153	188	282	151	227	136	204	119	178
	8	120	180	96.8	145	175	263	141	211	127	190	111	166
	9	114	171	91.4	137	161	243	130	195	117	175	102	154
	10	107	161	85.8	129	148	222	119	179	107	160	93.8	141
	11	100	151	80.0	120	134	201	109	164	96.9	145	85.1	128
	12	93.5	140	74.0	111	120	181	98.4	148	87.0	131	76.5	115
	13	86.5	130	68.1	102	107	161	88.2	133	77.4	116	68.1	102
	14	79.5	119	62.2	93.3	94.3	142	78.4	118	68.9	104	60.1	90.2
	15	72.6	109	56.4	84.7	82.3	124	68.9	104	60.9	91.6	52.5	78.7
	16	65.9	98.9	50.9	76.3	72.3	109	60.5	91.0	53.5	80.5	46.1	69.2
	17	59.5	89.2	45.5	68.2	64.0	96.2	53.6	80.6	47.4	71.3	40.9	61.3
	18	53.2	79.8	40.6	60.8	57.1	85.9	47.8	71.9	42.3	63.6	36.4	54.7
	19	47.8	71.6	36.4	54.6	51.3	77.1	42.9	64.5	38.0	57.1	32.7	49.1
	20	43.1	64.7	32.9	49.3	46.3	69.5	38.7	58.2	34.3	51.5	29.5	44.3
	21	39.1	58.6	29.8	44.7	42.0	63.1	35.1	52.8	31.1	46.7	26.8	40.2
	22	35.6	53.4	27.2	40.7	38.2	57.5	32.0	48.1	28.3	42.6	24.4	36.6
	23	32.6	48.9	24.8	37.3	35.0	52.6	29.3	44.0	25.9	38.9	22.3	33.5
	24	29.9	44.9	22.8	34.2	32.1	48.3	26.9	40.4	23.8	35.8	20.5	30.8
	25	27.6	41.4	21.0	31.5	29.6	44.5	24.8	37.3	21.9	33.0	18.9	28.3
	26	25.5	38.3	19.4	29.2					20.3	30.5	17.5	26.2
	27	23.7	35.5	18.0	27.0								
	28	22.0	33.0	16.8	25.1								
	29	20.5	30.8	15.6	23.4								
	30	19.2	28.7	14.6	21.9								
	32	16.8	25.3	12.8	19.3								
Properties													
M_{nx}/Ω_b / $\phi_b M_{nx}$	kip-ft	21.5	32.4	15.3	23.0	38.3	57.5	31.7	47.7	27.8	41.8	23.5	35.3
M_{ny}/Ω_b / $\phi_b M_{ny}$	kip-ft	18.7	28.1	13.2	19.9	28.5	42.8	23.5	35.3	20.5	30.9	17.3	26.0
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		771		588		1080		950		865		770	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		566		432		551		481		440		388	
r_{mx}/r_{my}		1.17		1.17		1.40		1.41		1.40		1.41	
r_{my}, in.		2.01		2.03		1.50		1.55		1.58		1.61	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

5

COMPOSITE
HSS6

Table IV-1B (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS6x4x				HSS6x3x							
		3⁄16		1⁄8		1⁄2		3⁄8		5⁄16		1⁄4	
t_{des}, in,		0.174		0.116		0.465		0.349		0.291		0.233	
Steel, lb/ft		12.0		8.16		25.0		19.8		17.0		13.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	126	189	102	153	208	313	164	247	145	217	126	189
	1	125	188	101	152	206	310	163	245	144	215	125	187
	2	123	185	99.9	150	201	302	159	239	140	210	122	183
	3	121	181	97.5	146	193	290	153	230	134	202	117	175
	4	117	175	94.2	141	182	273	145	218	127	190	111	166
	5	112	168	90.0	135	169	254	135	203	118	177	103	154
	6	106	160	85.3	128	154	231	124	187	108	162	94.2	141
	7	100	150	80.0	120	138	207	113	169	97.3	146	84.9	127
	8	93.5	140	74.2	111	122	183	100	151	87.1	131	75.2	113
	9	86.4	130	68.2	102	105	158	88.0	132	76.7	115	65.6	98.5
	10	79.1	119	62.1	93.2	89.9	135	76.0	114	66.6	100	56.7	85.2
	11	71.7	108	56.0	84.0	75.2	113	64.7	97.2	57.0	85.7	48.8	73.4
	12	64.5	96.7	50.0	74.9	63.2	95.0	54.4	81.7	48.0	72.2	41.4	62.3
	13	57.4	86.1	44.1	66.2	53.8	80.9	46.3	69.6	40.9	61.5	35.3	53.1
	14	50.6	76.0	38.6	57.8	46.4	69.8	39.9	60.0	35.3	53.0	30.4	45.7
	15	44.2	66.3	33.6	50.4	40.4	60.8	34.8	52.3	30.7	46.2	26.5	39.9
	16	38.9	58.3	29.5	44.3	35.5	53.4	30.6	46.0	27.0	40.6	23.3	35.0
	17	34.4	51.6	26.1	39.2	31.5	47.3	27.1	40.7	23.9	36.0	20.6	31.0
	18	30.7	46.0	23.3	35.0	28.1	42.2	24.2	36.3	21.4	32.1	18.4	27.7
	19	27.6	41.3	20.9	31.4			21.7	32.6	19.2	28.8	16.5	24.8
	20	24.9	37.3	18.9	28.3							14.9	22.4
	21	22.6	33.8	17.1	25.7								
	22	20.6	30.8	15.6	23.4								
	23	18.8	28.2	14.3	21.4								
	24	17.3	25.9	13.1	19.7								
	25	15.9	23.9	12.1	18.1								
	26	14.7	22.1	11.2	16.8								
	27	13.6	20.5	10.4	15.6								

Properties														
M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	18.6	28.0	13.3	20.1	31.5	47.3	26.2	39.4	23.1	34.7	19.6	29.5
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	13.7	20.5	9.69	14.6	18.6	28.0	15.6	23.5	13.8	20.7	11.6	17.5
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}			651		495		841		746		685		609	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}			327		248		261		232		213		189	
r_{mx}/r_{my}			1.41		1.41		1.80		1.79		1.79		1.80	
r_{my}, in.			1.63		1.66		1.12		1.17		1.19		1.22	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

Table IV-1B (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS6x3x				HSS5x4x							
		3⁄16		1⁄8		1⁄2		3⁄8		5⁄16		1⁄4	
t_{des}, in,		0.174		0.116		0.465		0.349		0.291		0.233	
Steel, lb/ft		10.7		7.31		25.0		19.8		17.0		13.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	105	158	83.9	126	208	313	167	250	149	223	130	195
	1	104	156	83.2	125	207	311	166	249	148	222	129	194
	2	102	153	81.2	122	204	307	164	246	146	219	127	191
	3	97.8	147	77.9	117	199	299	160	239	143	214	124	187
	4	92.6	139	73.6	110	192	289	154	231	138	207	120	181
	5	86.2	129	68.3	103	184	276	147	221	132	198	115	173
	6	79.1	119	62.4	93.6	174	262	140	209	125	187	109	164
	7	71.4	107	56.1	84.2	163	246	131	197	117	176	103	154
	8	63.4	95.1	49.6	74.4	152	228	123	184	109	163	95.7	144
	9	55.5	83.2	43.1	64.7	139	210	113	170	100	150	88.3	132
	10	47.7	71.6	36.9	55.4	127	191	104	156	91.4	137	80.6	121
	11	40.4	60.6	31.0	46.5	114	172	94.5	142	82.5	124	72.9	109
	12	34.0	50.9	26.1	39.1	102	154	85.1	128	73.9	111	65.3	98.0
	13	28.9	43.4	22.2	33.3	90.3	136	76.0	114	66.2	99.5	58.0	87.0
	14	24.9	37.4	19.1	28.7	78.9	119	67.2	101	58.7	88.2	51.0	76.5
	15	21.7	32.6	16.7	25.0	68.7	103	58.7	88.3	51.5	77.4	44.4	66.6
	16	19.1	28.6	14.7	22.0	60.4	90.8	51.6	77.6	45.3	68.0	39.0	58.5
	17	16.9	25.4	13.0	19.5	53.5	80.4	45.7	68.7	40.1	60.3	34.6	51.9
	18	15.1	22.6	11.6	17.4	47.7	71.7	40.8	61.3	35.8	53.7	30.8	46.3
	19	13.5	20.3	10.4	15.6	42.8	64.4	36.6	55.0	32.1	48.2	27.7	41.5
	20	12.2	18.3	9.38	14.1	38.7	58.1	33.0	49.7	29.0	43.5	25.0	37.5
	21			8.51	12.8	35.1	52.7	30.0	45.0	26.3	39.5	22.7	34.0
	22					31.9	48.0	27.3	41.0	23.9	36.0	20.6	31.0
	23					29.2	43.9	25.0	37.5	21.9	32.9	18.9	28.3
	24					26.8	40.4	22.9	34.5	20.1	30.2	17.3	26.0
	25							21.1	31.8	18.5	27.9	16.0	24.0
	26											14.8	22.2
Properties													
M_{nx}/Ω_b | $\phi_b M_{nx}$	kip-ft	15.6	23.5	11.3	16.9	28.4	42.7	23.7	35.6	20.9	31.3	17.7	26.5
M_{ny}/Ω_b | $\phi_b M_{ny}$	kip-ft	9.23	13.9	6.56	9.86	24.2	36.4	20.1	30.2	17.7	26.6	15.0	22.5
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		521		403		664		587		536		478	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		161		123		459		404		368		328	
r_{mx}/r_{my}		1.80		1.81		1.20		1.21		1.21		1.21	
r_{my}, in.		1.25		1.27		1.46		1.52		1.54		1.57	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS5

Table IV-1B (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS5x4x				HSS5x3x							
		3⁄16		1⁄8		1⁄2		3⁄8		5⁄16		1⁄4	
t_{des}, in,		0.174		0.116		0.465		0.349		0.291		0.233	
Steel, lb/ft		10.7		7.31		21.6		17.3		14.8		12.2	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	109	164	88.2	132	180	271	143	215	125	188	109	163
	1	109	163	87.7	132	179	269	142	213	124	186	108	162
	2	107	161	86.4	130	174	261	139	208	121	181	105	158
	3	105	157	84.3	126	166	250	133	200	116	174	101	151
	4	101	152	81.4	122	156	235	126	189	109	164	95.2	143
	5	97.1	146	77.8	117	144	217	117	176	101	152	88.3	133
	6	92.2	138	73.6	110	131	197	107	161	93.1	140	80.7	121
	7	86.7	130	69.0	103	117	175	96.2	145	84.2	127	72.5	109
	8	80.8	121	64.0	96.0	102	154	85.2	128	75.0	113	64.1	96.1
	9	74.5	112	58.8	88.1	87.9	132	74.2	112	65.8	99.0	55.7	83.5
	10	68.1	102	53.4	80.1	74.3	112	63.7	95.7	56.9	85.5	48.0	72.1
	11	61.7	92.5	48.1	72.1	61.7	92.7	53.6	80.5	48.4	72.7	41.0	61.7
	12	55.3	83.0	42.9	64.3	51.8	77.9	45.0	67.7	40.7	61.1	34.6	52.0
	13	49.2	73.7	37.8	56.7	44.2	66.4	38.4	57.7	34.7	52.1	29.5	44.3
	14	43.3	64.9	33.0	49.5	38.1	57.2	33.1	49.7	29.9	44.9	25.4	38.2
	15	37.7	56.6	28.7	43.1	33.2	49.9	28.8	43.3	26.0	39.1	22.1	33.3
	16	33.2	49.7	25.2	37.9	29.2	43.8	25.3	38.1	22.9	34.4	19.5	29.2
	17	29.4	44.0	22.4	33.5	25.8	38.8	22.4	33.7	20.3	30.5	17.2	25.9
	18	26.2	39.3	19.9	29.9	23.0	34.6	20.0	30.1	18.1	27.2	15.4	23.1
	19	23.5	35.3	17.9	26.9			18.0	27.0	16.2	24.4	13.8	20.7
	20	21.2	31.8	16.2	24.2								
	21	19.2	28.9	14.7	22.0								
	22	17.5	26.3	13.4	20.0								
	23	16.0	24.1	12.2	18.3								
	24	14.7	22.1	11.2	16.8								
	25	13.6	20.4	10.3	15.5								
	26	12.6	18.8	9.56	14.3								
	27			8.86	13.3								
Properties													
M_{nx}/Ω_b | $\phi_b M_{nx}$	kip-ft	14.1	21.1	10.1	15.1	22.9	34.4	19.3	29.0	17.1	25.7	14.5	21.9
M_{ny}/Ω_b | $\phi_b M_{ny}$	kip-ft	11.9	17.9	8.47	12.7	15.6	23.5	13.2	19.8	11.7	17.6	9.95	15.0
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		408		311		507		455		420		374	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		279		212		215		194		178		159	
r_{mx}/r_{my}		1.21		1.21		1.54		1.53		1.54		1.53	
r_{my}, in.		1.60		1.62		1.09		1.14		1.17		1.19	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS5–HSS4

Table IV-1B (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS5x3x				HSS5x2½x						HSS4x3x	
		3⁄16		⅛		¼		3⁄16		⅛		⅜	
t_{des}, in,		0.174		0.116		0.233		0.174		0.116		0.349	
Steel, lb/ft		9.42		6.46		11.4		8.8		6.03		14.7	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	90.7	136	72.3	108	98.0	147	81.5	122	64.2	96.3	122	184
	1	89.9	135	71.7	108	96.9	145	80.5	121	63.5	95.2	121	182
	2	87.8	132	69.9	105	93.5	140	77.8	117	61.3	92.0	118	178
	3	84.3	126	67.1	101	88.2	132	73.5	110	57.9	86.8	113	170
	4	79.7	119	63.3	94.9	81.2	122	67.8	102	53.4	80.1	107	161
	5	74.1	111	58.7	88.1	73.0	110	61.2	91.8	48.1	72.2	98.9	149
	6	67.8	102	53.6	80.4	64.3	96.6	53.9	80.9	42.4	63.6	90.0	135
	7	61.0	91.5	48.1	72.1	56.1	84.3	46.5	69.7	36.5	54.8	80.6	121
	8	54.0	81.0	42.4	63.7	47.9	71.9	39.1	58.7	30.7	46.1	70.9	107
	9	47.1	70.6	36.9	55.3	40.0	60.1	32.2	48.3	25.3	37.9	61.3	92.1
	10	40.4	60.6	31.5	47.2	32.7	49.2	26.2	39.3	20.5	30.7	52.1	78.3
	11	34.0	51.0	26.4	39.6	27.0	40.6	21.6	32.5	16.9	25.4	43.5	65.3
	12	28.6	42.9	22.2	33.2	22.7	34.1	18.2	27.3	14.2	21.3	36.5	54.9
	13	24.3	36.5	18.9	28.3	19.4	29.1	15.5	23.3	12.1	18.2	31.1	46.8
	14	21.0	31.5	16.3	24.4	16.7	25.1	13.4	20.1	10.4	15.7	26.8	40.3
	15	18.3	27.4	14.2	21.3	14.5	21.9	11.6	17.5	9.09	13.6	23.4	35.1
	16	16.1	24.1	12.5	18.7	12.8	19.2	10.2	15.4	7.99	12.0	20.5	30.9
	17	14.2	21.4	11.0	16.6			9.06	13.6	7.08	10.6	18.2	27.4
	18	12.7	19.0	9.85	14.8							16.2	24.4
	19	11.4	17.1	8.84	13.3								
	20	10.3	15.4	7.98	12.0								
Properties													
M_{nx}/Ω_b | $\phi_b M_{nx}$	kip-ft	11.7	17.5	8.42	12.7	13.0	19.5	10.4	15.7	7.58	11.4	13.4	20.1
M_{ny}/Ω_b | $\phi_b M_{ny}$	kip-ft	7.94	11.9	5.68	8.54	7.71	11.6	6.18	9.29	4.44	6.67	10.8	16.3
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}		321		250		323		277		220		250	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}		135		105		100		85.7		67.2		155	
r_{mx}/r_{my}		1.54		1.54		1.80		1.80		1.81		1.27	
r_{my}, in.		1.22		1.25		0.999		1.02		1.05		1.11	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

5

COMPOSITE
HSS4

Table IV-1B (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS4x3x								HSS4x2½x			
		5⁄16		¼		3⁄16		⅛		⅜		5⁄16	
t_{des}, in,		0.291		0.233		0.174		0.116		0.349		0.291	
Steel, lb/ft		12.7		10.5		8.15		5.61		13.4		11.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	105	158	91.7	138	76.5	115	60.6	91.0	112	168	96.7	145
	1	105	157	90.9	136	75.9	114	60.1	90.2	111	166	95.6	144
	2	102	153	88.6	133	74.0	111	58.6	87.9	107	160	92.3	139
	3	97.9	147	84.8	127	71.0	106	56.2	84.3	100	151	87.0	131
	4	92.4	139	79.9	120	66.9	100	53.0	79.4	91.8	138	80.1	120
	5	85.8	129	73.9	111	62.0	93.1	49.1	73.6	82.2	123	72.1	108
	6	78.3	118	67.2	101	56.6	84.9	44.7	67.1	71.7	108	63.4	95.2
	7	70.4	106	60.1	90.2	50.7	76.1	40.1	60.1	61.0	91.7	54.4	81.8
	8	62.2	93.4	52.8	79.4	44.7	67.1	35.3	52.9	50.7	76.2	45.6	68.6
	9	54.0	81.2	46.2	69.5	38.8	58.2	30.6	45.8	41.0	61.6	37.3	56.1
	10	46.2	69.4	39.8	59.9	33.1	49.6	26.0	39.0	33.2	49.9	30.2	45.4
	11	38.8	58.3	33.8	50.8	27.7	41.5	21.7	32.6	27.4	41.2	25.0	37.6
	12	32.6	49.0	28.4	42.7	23.3	34.9	18.3	27.4	23.0	34.6	21.0	31.6
	13	27.8	41.7	24.2	36.3	19.8	29.7	15.6	23.3	19.6	29.5	17.9	26.9
	14	23.9	36.0	20.9	31.3	17.1	25.6	13.4	20.1	16.9	25.4	15.4	23.2
	15	20.9	31.3	18.2	27.3	14.9	22.3	11.7	17.5	14.7	22.2	13.4	20.2
	16	18.3	27.5	16.0	24.0	13.1	19.6	10.3	15.4				
	17	16.2	24.4	14.1	21.3	11.6	17.4	9.1	13.7				
	18	14.5	21.8	12.6	19.0	10.3	15.5	8.12	12.2				
	19			11.3	17.0	9.28	13.9	7.29	10.9				
	20							6.57	9.86				

Properties

			HSS4x3x5⁄16 ASD	HSS4x3x5⁄16 LRFD	HSS4x3x¼ ASD	HSS4x3x¼ LRFD	HSS4x3x3⁄16 ASD	HSS4x3x3⁄16 LRFD	HSS4x3x⅛ ASD	HSS4x3x⅛ LRFD	HSS4x2½x⅜ ASD	HSS4x2½x⅜ LRFD	HSS4x2½x5⁄16 ASD	HSS4x2½x5⁄16 LRFD
M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	11.9	17.9	10.2	15.3	8.19	12.3	5.96	8.95	11.6	17.5	10.4	15.7
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	9.63	14.5	8.24	12.4	6.61	9.93	4.77	7.17	8.22	12.4	7.38	11.1
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}			232		208		178		140		212		197	
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}			143		128		110		86.4		96.1		89.5	
r_{mx}/r_{my}			1.27		1.27		1.27		1.27		1.49		1.48	
r_{my}, in.			1.13		1.16		1.19		1.21		0.922		0.947	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS4

Table IV-1B (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS4x2½x						HSS4x2x					
		¼		3⁄16		⅛		⅜		5⁄16		¼	
t_{des}, in,		0.233		0.174		0.116		0.349		0.291		0.233	
Steel, lb/ft		9.66		7.51		5.18		12.2		10.6		8.81	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	81.9	123	68.1	102	53.6	80.5	101	153	88.0	132	73.1	110
	1	80.9	121	67.3	101	53.0	79.5	99.5	150	86.4	130	71.8	108
	2	78.0	117	65.0	97.5	51.2	76.8	93.8	141	81.7	123	68.2	102
	3	73.4	110	61.3	91.9	48.2	72.4	84.9	128	74.5	112	62.5	93.9
	4	67.4	101	56.4	84.6	44.4	66.7	73.9	111	65.5	98.4	55.3	83.2
	5	60.5	91.0	50.7	76.0	40.0	60.0	61.9	93.0	55.4	83.3	47.3	71.2
	6	53.6	80.5	44.5	66.7	35.1	52.7	49.7	74.8	45.2	67.9	39.1	58.8
	7	46.4	69.7	38.1	57.2	30.1	45.2	38.4	57.7	35.5	53.4	31.2	46.9
	8	39.2	59.0	31.9	47.9	25.3	37.9	29.4	44.2	27.3	41.0	24.1	36.3
	9	32.5	48.8	26.2	39.4	20.7	31.0	23.2	34.9	21.5	32.4	19.1	28.7
	10	26.4	39.7	21.5	32.3	16.7	25.1	18.8	28.3	17.4	26.2	15.5	23.2
	11	21.8	32.8	17.7	26.7	13.8	20.8	15.5	23.4	14.4	21.7	12.8	19.2
	12	18.3	27.5	14.9	22.4	11.6	17.4	13.1	19.6	12.1	18.2	10.7	16.1
	13	15.6	23.5	12.7	19.1	9.91	14.9					9.15	13.7
	14	13.5	20.2	10.9	16.5	8.54	12.8						
	15	11.7	17.6	9.54	14.3	7.44	11.2						
	16	10.3	15.5	8.38	12.6	6.54	9.81						
	17					5.79	8.69						

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	9.00	13.5	7.27	10.9	5.29	7.96	9.93	14.9	8.96	13.5	7.78	11.7
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	6.36	9.55	5.13	7.70	3.70	5.56	5.89	8.85	5.34	8.02	4.63	6.96

	HSS4x2½x ¼	HSS4x2½x 3⁄16	HSS4x2½x ⅛	HSS4x2x ⅜	HSS4x2x 5⁄16	HSS4x2x ¼
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}	178	153	123	173	163	148
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}	80.9	69.4	55.0	53.5	50.5	46.0
r_{mx}/r_{my}	1.48	1.48	1.50	1.80	1.80	1.79
r_{my}, in.	0.973	0.999	1.03	0.729	0.754	0.779

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.

Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS4

Table IV-1B (continued)
Available Strength in Axial Compression, kips
Filled Rectangular HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS4x2x			
		3/16		1/8	
t_{des}, in,		0.174		0.116	
Steel, lb/ft		6.87		4.75	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	60.0	90.0	46.6	69.9
	1	58.9	88.4	45.8	68.7
	2	55.8	83.7	43.4	65.2
	3	50.9	76.4	39.8	59.6
	4	44.8	67.2	35.1	52.7
	5	38.0	57.0	29.9	44.9
	6	31.5	47.3	24.6	36.9
	7	25.5	38.3	19.6	29.3
	8	19.9	29.9	15.1	22.6
	9	15.7	23.7	11.9	17.9
	10	12.8	19.2	9.65	14.5
	11	10.5	15.8	7.98	12.0
	12	8.86	13.3	6.70	10.1
	13	7.55	11.3	5.71	8.57

Properties

ASD	LRFD	Unit	3/16 ASD	3/16 LRFD	1/8 ASD	1/8 LRFD
M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	6.33	9.51	4.64	6.98
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	3.75	5.64	2.73	4.10

Property	3/16	1/8
$P_{ex}(L_c)^2/10^4$, kip-in.^{2}	128	103
$P_{ey}(L_c)^2/10^4$, kip-in.^{2}	39.6	31.7
r_{mx}/r_{my}	1.80	1.80
r_{my}, in.	0.804	0.830

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_{my} equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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4

COMPOSITE
HSS16–HSS14

Table IV-2A

Available Strength in Axial Compression, kips

Filled Square HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS16×16×						HSS14×14×					
		½		⅜		5⁄16		⅝		½		⅜	
t_{des}, in,		0.465		0.349		0.291		0.581		0.465		0.349	
Steel, lb/ft		103		78.5		65.9		110		89.7		68.3	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	1090	1640	935	1400	856	1280	1040	1560	905	1360	768	1150
	1	1090	1640	935	1400	856	1280	1040	1560	905	1360	768	1150
	2	1090	1640	934	1400	855	1280	1040	1550	904	1360	767	1150
	3	1090	1640	933	1400	854	1280	1030	1550	902	1350	765	1150
	4	1090	1630	931	1400	852	1280	1030	1550	899	1350	763	1140
	5	1090	1630	928	1390	850	1270	1030	1540	896	1340	760	1140
	6	1080	1620	925	1390	847	1270	1020	1530	893	1340	757	1140
	7	1080	1620	921	1380	843	1260	1020	1530	888	1330	753	1130
	8	1070	1610	917	1380	839	1260	1010	1520	883	1320	749	1120
	9	1070	1600	912	1370	834	1250	1010	1510	877	1320	744	1120
	10	1060	1590	907	1360	829	1240	998	1500	871	1310	738	1110
	11	1050	1580	901	1350	824	1240	990	1490	864	1300	732	1100
	12	1050	1570	894	1340	817	1230	982	1470	856	1280	725	1090
	13	1040	1560	887	1330	811	1220	972	1460	848	1270	718	1080
	14	1030	1550	880	1320	804	1210	962	1440	839	1260	710	1070
	15	1020	1530	872	1310	796	1190	952	1430	830	1240	702	1050
	16	1010	1520	863	1300	788	1180	940	1410	820	1230	693	1040
	17	1000	1500	855	1280	780	1170	929	1390	809	1210	684	1030
	18	992	1490	845	1270	771	1160	916	1370	798	1200	675	1010
	19	981	1470	835	1250	762	1140	903	1350	787	1180	665	997
	20	970	1450	825	1240	752	1130	890	1330	775	1160	654	982
	21	958	1440	815	1220	743	1110	876	1310	763	1140	644	966
	22	945	1420	804	1210	732	1100	862	1290	750	1130	633	949
	23	933	1400	793	1190	722	1080	847	1270	737	1110	622	932
	24	920	1380	781	1170	711	1070	832	1250	724	1090	610	915
	25	906	1360	769	1150	700	1050	816	1220	711	1070	598	897
	26	893	1340	757	1140	688	1030	801	1200	697	1050	586	879
	27	878	1320	745	1120	676	1010	785	1180	683	1020	574	861
	28	864	1300	732	1100	664	997	768	1150	668	1000	561	842
	29	849	1270	719	1080	652	979	752	1130	654	980	549	823
	30	834	1250	706	1060	640	960	735	1100	639	958	536	804
	32	804	1210	679	1020	615	922	701	1050	609	913	510	765
	34	773	1160	651	977	589	884	666	999	579	868	484	725
	36	741	1110	624	935	563	845	632	947	548	822	457	686
	38	709	1060	595	893	537	805	597	895	518	776	431	646
	40	676	1010	567	850	510	765	562	843	487	731	405	607
Properties													
M_n/Ω_b $\phi_b M_n$	kip-ft	456	685	358	538	305	459	409	615	341	513	268	403
$P_e(L_c)^2/10^4$, kip-in.^{2}		43900		36000		31900		32700		28200		23100	
r_m, in.		6.31		6.37		6.39		5.44		5.49		5.55	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

4

COMPOSITE
HSS14–HSS12

Table IV-2A (continued)

Available Strength in Axial Compression, kips

Filled Square HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS14×14×		HSS12×12×									
		5⁄16		5⁄8		1⁄2		3⁄8		5⁄16		1⁄4	
t_{des}, in.		0.291		0.581		0.465		0.349		0.291		0.233	
Steel, lb/ft		57.4		93.3		76.1		58.1		48.9		39.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	698	1050	842	1260	731	1100	617	925	557	835	496	744
	1	698	1050	841	1260	730	1100	617	925	556	834	496	744
	2	697	1050	840	1260	729	1090	616	923	555	833	495	742
	3	696	1040	838	1260	727	1090	614	921	554	831	494	740
	4	694	1040	835	1250	724	1090	612	918	552	827	492	737
	5	691	1040	831	1250	721	1080	609	913	549	823	489	734
	6	688	1030	826	1240	717	1080	605	908	546	818	486	729
	7	684	1030	820	1230	712	1070	601	901	542	813	482	724
	8	680	1020	814	1220	707	1060	596	894	537	806	478	718
	9	676	1010	806	1210	700	1050	591	886	532	799	474	711
	10	670	1010	798	1200	693	1040	585	877	527	790	469	703
	11	665	997	790	1180	686	1030	578	867	521	781	463	695
	12	658	987	780	1170	678	1020	571	857	514	771	457	686
	13	651	977	770	1150	669	1000	564	845	507	761	451	676
	14	644	966	759	1140	660	989	555	833	500	750	444	666
	15	637	955	747	1120	650	974	547	820	492	738	437	655
	16	629	943	735	1100	639	959	538	807	484	726	429	643
	17	620	930	723	1080	628	943	528	793	475	713	421	631
	18	611	917	709	1060	617	925	519	778	466	699	413	619
	19	602	903	696	1040	605	908	508	763	457	685	404	606
	20	592	889	682	1020	593	890	498	747	447	671	395	593
	21	582	874	667	1000	580	871	487	731	437	656	386	579
	22	572	858	652	978	568	851	476	714	427	641	377	565
	23	562	843	637	955	554	832	465	697	417	625	367	551
	24	551	826	621	932	541	812	453	680	406	609	358	536
	25	540	810	605	908	527	791	441	662	395	593	348	522
	26	529	793	589	884	514	770	430	644	384	577	338	507
	27	517	776	573	859	500	749	418	626	373	560	328	492
	28	506	759	557	835	485	728	405	608	362	544	318	477
	29	494	741	540	810	471	707	393	590	351	527	308	462
	30	482	723	523	785	457	685	381	571	340	510	298	446
	32	458	687	490	735	428	643	356	535	318	477	277	416
	34	434	651	457	686	400	600	332	498	296	443	257	386
	36	410	614	425	637	372	558	308	462	274	411	238	356
	38	385	578	393	589	344	516	285	427	253	379	218	328
	40	361	542	362	543	317	476	262	393	232	348	200	300
Properties													
M_n/Ω_b / $\phi_b M_n$	kip-ft	230	345	292	440	244	367	192	289	165	248	136	205
$P_e(L_c)^2/10^4$, kip-in.^{2}		20400		19200		16900		13900		12300		10500	
r_m, in.		5.58		4.62		4.68		4.73		4.76		4.79	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

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COMPOSITE
HSS12–HSS10

Table IV-2A (continued)
Available Strength in Axial Compression, kips
Filled Square HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS12×12×		HSS10×10×									
		3/16		5/8		1/2		3/8		5/16		1/4	
t_{des}, in.		0.174		0.581		0.465		0.349		0.291		0.233	
Steel, lb/ft		29.8		76.3		62.5		47.9		40.4		32.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	417	626	657	986	570	854	477	715	428	642	378	568
	1	417	625	657	985	569	854	477	715	428	642	378	567
	2	416	624	655	983	568	852	475	713	427	640	377	566
	3	415	622	653	979	566	848	474	711	425	638	376	564
	4	413	620	649	974	563	844	471	707	423	634	374	560
	5	411	617	645	967	559	838	468	702	420	630	371	557
	6	408	613	639	959	554	831	464	696	416	625	368	552
	7	405	608	633	949	549	823	460	689	412	618	364	546
	8	402	603	626	938	543	814	454	682	408	611	360	540
	9	398	597	617	926	536	803	449	673	402	603	355	532
	10	394	590	608	913	528	792	442	663	396	595	350	524
	11	389	583	599	898	520	779	435	653	390	585	344	516
	12	384	576	588	882	511	766	428	642	383	575	338	506
	13	378	567	577	865	501	752	420	630	376	564	331	496
	14	372	559	565	847	491	736	411	617	368	552	324	486
	15	366	549	552	829	480	720	402	603	360	540	316	475
	16	360	539	539	809	469	704	393	589	351	527	309	463
	17	353	529	526	789	458	686	383	575	343	514	301	451
	18	346	519	512	768	446	668	373	560	333	500	293	439
	19	338	508	497	746	433	650	363	544	324	486	284	426
	20	331	496	483	724	421	631	352	529	314	472	275	413
	21	323	485	468	701	408	612	342	512	305	457	267	400
	22	315	473	452	679	395	592	331	496	295	442	258	386
	23	307	461	437	655	382	572	320	479	285	427	249	373
	24	299	448	421	632	368	552	308	463	275	412	239	359
	25	291	436	406	609	355	532	297	446	264	397	230	345
	26	282	423	390	585	341	512	286	429	254	381	221	332
	27	274	410	374	562	328	492	275	412	244	366	212	318
	28	265	397	359	538	315	472	264	395	234	351	203	304
	29	256	385	343	515	301	452	252	379	224	336	194	291
	30	248	372	328	492	288	432	241	362	214	321	185	278
	32	231	346	298	448	262	393	220	330	194	291	168	252
	34	214	321	271	407	237	356	199	298	176	263	151	227
	36	197	296	244	367	213	320	179	268	157	236	135	202
	38	181	271	219	329	191	287	160	240	141	212	121	182
	40	165	248	198	297	173	259	145	217	127	191	109	164
Properties													
M_n/Ω_b $\phi_b M_n$	kip-ft	100	151	194	292	163	246	130	195	111	168	92.4	139
$P_e(L_c)^2/10^4$, kip-in.^{2}		8690		10300		9070		7600		6690		5740	
r_m, in.		4.82		3.80		3.86		3.92		3.94		3.97	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS10–HSS9

Table IV-2A (continued)

Available Strength in Axial Compression, kips

Filled Square HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS10×10×		HSS9×9×									
		3⁄16		5⁄8		½		3⁄8		5⁄16		¼	
t_{des}, in.		0.174		0.581		0.465		0.349		0.291		0.233	
Steel, lb/ft		24.7		67.8		55.7		42.8		36.1		29.2	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	327	491	571	857	493	739	412	618	368	553	324	487
	1	327	491	571	856	492	739	412	617	368	552	324	486
	2	326	489	569	854	491	737	410	616	367	550	323	485
	3	325	487	567	850	489	733	409	613	365	548	322	483
	4	323	484	563	844	486	728	406	609	363	544	320	479
	5	321	481	558	837	481	722	403	604	360	540	317	475
	6	318	476	552	828	477	715	398	598	356	534	313	470
	7	314	471	545	818	471	706	394	591	352	528	310	464
	8	310	465	537	806	464	696	388	582	347	520	305	458
	9	306	459	529	793	457	685	382	573	341	512	300	450
	10	301	452	519	778	449	673	376	563	335	503	295	442
	11	296	444	509	763	440	660	368	552	329	493	289	433
	12	290	435	497	746	431	646	361	541	322	483	282	424
	13	284	426	486	728	421	631	352	528	314	471	276	414
	14	278	417	473	710	410	615	344	515	306	460	269	403
	15	271	407	460	690	399	598	334	502	298	447	261	392
	16	264	396	446	670	388	581	325	487	290	434	254	380
	17	257	386	432	649	376	564	315	473	281	421	246	368
	18	250	374	418	627	364	545	305	458	272	408	237	356
	19	242	363	403	606	351	527	295	442	262	394	229	344
	20	234	351	389	585	339	508	284	427	253	379	221	331
	21	226	340	375	563	326	489	274	411	243	365	212	318
	22	218	328	360	542	313	469	263	395	234	351	203	305
	23	210	315	346	520	300	450	252	378	224	336	195	292
	24	202	303	331	498	287	430	242	362	214	322	186	279
	25	194	291	317	476	274	411	231	346	205	307	178	266
	26	186	279	302	455	261	392	220	330	195	293	169	254
	27	178	267	288	433	249	373	210	314	186	279	161	241
	28	170	255	274	412	236	354	199	299	176	265	152	229
	29	162	243	260	391	224	336	189	283	167	251	144	216
	30	154	231	247	371	212	317	179	268	158	237	136	204
	32	139	209	220	331	188	282	159	239	141	211	121	181
	34	124	186	195	293	167	250	141	212	125	187	107	160
	36	111	166	174	262	149	223	126	189	111	167	953	143
	38	99.5	149	156	235	133	200	113	169	99.7	150	85.6	128
	40	89.8	135	141	212	120	181	102	153	90.0	135	77.2	116
Properties													
M_n/Ω_b $\phi_b M_n$	kip-ft	71.7	108	153	230	129	194	103	155	88.6	133	73.5	110
$P_e(L_c)^2/10^4$, kip-in.^{2}		4720		7140		6330		5360		4730		4060	
r_m, in.		4.00		3.40		3.45		3.51		3.54		3.56	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS9–HSS8

Table IV-2A (continued)
Available Strength in Axial Compression, kips
Filled Square HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS9×9×				HSS8×8×							
		3⁄16		1⁄8^c		5⁄8		½		3⁄8		5⁄16	
t_{des}, in.		0.174		0.116		0.581		0.465		0.349		0.291	
Steel, lb/ft		22.2		15.0		59.3		48.9		37.7		31.8	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	279	418	204	305	491	738	422	633	350	526	312	469
	1	278	418	203	305	490	737	422	632	350	525	312	468
	2	278	416	203	304	489	735	420	630	349	523	311	466
	3	276	414	202	303	486	730	418	626	347	520	309	464
	4	274	411	200	301	482	724	414	621	344	516	307	460
	5	272	408	199	298	477	717	410	614	340	510	303	455
	6	269	403	197	295	471	707	404	606	336	504	299	449
	7	265	398	194	291	463	697	398	597	331	496	295	442
	8	261	392	191	287	455	684	391	586	325	488	290	435
	9	257	385	188	282	446	671	383	574	319	478	284	426
	10	252	378	185	277	436	656	374	561	312	467	278	417
	11	247	370	181	272	426	640	365	547	304	456	271	406
	12	241	361	177	266	414	623	355	532	296	444	264	396
	13	235	352	173	260	402	605	344	516	287	431	256	384
	14	229	343	169	253	390	586	333	500	278	417	248	372
	15	222	333	164	246	377	566	322	483	269	403	240	360
	16	215	323	159	239	363	546	310	465	259	389	231	347
	17	208	312	154	231	349	525	298	447	249	374	222	333
	18	201	301	149	224	335	504	286	428	239	359	213	320
	19	193	290	144	216	321	482	273	410	229	344	204	306
	20	186	279	139	208	307	461	261	391	219	328	195	293
	21	178	268	133	200	292	439	248	372	209	313	186	279
	22	171	256	128	192	278	417	235	353	198	298	177	265
	23	163	245	122	184	263	396	223	334	188	282	168	251
	24	156	234	117	175	249	374	211	316	178	267	159	238
	25	148	222	112	167	235	353	199	298	168	252	150	224
	26	141	211	106	159	221	333	187	281	158	237	141	211
	27	133	200	101	151	208	313	176	265	149	223	132	199
	28	126	189	95.8	144	195	293	165	249	139	209	124	186
	29	119	179	90.7	136	182	274	155	233	130	195	116	174
	30	112	168	85.7	129	170	256	145	217	122	182	108	162
	32	98.8	148	75.9	114	149	225	127	191	107	160	95.1	143
	34	87.5	131	67.2	101	132	199	113	169	94.6	142	84.2	126
	36	78.1	117	60.0	89.9	118	177	100	151	84.4	127	75.1	113
	38	70.1	105	53.8	80.7	106	159	90.2	136	75.8	114	67.4	101
	40	63.2	94.9	48.6	72.9	95.6	144	81.4	122	68.4	103	60.8	91.3
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	57.3	86.2	33.8	50.7	117	176	99.5	150	79.5	119	68.7	103
$P_e(L_c)^2/10^4$, kip-in.^{2}		3320		2550		4730		4220		3590		3200	
r_m, in.		3.59		3.62		2.99		3.04		3.10		3.13	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

[c] Shape is slender for F_y = 50 ksi; tabulated values have been adjusted accordingly.

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS8–HSS7

Table IV-2A (continued)

Available Strength in Axial Compression, kips

Filled Square HSS

F_y = 50 ksi
f'_c = 4 ksi

	Shape	HSS8×8×						HSS7×7×					
		¼		3⁄16		⅛		⅝		½		⅜	
	t_{des}, in.	0.233		0.174		0.116		0.581		0.465		0.349	
	Steel, lb/ft	25.8		19.6		13.3		50.8		42.1		32.6	
	Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	274	411	234	351	186	278	419	630	352	528	292	437
	1	274	410	233	350	185	278	418	629	352	528	291	437
	2	273	409	233	349	185	277	417	626	350	525	290	435
	3	271	406	231	347	183	275	413	621	347	521	288	431
	4	269	403	229	344	182	273	409	614	343	515	284	427
	5	266	399	227	340	180	269	403	606	338	508	280	421
	6	262	393	223	335	177	266	396	595	333	499	276	414
	7	258	387	220	330	174	261	388	583	326	489	270	405
	8	254	380	216	324	171	256	379	569	318	477	264	396
	9	249	373	211	317	167	250	369	554	309	464	257	386
	10	243	364	206	309	163	244	358	538	300	450	250	375
	11	237	355	201	301	159	238	346	520	290	435	242	363
	12	230	346	195	293	154	231	334	502	280	420	233	350
	13	224	335	189	284	149	223	321	482	269	404	224	336
	14	216	325	183	274	144	216	307	462	258	387	215	323
	15	209	314	176	264	138	208	294	441	247	371	206	308
	16	201	302	170	254	133	199	280	420	235	354	196	294
	17	194	290	163	244	127	191	265	399	224	336	186	279
	18	186	278	156	234	122	183	251	377	212	319	176	265
	19	177	266	149	223	116	174	237	356	200	301	167	250
	20	169	254	142	212	110	165	223	335	189	284	157	235
	21	161	242	134	202	105	157	209	314	177	267	147	221
	22	153	230	127	191	98.8	148	195	293	166	250	138	206
	23	145	218	120	181	93.2	140	182	273	155	233	128	192
	24	137	206	114	170	87.7	132	169	253	145	217	119	179
	25	129	194	107	160	82.3	123	156	234	134	201	110	166
	26	122	182	100	150	77.0	115	144	216	124	186	102	153
	27	114	171	93.7	141	71.8	108	133	201	115	173	94.6	142
	28	107	160	87.3	131	66.7	100	124	186	107	161	88.0	132
	29	99.5	149	81.4	122	62.2	93.3	116	174	99.6	150	82.0	123
	30	92.9	139	76.0	114	58.1	87.2	108	162	93.1	140	76.6	115
	32	81.7	123	66.8	100	51.1	76.6	95.0	143	81.8	123	67.4	101
	34	72.4	109	59.2	88.8	45.3	67.9	84.1	126	72.4	109	59.7	89.5
	36	64.5	96.8	52.8	79.2	40.4	60.6	75.1	113	64.6	97.1	53.2	79.8
	38	57.9	86.9	47.4	71.1	36.2	54.4	67.4	101	58.0	87.2	47.8	71.6
	40	52.3	78.4	42.8	64.2	32.7	49.1	60.8	91.4	52.3	78.7	43.1	64.7

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	57.0	85.7	44.6	67.1	29.8	44.8	86.2	130	73.5	110	59.2	89.0
$P_e(L_c)^2/10^4$, kip-in.^{2}			2750		2250		1720		2970		2650		2270	
r_m, in.			3.15		3.18		3.21		2.58		2.63		2.69	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS7–HSS6

Table IV-2A (continued)
Available Strength in Axial Compression, kips
Filled Square HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS7×7×								HSS6×6×			
		5⁄16		¼		3⁄16		⅛		⅝		½	
t_{des}, in.		0.291		0.233		0.174		0.116		0.581		0.465	
Steel, lb/ft		27.6		22.4		17.1		11.6		42.3		35.2	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	260	389	227	340	192	288	156	235	350	526	292	438
	1	259	389	226	340	192	287	156	234	350	525	291	437
	2	258	387	225	338	191	286	155	233	347	522	289	435
	3	256	384	224	335	189	284	154	231	343	516	286	430
	4	253	380	221	332	187	281	152	228	338	508	282	424
	5	250	375	218	327	184	277	150	225	331	498	277	416
	6	246	369	214	322	181	272	147	221	323	486	270	406
	7	241	361	210	315	177	266	144	216	314	472	263	395
	8	235	353	205	308	173	260	140	210	304	456	255	383
	9	229	344	200	300	169	253	136	204	292	439	246	369
	10	223	334	194	291	164	245	132	198	280	421	236	355
	11	216	324	188	282	158	237	127	191	267	402	226	339
	12	208	312	181	272	152	229	122	183	254	382	215	323
	13	200	301	174	262	146	220	117	176	240	361	204	306
	14	192	288	167	251	140	210	112	168	226	340	193	289
	15	184	276	160	240	134	201	106	159	212	318	181	272
	16	175	263	152	229	127	191	101	151	198	297	170	255
	17	167	250	145	217	121	181	95.2	143	184	276	158	238
	18	158	237	137	206	114	171	89.7	134	170	255	147	221
	19	149	224	130	194	108	161	84.1	126	156	235	136	204
	20	141	211	122	183	101	152	78.7	118	143	215	125	188
	21	132	198	114	172	94.7	142	73.3	110	130	196	115	172
	22	124	185	107	161	88.4	133	68.1	102	119	179	104	157
	23	115	173	99.9	150	82.2	123	63.0	94.6	109	163	95.6	144
	24	107	161	92.9	139	76.3	114	58.0	87.0	99.8	150	87.8	132
	25	99.5	149	85.9	129	70.3	106	53.5	80.2	92.0	138	80.9	122
	26	92.0	138	79.4	119	65.0	97.6	49.4	74.2	85.1	128	74.8	112
	27	85.3	128	73.7	111	60.3	90.5	45.8	68.8	78.9	119	69.4	104
	28	79.3	119	68.5	103	56.1	84.1	42.6	63.9	73.4	110	64.5	96.9
	29	73.9	111	63.9	95.8	52.3	78.4	39.7	59.6	68.4	103	60.1	90.4
	30	69.1	104	59.7	89.5	48.9	73.3	37.1	55.7	63.9	96.0	56.2	84.4
	32	60.7	91.1	52.4	78.7	42.9	64.4	32.6	49.0	56.2	84.4	49.4	74.2
	34	53.8	80.7	46.5	69.7	38.0	57.1	28.9	43.4	49.7	74.8	43.7	65.7
	36	48.0	72.0	41.4	62.2	33.9	50.9	25.8	38.7	44.4	66.7	39.0	58.6
	38	43.1	64.6	37.2	55.8	30.4	45.7	23.1	34.7				
	40	38.9	58.3	33.6	50.3	27.5	41.2	20.9	31.3				
Properties													
M_n/Ω_b $\phi_b M_n$	kip-ft	51.2	77.0	42.7	64.2	33.5	50.3	23.3	35.0	60.0	90.2	51.7	77.8
$P_e(L_c)^2/10^4$, kip-in.^{2}		2040		1760		1440		1100		1720		1550	
r_m, in.		2.72		2.75		2.77		2.80		2.17		2.23	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.

Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS6–HSS5½

Table IV-2A (continued)
Available Strength in Axial Compression, kips
Filled Square HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS6×6×										HSS5½×5½×	
		⅜		5⁄16		¼		3⁄16		⅛		⅜	
t_{des}, in.		0.349		0.291		0.233		0.174		0.116		0.349	
Steel, lb/ft		27.5		23.3		19.0		14.5		9.86		24.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	237	356	211	316	183	274	154	231	124	186	211	317
	1	237	355	210	315	183	274	153	230	124	186	210	316
	2	235	353	209	313	181	272	152	229	123	184	209	313
	3	233	349	207	310	180	269	151	226	122	182	206	309
	4	229	344	204	305	177	265	149	223	120	179	202	304
	5	225	337	200	300	174	261	146	219	117	176	198	297
	6	219	329	195	293	170	255	142	214	114	171	192	288
	7	213	320	190	285	165	248	138	208	111	166	186	279
	8	207	310	184	276	160	240	134	201	107	161	179	268
	9	199	299	177	266	155	232	129	194	103	155	171	257
	10	191	287	170	256	149	223	124	186	98.8	148	163	245
	11	183	274	163	245	142	213	119	178	94.2	141	154	232
	12	174	261	155	233	135	203	113	170	89.4	134	146	218
	13	165	248	147	221	129	193	107	161	84.5	127	137	205
	14	156	234	139	209	122	182	101	152	79.5	119	128	192
	15	146	220	131	196	114	172	95.1	143	74.4	112	119	179
	16	137	205	123	184	107	161	89.0	133	69.3	104	110	166
	17	128	191	114	172	100	150	82.9	124	64.3	96.5	102	153
	18	118	178	106	159	93.1	140	76.9	115	59.4	89.1	93.6	141
	19	109	164	98.3	147	86.1	129	71.1	107	54.6	81.9	85.6	129
	20	101	152	90.5	136	79.4	119	65.4	98.1	50.0	75.0	77.7	117
	21	92.9	140	83.0	125	72.9	109	59.9	89.8	45.4	68.2	70.5	106
	22	85.0	128	75.7	114	66.5	99.8	54.6	81.8	41.4	62.1	64.2	96.5
	23	77.8	117	69.2	104	60.8	91.3	49.9	74.9	37.9	56.8	58.7	88.3
	24	71.4	107	63.6	95.4	55.9	83.8	45.8	68.8	34.8	52.2	53.9	81.1
	25	65.8	98.9	58.6	87.9	51.5	77.2	42.2	63.4	32.1	48.1	49.7	74.7
	26	60.8	91.4	54.2	81.3	47.6	71.4	39.1	58.6	29.6	44.5	46.0	69.1
	27	56.4	84.8	50.2	75.4	44.2	66.2	36.2	54.3	27.5	41.2	42.6	64.1
	28	52.5	78.8	46.7	70.1	41.1	61.6	33.7	50.5	25.6	38.3	39.6	59.6
	29	48.9	73.5	43.6	65.3	38.3	57.4	31.4	47.1	23.8	35.7	36.9	55.5
	30	45.7	68.7	40.7	61.0	35.8	53.6	29.3	44.0	22.3	33.4	34.5	51.9
	32	40.2	60.4	35.8	53.7	31.4	47.1	25.8	38.7	19.6	29.4	30.3	45.6
	34	35.6	53.5	31.7	47.5	27.8	41.8	22.8	34.3	17.3	26.0	26.9	40.4
	36	31.7	47.7	28.3	42.4	24.8	37.3	20.4	30.6	15.5	23.2		
	38	28.5	42.8	25.4	38.1	22.3	33.4	18.3	27.4	13.9	20.8		

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	41.9	63.0	36.5	54.9	30.5	45.9	24.0	36.1	17.0	25.5	34.6	52.0
$P_e(L_c)^2/10^4$, kip-in.^{2}			1330		1200		1060		867		658		986	
r_m, in.			2.28		2.31		2.34		2.37		2.39		2.08	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.

Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS5½–HSS5

Table IV-2A (continued)
Available Strength in Axial Compression, kips
Filled Square HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape	HSS5½×5½×								HSS5×5×			
	5⁄16		¼		3⁄16		⅛		½		⅜	
t_{des}, in.	0.291		0.233		0.174		0.116		0.465		0.349	
Steel, lb/ft	21.2		17.3		13.3		9.01		28.4		22.4	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r												
0	187	281	162	243	136	204	109	163	236	355	186	279
1	187	280	162	243	135	203	108	163	235	353	185	278
2	185	278	161	241	134	202	108	161	233	350	183	275
3	183	275	159	238	133	199	106	159	229	345	181	271
4	180	270	156	234	130	196	104	156	224	337	177	265
5	176	264	152	229	128	191	102	152	218	328	172	258
6	171	256	148	223	124	186	98.7	148	210	316	166	250
7	165	248	144	215	120	180	95.3	143	202	303	160	240
8	159	239	138	208	116	173	91.6	137	192	289	153	229
9	153	229	133	199	111	166	87.5	131	182	274	145	218
10	145	218	127	190	106	158	83.2	125	172	258	137	206
11	138	207	120	180	100	150	78.7	118	161	241	129	193
12	130	195	113	170	94.5	142	74.0	111	149	224	120	180
13	122	183	107	160	88.8	133	69.2	104	138	207	111	167
14	114	171	99.6	149	82.9	124	64.4	96.6	127	190	103	154
15	106	159	92.7	139	77.1	116	59.6	89.4	115	173	94.0	141
16	97.9	147	85.8	129	71.3	107	54.9	82.3	105	157	85.6	129
17	90.1	135	79.1	119	65.6	98.4	50.3	75.4	94.2	142	77.5	116
18	82.4	124	72.5	109	60.1	90.1	45.8	68.7	84.1	126	69.6	105
19	75.1	113	66.1	99.1	54.7	82.1	41.4	62.1	75.5	113	62.5	93.9
20	68.0	102	59.9	89.8	49.5	74.3	37.4	56.1	68.1	102	56.4	84.8
21	61.6	92.7	54.3	81.4	44.9	67.4	33.9	50.9	61.8	92.9	51.2	76.9
22	56.2	84.4	49.5	74.2	40.9	61.4	30.9	46.4	56.3	84.6	46.6	70.0
23	51.4	77.2	45.3	67.9	37.4	56.1	28.3	42.4	51.5	77.4	42.6	64.1
24	47.2	70.9	41.6	62.3	34.4	51.6	26.0	38.9	47.3	71.1	39.2	58.9
25	43.5	65.4	38.3	57.5	31.7	47.5	23.9	35.9	43.6	65.5	36.1	54.2
26	40.2	60.4	35.4	53.1	29.3	43.9	22.1	33.2	40.3	60.6	33.4	50.2
27	37.3	56.0	32.8	49.3	27.2	40.7	20.5	30.8	37.4	56.2	30.9	46.5
28	34.7	52.1	30.5	45.8	25.3	37.9	19.1	28.6	34.8	52.2	28.8	43.2
29	32.3	48.6	28.5	42.7	23.5	35.3	17.8	26.7	32.4	48.7	26.8	40.3
30	30.2	45.4	26.6	39.9	22.0	33.0	16.6	24.9	30.3	45.5	25.1	37.7
32	26.5	39.9	23.4	35.1	19.3	29.0	14.6	21.9				
34	23.5	35.3	20.7	31.1	17.1	25.7	12.9	19.4				
36					15.3	22.9	11.5	17.3				
Properties												
M_n/Ω_b / $\phi_b M_n$, kip-ft	30.2	45.3	25.2	37.9	19.9	29.9	14.1	21.2	33.9	51.0	27.8	41.9
$P_e(L_c)^2/10^4$, kip-in.^{2}	891		786		650		491		813		708	
r_m, in.	2.11		2.13		2.16		2.19		1.82		1.87	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS5–HSS4½

Table IV-2A (continued)

Available Strength in Axial Compression, kips

Filled Square HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS5×5×								HSS4½×4½×			
		5⁄16		¼		3⁄16		⅛		½		⅜	
t_{des}, in.		0.291		0.233		0.174		0.116		0.465		0.349	
Steel, lb/ft		19.1		15.6		12.0		8.16		25.0		19.8	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	165	247	142	214	119	178	94.4	142	208	313	164	247
	1	164	246	142	213	118	178	94.1	141	207	311	163	246
	2	163	244	141	211	117	176	93.2	140	205	308	162	243
	3	160	240	139	208	116	173	91.7	138	201	302	159	238
	4	157	235	136	203	113	170	89.7	135	195	293	154	232
	5	152	228	132	198	110	165	87.2	131	188	283	149	224
	6	147	221	128	191	107	160	84.2	126	180	270	143	215
	7	141	212	123	184	102	154	80.8	121	171	256	136	205
	8	135	203	117	176	97.9	147	77.0	116	160	241	129	194
	9	128	192	111	167	93.0	139	73.0	109	150	225	121	182
	10	121	181	105	158	87.8	132	68.7	103	139	208	112	169
	11	113	170	98.7	148	82.4	124	64.3	96.4	127	191	104	156
	12	105	158	92.0	138	76.9	115	59.7	89.6	116	174	95.3	143
	13	97.6	146	85.3	128	71.3	107	55.2	82.8	105	157	86.7	130
	14	89.7	135	78.6	118	65.7	98.5	50.7	76.0	93.9	141	78.3	118
	15	82.0	123	72.0	108	60.2	90.3	46.2	69.3	83.4	125	70.2	105
	16	74.6	112	65.6	98.3	54.8	82.2	41.9	62.8	73.5	110	62.3	93.7
	17	67.8	102	59.3	89.0	49.6	74.4	37.7	56.6	65.1	97.8	55.2	83.0
	18	61.2	91.9	53.2	79.9	44.5	66.8	33.7	50.5	58.0	87.2	49.2	74.0
	19	54.9	82.5	47.8	71.7	40.0	60.0	30.2	45.3	52.1	78.3	44.2	66.4
	20	49.6	74.5	43.1	64.7	36.1	54.1	27.3	40.9	47.0	70.7	39.9	59.9
	21	44.9	67.6	39.1	58.7	32.7	49.1	24.7	37.1	42.6	64.1	36.2	54.4
	22	41.0	61.5	35.6	53.5	29.8	44.7	22.5	33.8	38.9	58.4	33.0	49.5
	23	37.5	56.3	32.6	48.9	27.3	40.9	20.6	30.9	35.5	53.4	30.2	45.3
	24	34.4	51.7	29.9	44.9	25.1	37.6	18.9	28.4	32.6	49.1	27.7	41.6
	25	31.7	47.7	27.6	41.4	23.1	34.6	17.5	26.2	30.1	45.2	25.5	38.4
	26	29.3	44.1	25.5	38.3	21.3	32.0	16.1	24.2	27.8	41.8	23.6	35.5
	27	27.2	40.9	23.7	35.5	19.8	29.7	15.0	22.5			21.9	32.9
	28	25.3	38.0	22.0	33.0	18.4	27.6	13.9	20.9				
	29	23.6	35.4	20.5	30.8	17.2	25.7	13.0	19.5				
	30	22.0	33.1	19.2	28.8	16.0	24.1	12.1	18.2				
	32			16.8	25.3	14.1	21.1	10.7	16.0				
Properties													
M_n/Ω_b / $\phi_b M_n$	kip-ft	24.3	36.5	20.5	30.8	16.2	24.3	11.5	17.3	26.3	39.5	21.8	32.8
$P_e(L_c)^2/10^4$, kip-in.^{2}		641		566		474		358		558		491	
r_m, in.		1.90		1.93		1.96		1.99		1.61		1.67	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.

Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS4½–HSS4

Table IV-2A (continued)
Available Strength in Axial Compression, kips
Filled Square HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS4½×4½×								HSS4×4×			
		5⁄16		¼		3⁄16		⅛		½		⅜	
t_{des}, in.		0.291		0.233		0.174		0.116		0.465		0.349	
Steel, lb/ft		17.0		13.9		10.7		7.31		21.6		17.3	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	143	214	124	185	103	154	80.9	121	180	271	143	215
	1	142	214	123	185	102	153	80.6	121	179	269	142	214
	2	141	211	122	183	101	152	79.7	120	176	265	140	211
	3	138	207	119	179	99.2	149	78.2	117	172	258	137	206
	4	134	202	116	175	96.6	145	76.1	114	166	249	132	199
	5	130	195	112	169	93.4	140	73.5	110	158	237	127	190
	6	124	187	108	162	89.7	135	70.4	106	149	224	120	180
	7	118	178	103	154	85.5	128	67.0	100	139	209	113	169
	8	112	168	97.0	146	80.9	121	63.2	94.8	128	193	105	157
	9	105	157	91.0	136	75.9	114	59.2	88.8	117	176	96.4	145
	10	97.3	146	84.7	127	70.8	106	55.0	82.5	106	160	87.9	132
	11	90.2	136	78.2	117	65.5	98.2	50.7	76.1	95.0	143	79.4	119
	12	82.9	125	71.7	108	60.1	90.2	46.4	69.6	84.1	126	71.0	107
	13	75.7	114	65.3	97.9	54.8	82.2	42.2	63.2	73.6	111	62.8	94.4
	14	68.6	103	58.9	88.4	49.6	74.4	38.0	57.0	63.7	95.8	55.0	82.7
	15	61.7	92.8	52.8	79.2	44.5	66.8	34.0	50.9	55.5	83.5	47.9	72.0
	16	55.1	82.9	46.9	70.3	39.7	59.5	30.1	45.1	48.8	73.3	42.1	63.3
	17	48.8	73.4	41.5	62.4	35.1	52.7	26.6	40.0	43.2	65.0	37.3	56.1
	18	43.6	65.5	37.0	55.6	31.3	47.0	23.8	35.6	38.6	58.0	33.3	50.0
	19	39.1	58.8	33.3	49.9	28.1	42.2	21.3	32.0	34.6	52.0	29.9	44.9
	20	35.3	53.0	30.0	45.1	25.4	38.1	19.2	28.9	31.2	46.9	27.0	40.5
	21	32.0	48.1	27.2	40.9	23.0	34.5	17.5	26.2	28.3	42.6	24.4	36.7
	22	29.2	43.8	24.8	37.3	21.0	31.5	15.9	23.9	25.8	38.8	22.3	33.5
	23	26.7	40.1	22.7	34.1	19.2	28.8	14.6	21.8	23.6	35.5	20.4	30.6
	24	24.5	36.8	20.8	31.3	17.6	26.4	13.4	20.0			18.7	28.1
	25	22.6	34.0	19.2	28.8	16.2	24.4	12.3	18.5				
	26	20.9	31.4	17.8	26.7	15.0	22.5	11.4	17.1				
	27	19.4	29.1	16.5	24.7	13.9	20.9	10.6	15.8				
	28	18.0	27.1	15.3	23.0	13.0	19.4	9.82	14.7				
	29					12.1	18.1	9.15	13.7				

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	19.2	28.8	16.2	24.3	12.8	19.3	9.17	13.8	19.8	29.7	16.6	25.0
$P_e(L_c)^2/10^4$, kip-in.^{2}			446		394		333		253		362		325	
r_m, in.			1.70		1.73		1.75		1.78		1.41		1.47	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS4–HSS3½

Table IV-2A (continued)
Available Strength in Axial Compression, kips
Filled Square HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS4×4×								HSS3½×3½×			
		5⁄16		¼		3⁄16		⅛		⅜		5⁄16	
t_{des}, in.		0.291		0.233		0.174		0.116		0.349		0.291	
Steel, lb/ft		14.8		12.2		9.42		6.46		14.7		12.7	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	123	184	105	158	87.1	131	68.4	103	122	184	105	158
	1	122	184	105	157	86.7	130	68	102	122	183	105	157
	2	120	181	103	155	85.5	128	67.1	101	119	179	103	154
	3	118	177	101	151	83.5	125	65.5	98.2	115	173	99.6	150
	4	114	171	97.6	146	80.8	121	63.3	94.9	110	166	95.2	143
	5	109	164	93.4	140	77.5	116	60.6	90.9	104	156	90.0	135
	6	103	156	88.6	133	73.6	110	57.5	86.2	96.4	145	83.9	126
	7	97.3	146	83.2	125	69.2	104	54.0	81.0	88.5	133	77.3	116
	8	90.6	136	77.4	116	64.5	96.7	50.2	75.4	80.1	120	70.3	106
	9	83.6	126	71.3	107	59.5	89.3	46.3	69.4	71.6	108	63.1	94.9
	10	76.4	115	65.0	97.6	54.5	81.7	42.2	63.4	63.1	94.8	56.0	84.1
	11	69.2	104	58.8	88.1	49.3	74.0	38.2	57.3	54.9	82.5	49.0	73.7
	12	62.0	93.2	52.6	78.9	44.3	66.4	34.2	51.3	47.1	70.7	42.4	63.7
	13	55.1	82.8	46.7	70.2	39.4	59.1	30.3	45.4	40.1	60.3	36.2	54.4
	14	48.5	72.8	41.3	62.1	34.7	52.0	26.6	39.9	34.6	52.0	31.2	46.9
	15	42.2	63.5	36.1	54.3	30.2	45.4	23.2	34.7	30.1	45.3	27.2	40.8
	16	37.1	55.8	31.7	47.7	26.6	39.9	20.4	30.5	26.5	39.8	23.9	35.9
	17	32.9	49.4	28.1	42.3	23.5	35.3	18.0	27.1	23.5	35.2	21.2	31.8
	18	29.3	44.1	25.1	37.7	21.0	31.5	16.1	24.1	20.9	31.4	18.9	28.4
	19	26.3	39.6	22.5	33.8	18.8	28.3	14.4	21.7	18.8	28.2	16.9	25.5
	20	23.8	35.7	20.3	30.5	17.0	25.5	13.0	19.5	16.9	25.5	15.3	23
	21	21.5	32.4	18.4	27.7	15.4	23.1	11.8	17.7	15.4	23.1	13.9	20.8
	22	19.6	29.5	16.8	25.2	14.1	21.1	10.8	16.2				
	23	18.0	27.0	15.4	23.1	12.9	19.3	9.85	14.8				
	24	16.5	24.8	14.1	21.2	11.8	17.7	9.05	13.6				
	25			13.0	19.5	10.9	16.3	8.34	12.5				
	26							7.71	11.6				
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	14.7	22.0	12.4	18.7	9.92	14.9	7.12	10.7	12.1	18.2	10.8	16.2
$P_e(L_c)^2/10^4$, kip-in.^{2}		296		263		223		171		201		185	
r_m, in.		1.49		1.52		1.55		1.58		1.26		1.29	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS3½–HSS3

Table IV-2A (continued)
Available Strength in Axial Compression, kips
Filled Square HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS3½×3½×						HSS3×3×					
		¼		3⁄16		⅛		⅜		5⁄16		¼	
t_{des}, in.		0.233		0.174		0.116		0.349		0.291		0.233	
Steel, lb/ft		10.5		8.15		5.61		12.2		10.6		8.81	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	88.3	132	72.8	109	56.6	85.0	101	153	88	132	73.1	110
	1	87.8	132	72.4	109	56.3	84.4	101	151	87.2	131	72.4	109
	2	86.1	129	71.1	107	55.2	82.9	97.8	147	84.9	128	70.6	106
	3	83.4	125	68.9	103	53.6	80.3	93.3	140	81.2	122	67.6	102
	4	79.7	120	65.9	98.9	51.3	76.9	87.4	131	76.2	115	63.7	95.8
	5	75.2	113	62.4	93.5	48.5	72.8	80.3	121	70.2	106	59.0	88.7
	6	70.1	105	58.2	87.4	45.3	68.0	72.4	109	63.6	95.6	53.7	80.7
	7	64.8	97.4	53.7	80.6	41.8	62.7	64.1	96.4	56.6	85.0	48.1	72.2
	8	59.2	89.0	48.9	73.4	38.1	57.1	55.7	83.7	49.4	74.2	42.3	63.5
	9	53.4	80.3	44.0	66.0	34.3	51.4	47.5	71.4	42.4	63.7	36.6	55.0
	10	47.6	71.6	39.1	58.7	30.5	45.7	39.8	59.8	35.7	53.6	31.1	46.7
	11	41.9	63.0	34.3	51.5	26.7	40.1	32.9	49.4	29.6	44.5	25.9	39.0
	12	36.5	54.9	29.8	44.6	23.2	34.8	27.6	41.5	24.9	37.4	21.8	32.8
	13	31.3	47.1	25.4	38.2	19.8	29.7	23.5	35.4	21.2	31.8	18.6	27.9
	14	27.0	40.6	21.9	32.9	17.1	25.6	20.3	30.5	18.3	27.4	16.0	24.1
	15	23.5	35.4	19.1	28.7	14.9	22.3	17.7	26.6	15.9	23.9	13.9	21.0
	16	20.7	31.1	16.8	25.2	13.1	19.6	15.5	23.3	14.0	21.0	12.3	18.4
	17	18.3	27.5	14.9	22.3	11.6	17.4	13.8	20.7	12.4	18.6	10.9	16.3
	18	16.3	24.6	13.3	19.9	10.3	15.5			11.0	16.6	9.69	14.6
	19	14.7	22.0	11.9	17.9	9.28	13.9						
	20	13.2	19.9	10.8	16.1	8.38	12.6						
	21	12.0	18.0	9.75	14.6	7.60	11.4						
	22	10.9	16.4	8.89	13.3	6.92	10.4						

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	9.22	13.9	7.39	11.1	5.32	7.99	8.34	12.5	7.50	11.3	6.48	9.74
$P_e(L_c)^2/10^4$, kip-in.^{2}			166		141		110		115		107		96.9	
r_m, in.			1.32		1.35		1.37		1.06		1.08		1.11	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS3–HSS2½

Table IV-2A (continued)

Available Strength in Axial Compression, kips

Filled Square HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape		HSS3×3×				HSS2½×2½×							
		3⁄16		1⁄8		5⁄16		1⁄4		3⁄16		1⁄8	
t_{des}, in,		0.174		0.116		0.291		0.233		0.174		0.116	
Steel, lb/ft		6.87		4.75		8.45		7.11		5.59		3.90	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	59.2	88.7	45.5	68.3	70.4	106	59	88.6	46.3	69.5	35.5	53.2
	1	58.7	88	45.1	67.7	69.4	104	58.2	87.5	45.7	68.6	35.0	52.6
	2	57.2	85.7	44.0	66.0	66.6	100	56.0	84.2	44.0	66.1	33.8	50.7
	3	54.8	82.1	42.2	63.3	62.3	93.6	52.6	79.0	41.4	62.2	31.8	47.7
	4	51.6	77.4	39.8	59.7	56.6	85.1	48.1	72.3	38.1	57.2	29.2	43.9
	5	47.7	71.6	36.9	55.4	50.1	75.3	42.9	64.4	34.2	51.3	26.2	39.3
	6	43.4	65.1	33.7	50.6	43.1	64.8	37.2	56.0	29.9	45.0	23.0	34.4
	7	38.8	58.3	30.2	45.4	36.1	54.3	31.5	47.4	25.6	38.5	19.6	29.4
	8	34.2	51.2	26.7	40.0	29.5	44.3	26.0	39.1	21.4	32.2	16.4	24.5
	9	29.5	44.3	23.2	34.8	23.5	35.2	20.9	31.5	17.4	26.2	13.3	19.9
	10	25.2	37.8	19.8	29.7	19.0	28.6	17.0	25.5	14.1	21.2	10.8	16.2
	11	21.2	31.8	16.6	24.8	15.7	23.6	14.0	21.1	11.7	17.5	8.90	13.3
	12	17.8	26.8	13.9	20.9	13.2	19.8	11.8	17.7	9.80	14.7	7.48	11.2
	13	15.2	22.8	11.9	17.8	11.2	16.9	10.0	15.1	8.35	12.6	6.37	9.56
	14	13.1	19.7	10.2	15.3	9.69	14.6	8.65	13.0	7.20	10.8	5.49	8.24
	15	11.4	17.1	8.91	13.4			7.53	11.3	6.27	9.43	4.79	7.18
	16	10.0	15.1	7.83	11.7							4.21	6.31
	17	8.87	13.3	6.93	10.4								
	18	7.91	11.9	6.19	9.28								
	19	7.10	10.7	5.55	8.33								
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	5.23	7.86	3.81	5.73	4.83	7.25	4.22	6.35	3.47	5.21	2.55	3.83
$P_e(L_c)^2/10^4$, kip-in.^{2}		83.1		65.8		55.4		50.9		44.1		35.4	
r_m, in.		1.14		1.17		0.880		0.908		0.937		0.965	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.

Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS2¼–HSS2

Table IV-2A (continued)
Available Strength in Axial Compression, kips
Filled Square HSS

F_y = 50 ksi
f'_c = 4 ksi

Shape	HSS2¼×2¼×						HSS2×2×					
	¼		3⁄16		⅛		¼		3⁄16		⅛	
t_{des}, in.	0.233		0.174		0.116		0.233		0.174		0.116	
Steel, lb/ft	6.26		4.96		3.48		5.41		4.32		3.05	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r												
0	52.1	78.3	41.0	61.6	30.8	46.2	45.2	67.9	35.6	53.5	26.3	39.4
1	51.3	77.0	40.4	60.7	30.3	45.5	44.3	66.5	34.9	52.5	25.8	38.7
2	48.8	73.4	38.6	58.0	29.0	43.5	41.5	62.4	32.9	49.5	24.3	36.5
3	45.0	67.7	35.8	53.8	26.9	40.3	37.3	56.1	29.9	44.9	22.1	33.1
4	40.2	60.4	32.2	48.4	24.2	36.3	32.2	48.4	26.0	39.1	19.3	29.0
5	34.7	52.2	28.1	42.3	21.1	31.7	26.6	40.0	21.8	32.8	16.2	24.3
6	29.1	43.7	23.8	35.8	17.9	26.9	21.0	31.6	17.6	26.4	13.1	19.7
7	23.5	35.4	19.6	29.4	14.7	22.1	15.9	24.0	13.6	20.5	10.3	15.5
8	18.4	27.7	15.6	23.4	11.7	17.6	12.2	18.3	10.4	15.7	7.93	11.9
9	14.6	21.9	12.3	18.5	9.26	13.9	9.64	14.5	8.24	12.4	6.27	9.42
10	11.8	17.7	9.97	15.0	7.50	11.3	7.81	11.7	6.67	10.0	5.08	7.63
11	9.75	14.7	8.24	12.4	6.20	9.30	6.46	9.70	5.52	8.29	4.20	6.31
12	8.19	12.3	6.92	10.4	5.21	7.81			4.63	6.97	3.53	5.30
13	6.98	10.5	5.90	8.87	4.44	6.66						
14					3.83	5.74						
Properties												
M_n/Ω_b | $\phi_b M_n$ | kip-ft	3.30	4.96	2.72	4.09	2.02	3.03	2.47	3.72	2.07	3.11	1.55	2.33
$P_e(L_c)^2/10^4$, kip-in.^{2}	34.9		30.6		24.6		22.7		20.2		16.4	
r_m, in.	0.806		0.835		0.863		0.704		0.733		0.761	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

5

COMPOSITE
HSS16–HSS14

Table IV-2B

Available Strength in Axial Compression, kips

Filled Square HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS16×16×						HSS14×14×					
		½		⅜		5⁄16		⅝		½		⅜	
t_{des}, in.		0.465		0.349		0.291		0.581		0.465		0.349	
Steel, lb/ft		103		78.5		65.9		110		89.7		68.3	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	1190	1780	1030	1550	957	1440	1110	1660	978	1470	843	1260
	1	1190	1780	1030	1550	957	1440	1110	1660	977	1470	843	1260
	2	1190	1780	1030	1550	956	1430	1110	1660	976	1460	842	1260
	3	1190	1780	1030	1550	954	1430	1100	1650	974	1460	840	1260
	4	1180	1780	1030	1540	952	1430	1100	1650	971	1460	838	1260
	5	1180	1770	1030	1540	949	1420	1100	1640	968	1450	834	1250
	6	1180	1760	1020	1530	946	1420	1090	1640	963	1450	830	1250
	7	1170	1760	1020	1530	941	1410	1090	1630	958	1440	826	1240
	8	1170	1750	1010	1520	936	1400	1080	1620	953	1430	821	1230
	9	1160	1740	1010	1510	931	1400	1070	1610	946	1420	815	1220
	10	1150	1730	1000	1500	925	1390	1060	1600	939	1410	808	1210
	11	1140	1720	994	1490	918	1380	1050	1580	931	1400	801	1200
	12	1140	1700	986	1480	911	1370	1050	1570	922	1380	793	1190
	13	1130	1690	978	1470	903	1350	1030	1550	913	1370	785	1180
	14	1120	1680	969	1450	894	1340	1020	1540	903	1350	776	1160
	15	1110	1660	960	1440	885	1330	1010	1520	892	1340	766	1150
	16	1100	1650	950	1430	876	1310	1000	1500	881	1320	756	1130
	17	1090	1630	940	1410	866	1300	987	1480	869	1300	746	1120
	18	1070	1610	929	1390	855	1280	973	1460	857	1290	735	1100
	19	1060	1590	917	1380	844	1270	959	1440	844	1270	723	1080
	20	1050	1570	905	1360	833	1250	944	1420	831	1250	711	1070
	21	1030	1550	893	1340	821	1230	929	1390	817	1230	699	1050
	22	1020	1530	880	1320	809	1210	913	1370	803	1200	686	1030
	23	1010	1510	867	1300	796	1190	897	1340	789	1180	673	1010
	24	991	1490	854	1280	784	1180	880	1320	774	1160	660	990
	25	976	1460	840	1260	770	1160	863	1290	758	1140	647	970
	26	960	1440	826	1240	757	1140	846	1270	743	1110	633	949
	27	944	1420	811	1220	743	1110	828	1240	727	1090	619	928
	28	928	1390	796	1190	729	1090	810	1220	711	1070	604	906
	29	911	1370	781	1170	714	1070	792	1190	695	1040	590	885
	30	895	1340	766	1150	700	1050	773	1160	678	1020	575	863
	32	860	1290	735	1100	670	1010	736	1100	645	967	546	818
	34	825	1240	703	1050	640	960	698	1050	611	917	516	774
	36	789	1180	671	1010	610	915	660	991	577	866	486	729
	38	753	1130	639	958	579	869	623	934	544	815	456	685
	40	717	1080	606	910	549	823	585	878	510	765	427	641
Properties													
M_n/Ω_b / $\phi_b M_n$	kip-ft	463	696	364	546	310	466	415	624	346	521	273	410
$P_e(L_c)^2/10^4$, kip-in.^{2}		45300		37300		33200		33500		29000		23900	
r_m, in.		6.31		6.37		6.39		5.44		5.49		5.55	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

5

COMPOSITE
HSS14–HSS12

Table IV-2B (continued)
Available Strength in Axial Compression, kips
Filled Square HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS14×14×		HSS12×12×									
		5⁄16		5⁄8		1⁄2		3⁄8		5⁄16		1⁄4	
t_{des}, in,		**0.291**		**0.581**		**0.465**		**0.349**		**0.291**		**0.233**	
Steel, lb/ft		**57.4**		**93.3**		**76.1**		**58.1**		**48.9**		**39.4**	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length, L_c (ft), with respect to the least radius of gyration, r	**0**	775	1160	891	1340	783	1170	671	1010	612	918	553	829
	1	775	1160	891	1340	782	1170	671	1010	612	917	552	828
	2	774	1160	890	1330	781	1170	670	1000	610	916	551	827
	3	772	1160	887	1330	779	1170	668	1000	609	913	550	824
	4	769	1150	884	1330	776	1160	665	998	606	909	547	821
	5	766	1150	879	1320	772	1160	662	993	603	905	544	816
	6	763	1140	874	1310	767	1150	658	987	599	899	541	811
	7	758	1140	868	1300	762	1140	653	979	595	892	536	805
	8	753	1130	861	1290	756	1130	647	971	590	884	531	797
	9	748	1120	853	1280	749	1120	641	962	584	876	526	789
	10	742	1110	844	1270	741	1110	634	952	577	866	520	780
	11	735	1100	834	1250	733	1100	627	940	570	855	513	770
	12	727	1090	824	1240	724	1090	619	928	563	844	506	759
	13	719	1080	813	1220	714	1070	610	915	555	832	499	748
	14	711	1070	801	1200	703	1060	601	901	546	819	490	736
	15	702	1050	788	1180	692	1040	591	887	537	805	482	723
	16	692	1040	775	1160	681	1020	581	871	527	791	473	709
	17	682	1020	761	1140	669	1000	570	855	517	776	463	695
	18	672	1010	747	1120	656	984	559	838	507	760	454	680
	19	661	992	732	1100	643	965	547	821	496	744	443	665
	20	650	975	716	1070	630	944	535	803	485	727	433	650
	21	638	957	700	1050	616	923	523	785	473	710	422	634
	22	626	939	684	1030	601	902	511	766	462	693	411	617
	23	614	921	667	1000	587	880	498	747	450	675	400	601
	24	601	902	651	976	572	858	485	727	438	656	389	584
	25	588	883	633	950	557	835	471	707	425	638	378	566
	26	575	863	616	924	542	813	458	687	413	619	366	549
	27	562	843	598	897	526	790	445	667	400	601	354	532
	28	549	823	581	871	511	766	431	646	388	582	343	514
	29	535	803	563	844	495	743	417	626	375	563	331	497
	30	521	782	545	817	480	719	404	605	362	544	319	479
	32	494	740	509	764	448	673	376	564	337	506	296	444
	34	466	699	474	711	417	626	349	524	312	468	273	410
	36	438	657	439	658	387	580	323	484	288	432	251	376
	38	410	615	405	607	357	535	297	445	264	396	229	344
	40	383	574	372	557	328	492	272	408	241	362	208	312
Properties													
M_n/Ω_b | **$\phi_b M_n$**	**kip-ft**	233	351	296	445	247	372	195	294	168	252	139	208
$P_e(L_c)^2/10^4$, kip-in.^{2}		21200		19600		17300		14300		12700		10900	
r_m, in.		5.58		4.62		4.68		4.73		4.76		4.79	

ASD	**LRFD**
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

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COMPOSITE
HSS12–HSS10

Table IV-2B (continued)
Available Strength in Axial Compression, kips
Filled Square HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS12×12×		HSS10×10×									
		3⁄16		5⁄8		½		3⁄8		5⁄16		¼	
t_{des}, in.		0.174		0.581		0.465		0.349		0.291		0.233	
Steel, lb/ft		29.8		76.3		62.5		47.9		40.4		32.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	470	706	690	1040	604	907	514	770	466	699	417	626
	1	470	705	690	1030	604	906	513	770	465	698	417	625
	2	469	704	688	1030	603	904	512	768	464	696	416	623
	3	468	702	685	1030	600	900	510	765	462	694	414	621
	4	466	699	682	1020	597	895	507	761	460	690	411	617
	5	463	695	677	1020	593	889	504	755	456	685	408	612
	6	460	690	671	1010	588	881	499	749	452	679	405	607
	7	456	684	664	996	582	872	494	741	448	671	400	600
	8	452	678	656	984	575	862	488	732	442	663	395	593
	9	447	671	647	971	567	851	482	723	436	654	389	584
	10	442	663	638	956	559	838	475	712	429	644	383	575
	11	436	654	627	940	550	824	467	700	422	633	376	565
	12	430	645	616	923	540	810	458	687	414	621	369	554
	13	423	635	603	905	529	794	449	674	406	609	361	542
	14	416	624	591	886	518	777	440	660	397	596	353	530
	15	409	613	577	866	506	760	430	645	388	582	345	517
	16	401	601	563	844	494	741	419	629	378	567	336	504
	17	393	589	548	823	482	722	408	613	368	552	327	490
	18	384	576	533	800	469	703	397	596	358	537	317	476
	19	375	563	518	777	455	683	386	579	347	521	307	461
	20	366	550	502	753	441	662	374	561	336	505	297	446
	21	357	536	486	729	427	641	362	543	325	488	287	431
	22	348	521	469	704	413	620	350	525	314	471	277	415
	23	338	507	453	679	399	598	338	507	303	454	267	400
	24	328	492	436	654	384	577	325	488	291	437	256	384
	25	318	478	420	629	370	555	313	469	280	420	246	369
	26	308	463	403	604	355	533	301	451	269	403	235	353
	27	298	447	386	579	341	511	288	432	257	386	225	338
	28	288	432	370	555	326	490	276	414	246	369	215	322
	29	278	417	353	530	312	468	264	396	235	352	205	307
	30	268	402	337	506	298	447	252	378	224	336	195	292
	32	248	372	305	458	270	405	228	342	202	304	175	263
	34	228	343	275	412	244	365	205	308	182	273	157	235
	36	209	314	245	368	218	327	184	275	162	243	140	209
	38	191	286	220	330	195	293	165	247	145	218	125	188
	40	173	259	199	298	176	265	149	223	131	197	113	170

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	102	154	196	295	165	249	131	198	113	170	93.9	141
$P_e(L_c)^2/10^4$, kip-in.^{2}			9080		10400		9270		7810		6900		5940	
r_m, in.			4.82		3.80		3.86		3.92		3.94		3.97	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

5

COMPOSITE
HSS10–HSS9

Table IV-2B (continued)
Available Strength in Axial Compression, kips
Filled Square HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS10×10×		HSS9×9×									
		3⁄16		5⁄8		1⁄2		3⁄8		5⁄16		1⁄4	
t_{des}, in,		**0.174**		**0.581**		**0.465**		**0.349**		**0.291**		**0.233**	
Steel, lb/ft		**24.7**		**67.8**		**55.7**		**42.8**		**36.1**		**29.2**	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length, L_c (ft), with respect to the least radius of gyration, r	**0**	367	550	597	896	520	781	441	662	398	598	355	533
	1	367	550	597	895	520	780	441	661	398	597	355	533
	2	366	548	595	893	518	778	440	659	397	595	354	531
	3	364	546	592	888	516	774	437	656	395	592	352	528
	4	362	542	588	882	512	769	434	652	392	588	350	524
	5	359	538	583	874	508	762	431	646	389	583	346	520
	6	355	533	576	865	503	754	426	639	385	577	343	514
	7	351	527	569	854	496	744	421	631	380	569	338	507
	8	346	520	561	841	489	734	415	622	374	561	333	499
	9	341	512	551	827	481	722	408	612	368	552	327	491
	10	335	503	541	812	472	708	401	601	361	541	321	481
	11	329	494	530	795	463	694	393	589	354	530	314	471
	12	322	483	518	777	452	679	384	576	346	518	307	460
	13	315	473	505	758	442	662	375	562	337	506	299	449
	14	308	461	492	738	430	645	365	548	328	493	291	436
	15	300	449	478	717	418	627	355	532	319	479	282	424
	16	291	437	463	695	406	609	344	517	309	464	274	410
	17	283	424	449	673	393	590	334	500	300	449	265	397
	18	274	411	433	650	380	570	323	484	289	434	255	383
	19	265	398	418	626	366	550	311	467	279	418	246	369
	20	256	384	402	603	353	529	300	450	268	403	236	354
	21	247	370	386	579	339	509	288	432	258	387	226	340
	22	237	356	370	554	325	488	276	414	247	371	217	325
	23	228	342	353	530	311	467	265	397	236	355	207	310
	24	218	328	337	506	297	446	253	379	226	338	197	296
	25	209	313	321	482	283	425	241	362	215	322	188	281
	26	200	299	306	458	270	405	230	344	204	307	178	267
	27	190	285	290	435	256	384	218	327	194	291	169	253
	28	181	272	274	412	243	364	207	310	184	276	159	239
	29	172	258	260	391	230	345	196	293	174	261	150	226
	30	163	245	247	371	217	325	185	277	164	246	142	212
	32	146	219	220	331	192	288	164	245	145	217	125	187
	34	129	194	195	293	170	255	145	217	128	192	110	166
	36	115	173	174	262	152	227	129	194	114	171	98.5	148
	38	104	155	156	235	136	204	116	174	103	154	88.4	133
	40	93.4	140	141	212	123	184	105	157	92.6	139	79.8	120

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	72.8	109	155	233	131	197	104	157	89.9	135	74.6	112
$P_e(L_c)^2/10^4$, kip-in.^{2}			4910		7260		6450		5500		4870		4190	
r_m, in.			4.00		3.40		3.45		3.51		3.54		3.56	

ASD	**LRFD**
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS9–HSS8

Table IV-2B (continued)
Available Strength in Axial Compression, kips
Filled Square HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape	HSS9×9×		HSS8×8×									
	3⁄16		1⁄8^c		5⁄8		1⁄2		3⁄8		5⁄16	
t_{des}, in.	0.174		0.116		0.581		0.465		0.349		0.291	
Steel, lb/ft	22.2		15.0		59.3		48.9		37.7		31.8	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r												
0	311	466	230	346	509	763	443	665	373	560	336	504
1	310	465	230	345	508	762	443	664	373	559	335	503
2	309	464	229	344	506	759	441	662	371	557	334	501
3	307	461	228	342	503	754	438	658	369	553	332	498
4	305	458	227	340	498	748	435	652	366	549	329	494
5	302	453	225	337	493	739	430	645	362	543	326	488
6	299	448	222	333	486	729	424	636	357	536	321	482
7	294	442	219	328	478	717	417	626	351	527	316	474
8	290	435	216	323	469	703	409	614	345	518	310	465
9	284	427	212	318	459	688	401	601	338	507	304	456
10	279	418	208	312	448	672	391	587	330	495	297	445
11	272	408	203	305	436	654	381	572	322	483	289	434
12	266	398	198	298	423	635	371	556	313	469	281	422
13	259	388	193	290	410	615	359	539	303	455	273	409
14	251	377	188	282	396	594	347	521	294	440	264	396
15	243	365	182	274	382	572	335	503	283	425	254	382
16	235	353	177	265	367	550	322	483	273	409	245	367
17	227	340	171	256	352	528	309	464	262	393	235	353
18	219	328	165	247	336	504	296	444	251	377	225	338
19	210	315	158	238	321	482	283	424	240	360	215	323
20	201	302	152	228	307	461	269	404	229	343	205	308
21	192	289	146	219	292	439	256	384	218	326	195	293
22	184	276	139	209	278	417	243	364	206	310	185	277
23	175	262	133	199	263	396	229	344	195	293	175	262
24	166	249	127	190	249	374	216	325	185	277	165	248
25	158	236	120	181	235	353	204	305	174	261	155	233
26	149	224	114	171	221	333	191	287	163	245	146	219
27	141	211	108	162	208	313	179	268	153	230	137	205
28	133	199	102	153	195	293	167	250	143	214	128	191
29	125	187	96.1	144	182	274	155	233	133	200	119	178
30	117	175	90.4	136	170	256	145	218	124	187	111	167
32	103	154	79.5	119	149	225	128	191	109	164	97.7	147
34	90.9	136	70.4	106	132	199	113	170	96.9	145	86.5	130
36	81.1	122	62.8	94.2	118	177	101	151	86.4	130	77.2	116
38	72.8	109	56.4	84.5	106	159	90.5	136	77.6	116	69.3	104
40	65.7	98.5	50.9	76.3	95.6	144	81.7	123	70.0	105	62.5	93.8
Properties												
M_n/Ω_b / $\phi_b M_n$, kip-ft	58.3	87.6	34.2	51.4	118	178	101	151	80.5	121	69.7	105
$P_e(L_c)^2/10^4$, kip-in.^{2}	3450		2670		4800		4290		3680		3290	
r_m, in.	3.59		3.62		2.99		3.04		3.10		3.13	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

[c] Shape is slender for F_y = 50 ksi; tabulated values have been adjusted accordingly.

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

5

COMPOSITE
HSS8–HSS7

Table IV-2B (continued)
Available Strength in Axial Compression, kips
Filled Square HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS8×8×						HSS7×7×					
		¼		3⁄16		⅛		⅝		½		⅜	
t_{des}, in.		0.233		0.174		0.116		0.581		0.465		0.349	
Steel, lb/ft		25.8		19.6		13.3		50.8		42.1		32.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	298	447	259	388	209	314	422	633	368	552	308	463
	1	298	446	258	387	209	314	421	632	367	551	308	462
	2	297	445	257	386	208	312	419	628	365	548	306	460
	3	295	442	256	383	207	310	415	623	362	544	304	456
	4	292	438	253	380	205	307	411	616	358	537	301	451
	5	289	433	250	375	202	303	404	606	353	530	296	444
	6	285	427	246	370	199	298	397	595	347	520	291	437
	7	280	420	242	363	195	293	388	583	339	509	285	428
	8	275	412	237	356	191	287	379	569	331	497	278	417
	9	269	403	232	348	187	280	369	554	322	483	271	406
	10	263	394	226	339	182	273	358	538	312	468	263	394
	11	256	383	220	330	177	265	346	520	301	452	254	381
	12	248	372	213	320	171	256	334	502	290	435	245	367
	13	241	361	206	309	165	248	321	482	278	418	235	353
	14	232	349	199	298	159	238	307	462	266	400	225	338
	15	224	336	191	287	153	229	294	441	254	381	215	322
	16	215	323	184	275	146	219	280	420	241	362	204	307
	17	207	310	176	263	139	209	265	399	228	343	194	291
	18	198	296	168	251	133	199	251	377	216	324	183	275
	19	189	283	160	239	126	189	237	356	203	304	173	259
	20	179	269	151	227	119	179	223	335	190	285	162	243
	21	170	256	143	215	113	169	209	314	178	267	152	228
	22	161	242	135	203	106	159	195	293	166	250	142	212
	23	152	229	127	191	99.5	149	182	273	155	233	132	198
	24	144	215	120	179	93.1	140	169	253	145	217	122	183
	25	135	202	112	168	86.9	130	156	234	134	201	113	169
	26	126	190	105	157	80.8	121	144	216	124	186	104	156
	27	118	177	97.3	146	75.0	112	133	201	115	173	96.6	145
	28	110	165	90.5	136	69.7	105	124	186	107	161	89.8	135
	29	103	154	84.3	126	65.0	97.5	116	174	99.6	150	83.7	126
	30	95.8	144	78.8	118	60.7	91.1	108	162	93.1	140	78.3	117
	32	84.2	126	69.3	104	53.4	80.1	95.0	143	81.8	123	68.8	103
	34	74.6	112	61.4	92.0	47.3	70.9	84.1	126	72.4	109	60.9	91.4
	36	66.6	99.8	54.7	82.1	42.2	63.3	75.1	113	64.6	97.1	54.3	81.5
	38	59.7	89.6	49.1	73.7	37.8	56.8	67.4	101	58.0	87.2	48.8	73.2
	40	53.9	80.9	44.3	66.5	34.2	51.2	60.8	91.4	52.3	78.7	44.0	66.0
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	57.9	87.0	45.3	68.1	30.4	45.7	86.9	131	74.3	112	59.9	90.1
$P_e(L_c)^2/10^4$, kip-in.^{2}		2830		2330		1790		3000		2690		2310	
r_m, in.		3.15		3.18		3.21		2.58		2.63		2.69	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

5

COMPOSITE
HSS7–HSS6

Table IV-2B (continued)
Available Strength in Axial Compression, kips
Filled Square HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS7×7×								HSS6×6×			
		5/16		1/4		3/16		1/8		5/8		1/2	
t_{des}, in.		0.291		0.233		0.174		0.116		0.581		0.465	
Steel, lb/ft		27.6		22.4		17.1		11.6		42.3		35.2	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	277	416	245	367	211	316	176	264	350	526	298	447
	1	277	415	244	367	210	316	176	263	350	525	297	446
	2	275	413	243	365	209	314	175	262	347	522	295	443
	3	273	410	241	362	208	311	173	259	343	516	292	437
	4	270	405	239	358	205	308	171	256	338	508	287	431
	5	266	399	235	353	202	303	168	252	331	498	281	422
	6	262	392	231	346	198	297	165	247	323	486	274	411
	7	256	384	226	339	194	291	161	241	314	472	266	399
	8	250	375	221	331	189	283	156	234	304	456	257	386
	9	244	365	215	322	184	275	151	227	292	439	247	371
	10	236	354	208	312	178	267	146	219	280	421	237	355
	11	228	343	201	302	172	257	141	211	267	402	226	339
	12	220	330	194	291	165	247	135	202	254	382	215	323
	13	212	317	186	279	158	237	129	193	240	361	204	306
	14	203	304	178	267	151	226	122	183	226	340	193	289
	15	194	290	170	255	144	216	116	174	212	318	181	272
	16	184	276	161	242	136	204	109	164	198	297	170	255
	17	175	262	153	230	129	193	103	154	184	276	158	238
	18	165	248	145	217	121	182	96.4	145	170	255	147	221
	19	156	234	136	204	114	171	90.0	135	156	235	136	204
	20	146	220	128	192	107	160	83.8	126	143	215	125	188
	21	137	206	120	179	99.5	149	77.6	116	130	196	115	172
	22	128	192	111	167	92.5	139	71.7	108	119	179	104	157
	23	119	179	104	155	85.7	128	65.8	98.7	109	163	95.6	144
	24	111	166	95.9	144	78.9	118	60.4	90.7	99.8	150	87.8	132
	25	102	153	88.4	133	72.7	109	55.7	83.5	92.0	138	80.9	122
	26	94.3	141	81.7	123	67.2	101	51.5	77.2	85.1	128	74.8	112
	27	87.4	131	75.8	114	62.4	93.5	47.8	71.6	78.9	119	69.4	104
	28	81.3	122	70.5	106	58.0	87.0	44.4	66.6	73.4	110	64.5	96.9
	29	75.8	114	65.7	98.6	54.0	81.1	41.4	62.1	68.4	103	60.1	90.4
	30	70.8	106	61.4	92.1	50.5	75.8	38.7	58.0	63.9	96.0	56.2	84.4
	32	62.3	93.4	54.0	80.9	44.4	66.6	34.0	51.0	56.2	84.4	49.4	74.2
	34	55.1	82.7	47.8	71.7	39.3	59.0	30.1	45.2	49.7	74.8	43.7	65.7
	36	49.2	73.8	42.6	64.0	35.1	52.6	26.9	40.3	44.4	66.7	39.0	58.6
	38	44.1	66.2	38.3	57.4	31.5	47.2	24.1	36.2				
	40	39.8	59.8	34.5	51.8	28.4	42.6	21.8	32.6				
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	51.9	78.1	43.3	65.1	34.0	51.1	23.7	35.6	60.4	90.8	52.2	78.5
$P_e(L_c)^2/10^4$, kip-in.^{2}		2090		1810		1490		1140		1730		1570	
r_m, in.		2.72		2.75		2.77		2.80		2.17		2.23	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE

HSS6–HSS5½

Table IV-2B (continued)

Available Strength in Axial Compression, kips

Filled Square HSS

F_y = 50 ksi

f'_c = 5 ksi

Shape		HSS6×6×										HSS5½×5½×	
		⅜		5⁄16		¼		3⁄16		⅛		⅜	
t_{des}, in.		0.349		0.291		0.233		0.174		0.116		0.349	
Steel, lb/ft		27.5		23.3		19.0		14.5		9.86		24.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	249	374	223	334	196	294	167	251	138	207	221	331
	1	248	373	222	334	196	293	167	250	138	207	220	330
	2	247	370	221	332	194	291	166	249	137	205	218	328
	3	244	366	219	328	192	288	164	246	135	203	216	323
	4	240	361	215	323	189	284	161	242	133	199	212	317
	5	236	353	211	317	186	278	158	237	130	195	207	310
	6	230	345	206	309	181	272	154	231	127	190	201	301
	7	223	335	200	300	176	264	150	225	123	184	194	291
	8	216	324	194	291	170	256	145	217	118	177	186	279
	9	208	312	187	280	164	246	139	209	113	170	178	267
	10	200	299	179	269	158	236	134	200	108	162	169	254
	11	191	286	171	257	151	226	127	191	103	154	160	240
	12	181	272	163	244	143	215	121	181	97.3	146	151	226
	13	171	257	154	231	136	203	114	171	91.5	137	141	211
	14	161	242	145	218	128	192	108	161	85.7	129	131	197
	15	151	227	136	204	120	180	101	151	79.8	120	121	182
	16	141	212	127	191	112	168	93.9	141	74.0	111	112	168
	17	131	197	119	178	104	157	87.2	131	68.3	102	102	154
	18	122	182	110	165	96.8	145	80.6	121	62.7	94.1	93.6	141
	19	112	168	101	152	89.3	134	74.1	111	57.3	86.0	85.6	129
	20	103	154	93.0	139	82.0	123	67.9	102	52.0	78.0	77.7	117
	21	93.7	141	84.9	127	74.9	112	61.7	92.6	47.2	70.8	70.5	106
	22	85.3	128	77.3	116	68.3	102	56.3	84.4	43.0	64.5	64.2	96.5
	23	78.1	117	70.7	106	62.5	93.7	51.5	77.2	39.3	59.0	58.7	88.3
	24	71.7	108	65.0	97.5	57.4	86.0	47.3	70.9	36.1	54.2	53.9	81.1
	25	66.1	99.1	59.9	89.8	52.9	79.3	43.6	65.3	33.3	49.9	49.7	74.7
	26	61.1	91.7	55.4	83.0	48.9	73.3	40.3	60.4	30.8	46.2	46.0	69.1
	27	56.7	85.0	51.3	77.0	45.3	68.0	37.3	56.0	28.5	42.8	42.6	64.1
	28	52.7	79.0	47.7	71.6	42.1	63.2	34.7	52.1	26.5	39.8	39.6	59.6
	29	49.1	73.7	44.5	66.8	39.3	58.9	32.4	48.6	24.7	37.1	36.9	55.5
	30	45.9	68.8	41.6	62.4	36.7	55.1	30.3	45.4	23.1	34.7	34.5	51.9
	32	40.3	60.5	36.5	54.8	32.3	48.4	26.6	39.9	20.3	30.5	30.3	45.6
	34	35.7	53.6	32.4	48.6	28.6	42.9	23.6	35.3	18.0	27.0	26.9	40.4
	36	31.9	47.8	28.9	43.3	25.5	38.2	21.0	31.5	16.1	24.1		
	38	28.6	42.9	25.9	38.9	22.9	34.3	18.9	28.3	14.4	21.6		

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	42.4	63.7	37.0	55.5	30.9	46.5	24.4	36.6	17.2	25.9	35.0	52.5
$P_e(L_c)^2/10^4$, kip-in.^{2}			1360		1230		1090		894		683		1000	
r_m, in.			2.28		2.31		2.34		2.37		2.39		2.08	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.

Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS5½–HSS5

Table IV-2B (continued)
Available Strength in Axial Compression, kips
Filled Square HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS5½×5½×								HSS5×5×			
		5⁄16		¼		3⁄16		⅛		½		⅜	
t_{des}, in.		0.291		0.233		0.174		0.116		0.465		0.349	
Steel, lb/ft		21.2		17.3		13.3		9.01		28.4		22.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	197	296	173	260	147	221	120	181	236	355	194	290
	1	197	295	173	259	147	220	120	180	235	353	193	289
	2	195	293	171	257	146	218	119	179	233	350	191	287
	3	193	289	169	254	144	215	117	176	229	345	188	282
	4	189	284	166	249	141	211	115	173	224	337	184	276
	5	185	277	162	243	138	207	112	168	218	328	179	268
	6	180	270	158	236	134	201	109	163	210	316	172	258
	7	174	261	152	229	129	194	105	157	202	303	165	248
	8	167	251	147	220	124	186	100	151	192	289	157	236
	9	160	240	140	210	119	178	95.6	143	182	274	149	223
	10	152	228	134	200	113	169	90.6	136	172	258	140	210
	11	144	216	126	190	107	160	85.3	128	161	241	131	196
	12	135	203	119	179	100	151	79.9	120	149	224	121	182
	13	127	190	112	168	94.0	141	74.4	112	138	207	112	168
	14	118	177	104	156	87.5	131	68.9	103	127	190	103	154
	15	110	164	96.6	145	81.1	122	63.4	95.1	115	173	94.0	141
	16	101	152	89.2	134	74.7	112	58.1	87.1	105	157	85.6	129
	17	92.6	139	81.9	123	68.4	103	52.8	79.3	94.2	142	77.5	116
	18	84.5	127	74.8	112	62.4	93.6	47.8	71.7	84.1	126	69.6	105
	19	76.7	115	68.0	102	56.5	84.7	43.0	64.4	75.5	113	62.5	93.9
	20	69.2	104	61.3	92.0	51.0	76.5	38.8	58.2	68.1	102	56.4	84.8
	21	62.8	94.1	55.6	83.5	46.2	69.4	35.2	52.7	61.8	92.9	51.2	76.9
	22	57.2	85.8	50.7	76.0	42.1	63.2	32.0	48.1	56.3	84.6	46.6	70.0
	23	52.3	78.5	46.4	69.6	38.5	57.8	29.3	44.0	51.5	77.4	42.6	64.1
	24	48.1	72.1	42.6	63.9	35.4	53.1	26.9	40.4	47.3	71.1	39.2	58.9
	25	44.3	66.4	39.3	58.9	32.6	48.9	24.8	37.2	43.6	65.5	36.1	54.2
	26	40.9	61.4	36.3	54.4	30.2	45.2	22.9	34.4	40.3	60.6	33.4	50.2
	27	38.0	57.0	33.7	50.5	28.0	42.0	21.3	31.9	37.4	56.2	30.9	46.5
	28	35.3	53.0	31.3	46.9	26.0	39.0	19.8	29.7	34.8	52.2	28.8	43.2
	29	32.9	49.4	29.2	43.8	24.2	36.4	18.4	27.7	32.4	48.7	26.8	40.3
	30	30.8	46.1	27.3	40.9	22.7	34.0	17.2	25.8	30.3	45.5	25.1	37.7
	32	27.0	40.5	24.0	35.9	19.9	29.9	15.1	22.7				
	34	23.9	35.9	21.2	31.8	17.6	26.5	13.4	20.1				
	36					15.7	23.6	12.0	17.9				
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	30.5	45.9	25.6	38.4	20.2	30.3	14.3	21.5	34.2	51.4	28.1	42.3
$P_e(L_c)^2/10^4$, kip-in.^{2}		909		806		670		509		821		719	
r_m, in.		2.11		2.13		2.16		2.19		1.82		1.87	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

5

COMPOSITE

HSS5–HSS4½

Table IV-2B (continued)

Available Strength in Axial Compression, kips

Filled Square HSS

F_y = 50 ksi

f'_c = 5 ksi

Shape		HSS5×5×								HSS4½×4½×			
		5⁄16		¼		3⁄16		⅛		½		⅜	
t_{des}, in.		0.291		0.233		0.174		0.116		0.465		0.349	
Steel, lb/ft		19.1		15.6		12.0		8.16		25.0		19.8	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	173	259	151	227	128	192	104	156	208	313	167	251
	1	172	258	151	226	128	191	104	156	207	311	167	250
	2	171	256	149	224	126	189	103	154	205	308	165	247
	3	168	252	147	220	124	187	101	151	201	302	161	242
	4	164	246	144	216	122	182	98.6	148	195	293	157	235
	5	160	239	140	209	118	177	95.6	143	188	283	151	227
	6	154	231	135	202	114	171	92.1	138	180	270	145	217
	7	148	222	130	194	110	164	88.1	132	171	256	137	206
	8	141	211	124	185	104	157	83.8	126	160	241	129	194
	9	133	200	117	176	99.0	149	79.1	119	150	225	121	182
	10	126	188	110	165	93.2	140	74.2	111	139	208	112	169
	11	117	176	103	155	87.2	131	69.1	104	127	191	104	156
	12	109	164	96.1	144	81.1	122	63.9	95.9	116	174	95.3	143
	13	101	151	88.8	133	74.9	112	58.7	88.1	105	157	86.7	130
	14	92.5	139	81.6	122	68.8	103	53.6	80.4	93.9	141	78.3	118
	15	84.3	126	74.5	112	62.8	94.2	48.6	72.9	83.4	125	70.2	105
	16	76.3	115	67.6	101	56.9	85.4	43.8	65.7	73.5	110	62.3	93.7
	17	68.7	103	60.9	91.4	51.3	76.9	39.1	58.6	65.1	97.8	55.2	83.0
	18	61.4	92.0	54.4	81.7	45.8	68.7	34.9	52.3	58.0	87.2	49.2	74.0
	19	55.1	82.6	48.9	73.3	41.1	61.7	31.3	46.9	52.1	78.3	44.2	66.4
	20	49.7	74.5	44.1	66.2	37.1	55.6	28.2	42.4	47.0	70.7	39.9	59.9
	21	45.1	67.6	40.0	60.0	33.6	50.5	25.6	38.4	42.6	64.1	36.2	54.4
	22	41.1	61.6	36.4	54.7	30.7	46.0	23.3	35.0	38.9	58.4	33.0	49.5
	23	37.6	56.4	33.3	50.0	28.1	42.1	21.3	32.0	35.5	53.4	30.2	45.3
	24	34.5	51.8	30.6	45.9	25.8	38.6	19.6	29.4	32.6	49.1	27.7	41.6
	25	31.8	47.7	28.2	42.3	23.7	35.6	18.1	27.1	30.1	45.2	25.5	38.4
	26	29.4	44.1	26.1	39.1	22.0	32.9	16.7	25.1	27.8	41.8	23.6	35.5
	27	27.3	40.9	24.2	36.3	20.4	30.5	15.5	23.2			21.9	32.9
	28	25.4	38.0	22.5	33.8	18.9	28.4	14.4	21.6				
	29	23.6	35.5	21.0	31.5	17.6	26.5	13.4	20.1				
	30	22.1	33.1	19.6	29.4	16.5	24.7	12.5	18.8				
	32			17.2	25.8	14.5	21.7	11.0	16.5				
Properties													
M_n/Ω_b $\phi_b M_n$	kip-ft	24.6	37.0	20.7	31.2	16.4	24.6	11.7	17.6	26.5	39.8	22.0	33.1
$P_e(L_c)^2/10^4$, kip-in.^{2}		653		579		487		371		563		497	
r_m, in.		1.90		1.93		1.96		1.99		1.61		1.67	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.

Dashed line indicates the L_c beyond which the bare steel strength controls.

5

COMPOSITE
HSS4½–HSS4

Table IV-2B (continued)
Available Strength in Axial Compression, kips
Filled Square HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS4½×4½×								HSS4×4×			
		5⁄16		¼		3⁄16		⅛		½		⅜	
t_{des}, in.		0.291		0.233		0.174		0.116		0.465		0.349	
Steel, lb/ft		17.0		13.9		10.7		7.31		21.6		17.3	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	149	224	130	196	110	165	88.7	133	180	271	143	215
	1	149	223	130	195	109	164	88.3	132	179	269	142	214
	2	147	221	128	193	108	162	87.2	131	176	265	140	211
	3	144	216	126	189	106	159	85.5	128	172	258	137	206
	4	140	210	123	184	103	155	83.1	125	166	249	132	199
	5	135	203	118	177	99.7	150	80.1	120	158	237	127	190
	6	130	194	113	170	95.6	143	76.5	115	149	224	120	180
	7	123	185	108	162	90.9	136	72.6	109	139	209	113	169
	8	116	174	102	152	85.8	129	68.2	102	128	193	105	157
	9	108	163	95.1	143	80.3	120	63.7	95.5	117	176	96.4	145
	10	100	151	88.2	132	74.6	112	58.9	88.3	106	160	87.9	132
	11	92.4	139	81.3	122	68.8	103	54.0	81.1	95.0	143	79.4	119
	12	84.4	127	74.3	111	62.9	94.4	49.2	73.8	84.1	126	71.0	107
	13	76.4	115	67.4	101	57.1	85.7	44.4	66.6	73.6	111	62.8	94.4
	14	68.6	103	60.6	90.9	51.5	77.2	39.8	59.6	63.7	95.8	55.0	82.7
	15	61.7	92.8	54.1	81.2	46.0	69.0	35.3	53.0	55.5	83.5	47.9	72.0
	16	55.1	82.9	47.8	71.8	40.7	61.1	31.1	46.6	48.8	73.3	42.1	63.3
	17	48.8	73.4	42.4	63.6	36.1	54.1	27.5	41.3	43.2	65.0	37.3	56.1
	18	43.6	65.5	37.8	56.7	32.2	48.3	24.5	36.8	38.6	58.0	33.3	50.0
	19	39.1	58.8	33.9	50.9	28.9	43.3	22.0	33.0	34.6	52.0	29.9	44.9
	20	35.3	53.0	30.6	45.9	26.1	39.1	19.9	29.8	31.2	46.9	27.0	40.5
	21	32.0	48.1	27.8	41.7	23.6	35.5	18.0	27.1	28.3	42.6	24.4	36.7
	22	29.2	43.8	25.3	38.0	21.5	32.3	16.4	24.6	25.8	38.8	22.3	33.5
	23	26.7	40.1	23.2	34.7	19.7	29.6	15.0	22.6	23.6	35.5	20.4	30.6
	24	24.5	36.8	21.3	31.9	18.1	27.2	13.8	20.7			18.7	28.1
	25	22.6	34.0	19.6	29.4	16.7	25.0	12.7	19.1				
	26	20.9	31.4	18.1	27.2	15.4	23.1	11.8	17.6				
	27	19.4	29.1	16.8	25.2	14.3	21.5	10.9	16.4				
	28	18.0	27.1	15.6	23.4	13.3	19.9	10.1	15.2				
	29					12.4	18.6	9.46	14.2				

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	19.4	29.1	16.4	24.6	13.0	19.6	9.32	14.0	19.9	29.9	16.7	25.2
$P_e(L_c)^2/10^4$, kip-in.^{2}			454		402		342		261		365		328	
r_m, in.			1.70		1.73		1.75		1.78		1.41		1.47	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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Table IV-2B (continued)
Available Strength in Axial Compression, kips
Filled Square HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS4×4×								HSS3½×3½×			
		5⁄16		¼		3⁄16		⅛		⅜		5⁄16	
t_{des}, in,		0.291		0.233		0.174		0.116		0.349		0.291	
Steel, lb/ft		14.8		12.2		9.42		6.46		14.7		12.7	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	127	191	111	166	92.8	139	74.4	112	122	184	106	159
	1	127	190	110	165	92.3	139	74.0	111	122	183	105	158
	2	125	187	109	163	91.0	136	72.9	109	119	179	103	155
	3	121	182	106	159	88.8	133	71.1	107	115	173	99.6	150
	4	117	176	102	153	85.8	129	68.6	103	110	166	95.2	143
	5	112	168	97.7	147	82.1	123	65.5	98.3	104	156	90.0	135
	6	106	159	92.5	139	77.8	117	62.0	92.9	96.4	145	83.9	126
	7	99.0	149	86.7	130	73.0	110	58.0	87.0	88.5	133	77.3	116
	8	91.7	138	80.5	121	67.9	102	53.7	80.6	80.1	120	70.3	106
	9	84.1	126	73.9	111	62.5	93.7	49.3	73.9	71.6	108	63.1	94.9
	10	76.4	115	67.3	101	56.9	85.4	44.8	67.1	63.1	94.8	56.0	84.1
	11	69.2	104	60.6	90.9	51.4	77.0	40.2	60.3	54.9	82.5	49.0	73.7
	12	62.0	93.2	54.0	81.0	45.9	68.9	35.8	53.7	47.1	70.7	42.4	63.7
	13	55.1	82.8	47.7	71.5	40.6	60.9	31.5	47.3	40.1	60.3	36.2	54.4
	14	48.5	72.8	41.6	62.4	35.6	53.3	27.4	41.1	34.6	52.0	31.2	46.9
	15	42.2	63.5	36.3	54.4	31.0	46.5	23.9	35.8	30.1	45.3	27.2	40.8
	16	37.1	55.8	31.9	47.8	27.2	40.8	21.0	31.5	26.5	39.8	23.9	35.9
	17	32.9	49.4	28.2	42.3	24.1	36.2	18.6	27.9	23.5	35.2	21.2	31.8
	18	29.3	44.1	25.2	37.8	21.5	32.3	16.6	24.9	20.9	31.4	18.9	28.4
	19	26.3	39.6	22.6	33.9	19.3	29.0	14.9	22.3	18.8	28.2	16.9	25.5
	20	23.8	35.7	20.4	30.6	17.4	26.1	13.4	20.2	16.9	25.5	15.3	23.0
	21	21.5	32.4	18.5	27.7	15.8	23.7	12.2	18.3	15.4	23.1	13.9	20.8
	22	19.6	29.5	16.9	25.3	14.4	21.6	11.1	16.7				
	23	18.0	27.0	15.4	23.1	13.2	19.8	10.2	15.2				
	24	16.5	24.8	14.2	21.2	12.1	18.1	9.33	14.0				
	25			13.1	19.6	11.2	16.7	8.60	12.9				
	26							7.95	11.9				
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	14.8	22.2	12.6	18.9	10.1	15.1	7.23	10.9	12.2	18.3	10.9	16.3
$P_e(L_c)^2/10^4$, kip-in.^{2}		300		268		229		176		203		188	
r_m, in.		1.49		1.52		1.55		1.58		1.26		1.29	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

5

COMPOSITE
HSS3½–HSS3

Table IV-2B (continued)

Available Strength in Axial Compression, kips

Filled Square HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape		HSS3½×3½×						HSS3×3×					
		¼		3⁄16		⅛		⅜		5⁄16		¼	
t_{des}, in.		0.233		0.174		0.116		0.349		0.291		0.233	
Steel, lb/ft		10.5		8.15		5.61		12.2		10.6		8.81	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	92.2	138	77.1	116	61.2	91.8	101	153	88.0	132	74.5	112
	1	91.6	137	76.6	115	60.8	91.2	101	151	87.2	131	73.9	111
	2	89.8	135	75.1	113	59.6	89.4	97.8	147	84.9	128	71.9	108
	3	86.9	130	72.7	109	57.7	86.5	93.3	140	81.2	122	68.6	103
	4	83.0	124	69.5	104	55.1	82.7	87.4	131	76.2	115	64.4	96.6
	5	78.2	117	65.6	98.4	52.0	78.0	80.3	121	70.2	106	59.3	88.9
	6	72.7	109	61.1	91.7	48.4	72.6	72.4	109	63.6	95.6	53.7	80.7
	7	66.7	100	56.2	84.3	44.5	66.7	64.1	96.4	56.6	85.0	48.1	72.2
	8	60.4	90.7	51.0	76.5	40.3	60.5	55.7	83.7	49.4	74.2	42.3	63.5
	9	54.0	81.0	45.7	68.6	36.1	54.1	47.5	71.4	42.4	63.7	36.6	55.0
	10	47.7	71.6	40.5	60.7	31.9	47.8	39.8	59.8	35.7	53.6	31.1	46.7
	11	41.9	63.0	35.3	53.0	27.8	41.7	32.9	49.4	29.6	44.5	25.9	39.0
	12	36.5	54.9	30.5	45.7	24.0	35.9	27.6	41.5	24.9	37.4	21.8	32.8
	13	31.3	47.1	26.0	39.0	20.4	30.6	23.5	35.4	21.2	31.8	18.6	27.9
	14	27.0	40.6	22.4	33.6	17.6	26.4	20.3	30.5	18.3	27.4	16.0	24.1
	15	23.5	35.4	19.5	29.3	15.3	23.0	17.7	26.6	15.9	23.9	13.9	21.0
	16	20.7	31.1	17.2	25.7	13.5	20.2	15.5	23.3	14.0	21.0	12.3	18.4
	17	18.3	27.5	15.2	22.8	11.9	17.9	13.8	20.7	12.4	18.6	10.9	16.3
	18	16.3	24.6	13.6	20.3	10.6	16.0			11.0	16.6	9.69	14.6
	19	14.7	22.0	12.2	18.2	9.55	14.3						
	20	13.2	19.9	11.0	16.5	8.62	12.9						
	21	12.0	18.0	9.96	14.9	7.82	11.7						
	22	10.9	16.4	9.07	13.6	7.13	10.7						
Properties													
M_n/Ω_b / $\phi_b M_n$	kip-ft	9.31	14.0	7.49	11.3	5.39	8.11	8.39	12.6	7.55	11.4	6.54	9.83
$P_e(L_c)^2/10^4$, kip-in.^{2}		168		144		113		116		108		98.1	
r_m, in.		1.32		1.35		1.37		1.06		1.08		1.11	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE

HSS3–HSS2½

Table IV-2B (continued)

Available Strength in Axial Compression, kips

Filled Square HSS

F_y = 50 ksi

f'_c = 5 ksi

Shape		HSS3×3×				HSS2½×2½×							
		3⁄16		1⁄8		5⁄16		1⁄4		3⁄16		1⁄8	
t_{des}, in,		0.174		0.116		0.291		0.233		0.174		0.116	
Steel, lb/ft		6.87		4.75		8.45		7.11		5.59		3.90	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	62.1	93.2	48.8	73.1	70.4	106	59.0	88.6	48.3	72.4	37.7	56.5
	1	61.6	92.4	48.3	72.5	69.4	104	58.2	87.5	47.7	71.5	37.2	55.8
	2	60.0	90.0	47.1	70.6	66.6	100	56.0	84.2	45.8	68.8	35.8	53.7
	3	57.4	86.1	45.1	67.6	62.3	93.6	52.6	79.0	43.0	64.4	33.6	50.5
	4	53.9	80.9	42.4	63.6	56.6	85.1	48.1	72.3	39.2	58.8	30.8	46.2
	5	49.8	74.7	39.2	58.8	50.1	75.3	42.9	64.4	34.9	52.3	27.5	41.3
	6	45.2	67.8	35.6	53.5	43.1	64.8	37.2	56.0	30.2	45.3	24.0	35.9
	7	40.3	60.4	31.8	47.8	36.1	54.3	31.5	47.4	25.6	38.5	20.4	30.5
	8	35.2	52.9	27.9	41.9	29.5	44.3	26.0	39.1	21.4	32.2	16.9	25.3
	9	30.3	45.5	24.1	36.2	23.5	35.2	20.9	31.5	17.4	26.2	13.6	20.4
	10	25.6	38.4	20.4	30.6	19.0	28.6	17.0	25.5	14.1	21.2	11.0	16.5
	11	21.3	31.9	17.0	25.5	15.7	23.6	14.0	21.1	11.7	17.5	9.10	13.7
	12	17.9	26.8	14.3	21.4	13.2	19.8	11.8	17.7	9.80	14.7	7.65	11.5
	13	15.2	22.9	12.2	18.3	11.2	16.9	10.0	15.1	8.35	12.6	6.52	9.77
	14	13.1	19.7	10.5	15.7	9.69	14.6	8.65	13.0	7.20	10.8	5.62	8.43
	15	11.4	17.2	9.14	13.7			7.53	11.3	6.27	9.43	4.89	7.34
	16	10.1	15.1	8.04	12.1							4.30	6.45
	17	8.91	13.4	7.12	10.7								
	18	7.95	11.9	6.35	9.53								
	19	7.13	10.7	5.70	8.55								
Properties													
M_n/Ω_b / $\phi_b M_n$	kip-ft	5.29	7.95	3.87	5.81	4.86	7.30	4.26	6.40	3.50	5.27	2.58	3.88
$P_e(L_c)^2/10^4$, kip-in.^{2}		84.6		67.6		55.8		51.4		44.7		36.2	
r_m, in.		1.14		1.17		0.880		0.908		0.937		0.965	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS2¼–HSS2

Table IV-2B (continued)

Available Strength in Axial Compression, kips

Filled Square HSS

F_y = 50 ksi
f'_c = 5 ksi

Shape	HSS2¼×2¼×						HSS2×2×					
	¼		3⁄16		⅛		¼		3⁄16		⅛	
t_{des}, in.	0.233		0.174		0.116		0.233		0.174		0.116	
Steel, lb/ft	6.26		4.96		3.48		5.41		4.32		3.05	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r												
0	52.1	78.3	41.9	62.8	32.5	48.8	45.2	67.9	35.6	53.5	27.6	41.4
1	51.3	77.0	41.2	61.8	32.0	48.0	44.3	66.5	34.9	52.5	27.1	40.6
2	48.8	73.4	39.2	58.9	30.6	45.8	41.5	62.4	32.9	49.5	25.5	38.3
3	45.0	67.7	36.2	54.3	28.3	42.4	37.3	56.1	29.9	44.9	23.1	34.6
4	40.2	60.4	32.3	48.4	25.3	38.0	32.2	48.4	26.0	39.1	20.1	30.1
5	34.7	52.2	28.1	42.3	22.0	33.0	26.6	40.0	21.8	32.8	16.8	25.2
6	29.1	43.7	23.8	35.8	18.6	27.8	21.0	31.6	17.6	26.4	13.5	20.2
7	23.5	35.4	19.6	29.4	15.1	22.7	15.9	24.0	13.6	20.5	10.4	15.6
8	18.4	27.7	15.6	23.4	12.0	17.9	12.2	18.3	10.4	15.7	7.95	11.9
9	14.6	21.9	12.3	18.5	9.45	14.2	9.64	14.5	8.24	12.4	6.28	9.42
10	11.8	17.7	9.97	15.0	7.65	11.5	7.81	11.7	6.67	10.0	5.09	7.63
11	9.75	14.7	8.24	12.4	6.33	9.49	6.46	9.70	5.52	8.29	4.20	6.31
12	8.19	12.3	6.92	10.4	5.31	7.97			4.63	6.97	3.53	5.30
13	6.98	10.5	5.90	8.87	4.53	6.79						
14					3.90	5.86						
Properties												
M_n/Ω_b / $\phi_b M_n$ kip-ft	3.32	5.00	2.74	4.12	2.04	3.07	2.49	3.74	2.09	3.14	1.57	2.36
$P_e(L_c)^2/10^4$, kip-in.^{2}	35.2		31.0		25.1		22.9		20.4		16.7	
r_m, in.	0.806		0.835		0.863		0.704		0.733		0.761	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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④
COMPOSITE
HSS20.000–
HSS16.000

Table IV-3A
Available Strength in Axial Compression, kips
Filled Round HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		**HSS20.000×**				**HSS18.000×**				**HSS16.000×**			
		0.500		**0.375**		**0.500**		**0.375**		**0.625**		**0.500**	
t_{des}, in,		**0.465**		**0.349**		**0.465**		**0.349**		**0.581**		**0.465**	
Steel, lb/ft		**104**		**78.7**		**93.5**		**70.7**		**103**		**82.9**	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length, L_c (ft), with respect to the least radius of gyration, r	**0**	1200	1800	1050	1580	1020	1540	893	1340	975	1460	861	1290
	1	1200	1800	1050	1580	1020	1530	893	1340	975	1460	861	1290
	2	1200	1800	1050	1570	1020	1530	892	1340	973	1460	860	1290
	3	1200	1790	1050	1570	1020	1530	890	1340	972	1460	858	1290
	4	1190	1790	1050	1570	1020	1530	888	1330	969	1450	856	1280
	5	1190	1790	1040	1560	1020	1520	885	1330	966	1450	853	1280
	6	1190	1780	1040	1560	1010	1520	882	1320	962	1440	849	1270
	7	1180	1770	1040	1550	1010	1510	878	1320	957	1440	845	1270
	8	1180	1770	1030	1550	1000	1500	874	1310	952	1430	840	1260
	9	1170	1760	1030	1540	998	1500	869	1300	946	1420	834	1250
	10	1170	1750	1020	1530	992	1490	863	1290	939	1410	828	1240
	11	1160	1740	1020	1520	985	1480	857	1290	931	1400	821	1230
	12	1150	1730	1010	1510	978	1470	850	1280	923	1380	814	1220
	13	1150	1720	1000	1500	970	1460	843	1260	915	1370	806	1210
	14	1140	1710	994	1490	962	1440	836	1250	905	1360	797	1200
	15	1130	1690	986	1480	953	1430	828	1240	895	1340	788	1180
	16	1120	1680	978	1470	944	1420	819	1230	885	1330	779	1170
	17	1110	1670	969	1450	934	1400	810	1210	874	1310	769	1150
	18	1100	1650	959	1440	924	1390	800	1200	863	1290	758	1140
	19	1090	1640	949	1420	913	1370	790	1190	851	1280	747	1120
	20	1080	1620	939	1410	902	1350	780	1170	838	1260	736	1100
	21	1070	1600	928	1390	890	1340	769	1150	825	1240	724	1090
	22	1060	1580	917	1380	878	1320	758	1140	812	1220	712	1070
	23	1040	1560	906	1360	866	1300	747	1120	798	1200	700	1050
	24	1030	1550	894	1340	853	1280	735	1100	784	1180	687	1030
	25	1020	1530	882	1320	840	1260	723	1080	770	1150	674	1010
	26	1000	1510	869	1300	827	1240	711	1070	755	1130	661	991
	27	990	1480	856	1280	813	1220	698	1050	740	1110	647	971
	28	976	1460	843	1260	799	1200	685	1030	725	1090	633	950
	29	961	1440	830	1240	784	1180	672	1010	710	1060	619	929
	30	946	1420	816	1220	770	1150	659	988	694	1040	605	908
	32	916	1370	788	1180	740	1110	632	948	662	993	576	865
	34	885	1330	760	1140	710	1070	604	906	630	945	547	821
	36	853	1280	730	1100	679	1020	576	865	598	896	518	777
	38	821	1230	701	1050	648	972	548	822	565	848	489	733
	40	788	1180	671	1010	617	925	520	780	533	799	460	690
Properties													
M_n/Ω_b **$\phi_b M_n$**	**kip-ft**	476	716	373	560	380	571	298	447	355	533	294	443
$P_e(L_c)^2/10^4$, kip-in.^{2}		55100		45200		39000		31900		31100		26500	
r_m, in.		6.91		6.95		6.20		6.24		5.46		5.49	

ASD	**LRFD**
Ω_b = 1.67	**ϕ_b = 0.90**
Ω_c = 2.00	**ϕ_c = 0.75**

④

COMPOSITE
HSS16.000–
HSS14.000

Table IV-3A (continued)

Available Strength in Axial Compression, kips

Filled Round HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS16.000×								HSS14.000×			
		0.438		0.375		0.312		0.250		0.625		0.500	
t_{des}, in.		0.407		0.349		0.291		0.233		0.581		0.465	
Steel, lb/ft		72.9		62.6		52.3		42.1		89.4		72.2	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	802	1200	745	1120	686	1030	625	937	809	1210	710	1070
	1	802	1200	745	1120	686	1030	624	937	809	1210	710	1060
	2	801	1200	744	1120	685	1030	624	935	808	1210	709	1060
	3	799	1200	742	1110	683	1020	622	933	806	1210	707	1060
	4	797	1200	740	1110	681	1020	620	930	803	1200	705	1060
	5	794	1190	737	1110	678	1020	618	926	800	1200	701	1050
	6	790	1190	734	1100	675	1010	614	922	795	1190	698	1050
	7	786	1180	730	1090	671	1010	611	916	790	1190	693	1040
	8	782	1170	725	1090	667	1000	607	910	784	1180	688	1030
	9	776	1160	720	1080	662	993	602	903	778	1170	682	1020
	10	770	1160	715	1070	657	985	597	895	771	1160	676	1010
	11	764	1150	708	1060	651	976	591	886	763	1140	668	1000
	12	757	1140	702	1050	644	966	585	877	754	1130	661	991
	13	749	1120	694	1040	637	956	578	867	745	1120	653	979
	14	741	1110	687	1030	630	945	571	856	735	1100	644	966
	15	733	1100	678	1020	622	933	563	845	725	1090	635	952
	16	724	1090	670	1000	614	921	555	833	714	1070	625	937
	17	714	1070	661	991	605	907	547	821	703	1050	615	922
	18	704	1060	651	977	596	894	538	807	691	1040	604	906
	19	694	1040	641	962	586	879	529	794	678	1020	593	889
	20	683	1020	631	946	576	865	520	780	666	998	581	872
	21	672	1010	620	930	566	849	510	765	652	978	569	854
	22	660	990	609	914	556	834	500	750	639	958	557	836
	23	648	973	598	897	545	818	490	735	625	937	545	817
	24	636	954	586	880	534	801	479	719	611	916	532	798
	25	624	936	575	862	523	784	469	703	596	894	519	779
	26	611	917	563	844	511	767	458	687	581	872	506	759
	27	598	898	550	825	500	750	447	670	567	850	493	739
	28	585	878	538	807	488	732	436	654	551	827	480	719
	29	572	858	525	788	476	714	425	637	536	804	466	699
	30	559	838	513	769	464	696	413	620	521	782	453	679
	32	531	797	487	730	440	659	390	586	490	736	425	638
	34	504	756	461	691	415	623	367	551	460	690	398	597
	36	476	715	435	652	391	586	345	517	429	644	371	557
	38	449	673	409	613	366	549	322	483	399	599	345	517
	40	422	633	383	575	342	513	300	449	370	555	319	478
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	263	396	231	347	198	297	161	242	266	399	221	332
$P_e(L_c)^2/10^4$, kip-in.^{2}		24100		21600		19000		16400		19900		17100	
r_m, in.		5.51		5.53		5.55		5.58		4.75		4.79	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

4

COMPOSITE
HSS14.000–
HSS12.750

Table IV-3A (continued)
Available Strength in Axial Compression, kips
Filled Round HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		**HSS14.000×**						**HSS12.750×**					
		0.375		**0.312**		**0.250**		**0.500**		**0.375**		**0.250**	
t_{des}, in.		**0.349**		**0.291**		**0.233**		**0.465**		**0.349**		**0.233**	
Steel, lb/ft		**54.6**		**45.7**		**36.8**		**65.5**		**49.6**		**33.4**	
Design		**P_n/Ω_c**	**$\phi_c P_n$**	**P_n/Ω_c**	**$\phi_c P_n$**	**P_n/Ω_c**	**$\phi_c P_n$**	**P_n/Ω_c**	**$\phi_c P_n$**	**P_n/Ω_c**	**$\phi_c P_n$**	**P_n/Ω_c**	**$\phi_c P_n$**
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length, L_c (ft), with respect to the least radius of gyration, r	**0**	609	913	556	834	506	758	620	930	530	794	436	654
	1	609	913	556	834	505	758	620	930	529	794	436	653
	2	608	912	555	833	504	757	619	928	528	792	435	652
	3	606	909	553	830	503	754	617	925	527	790	433	650
	4	604	906	551	827	501	751	614	922	524	786	431	647
	5	601	902	549	823	498	747	611	917	521	782	429	643
	6	598	896	545	818	495	743	607	911	518	777	425	638
	7	593	890	542	812	491	737	602	904	514	770	422	632
	8	589	883	537	806	487	731	597	896	509	763	417	626
	9	584	875	532	798	482	724	591	887	503	755	412	619
	10	578	867	527	790	477	716	584	877	498	746	407	611
	11	571	857	521	781	471	707	577	866	491	737	401	602
	12	564	847	514	771	465	698	569	854	484	726	395	593
	13	557	836	507	761	458	688	561	841	477	715	388	583
	14	549	824	500	749	451	677	552	828	469	703	381	572
	15	541	811	492	738	444	666	543	814	460	690	374	561
	16	532	798	483	725	436	654	533	799	451	677	366	549
	17	523	784	475	712	428	641	522	784	442	663	358	537
	18	513	770	466	699	419	629	512	767	433	649	350	524
	19	503	755	456	685	410	615	500	751	423	634	341	511
	20	493	740	447	670	401	601	489	734	413	619	332	498
	21	483	724	437	655	392	587	477	716	402	603	323	484
	22	472	708	427	640	382	573	465	698	392	588	313	470
	23	461	691	416	625	372	558	453	679	381	571	304	456
	24	449	674	406	609	362	543	440	661	370	555	294	442
	25	438	657	395	593	352	528	428	642	359	538	285	427
	26	426	640	384	576	342	512	415	623	348	521	275	413
	27	415	622	373	560	331	497	402	603	336	504	265	398
	28	403	604	362	543	321	481	389	584	325	487	256	383
	29	391	586	351	526	310	466	376	564	314	470	246	369
	30	379	568	340	510	300	450	363	545	302	453	236	354
	32	355	532	318	476	279	419	337	506	280	420	217	326
	34	331	497	295	443	259	388	312	468	258	387	198	298
	36	307	461	274	410	238	358	287	431	236	354	180	270
	38	284	427	252	378	219	328	263	394	215	323	163	244
	40	262	393	232	347	200	300	239	359	195	292	147	220
Properties													
M_n/Ω_b / **$\phi_b M_n$**	**kip-ft**	173	261	149	223	122	184	180	271	142	213	101	151
$P_e(L_c)^2/10^4$, kip-in.^{2}		13900		12200		10500		12600		10200		7710	
r_m, in.		4.83		4.85		4.87		4.35		4.39		4.43	

ASD	**LRFD**
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

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4

COMPOSITE
HSS10.750–
HSS10.000

Table IV-3A (continued)

Available Strength in Axial Compression, kips

Filled Round HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS10.750×						HSS10.000×					
		0.500		0.375		0.250		0.625		0.500		0.375	
t_{des}, in.		0.465		0.349		0.233		0.581		0.465		0.349	
Steel, lb/ft		54.8		41.6		28.1		62.6		50.8		38.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	489	733	413	619	335	502	512	768	443	664	373	559
	1	489	733	413	619	335	502	512	767	442	663	373	559
	2	487	731	412	617	334	501	510	765	441	661	371	557
	3	485	728	410	615	332	498	508	762	439	658	370	554
	4	483	724	407	611	330	495	504	756	436	654	367	551
	5	479	718	404	606	327	491	500	750	432	648	364	546
	6	475	712	400	601	324	486	495	742	428	641	360	540
	7	469	704	396	594	320	480	488	733	422	633	355	533
	8	464	696	391	587	316	474	481	722	416	624	350	525
	9	457	686	385	578	311	466	474	710	410	614	345	517
	10	450	675	379	569	305	458	465	698	402	603	338	507
	11	442	663	372	559	299	449	456	684	394	591	331	497
	12	434	651	365	548	293	440	446	669	386	578	324	486
	13	425	638	358	536	286	430	435	653	377	565	316	474
	14	416	623	349	524	279	419	424	636	367	550	308	462
	15	406	609	341	511	272	408	412	618	357	535	299	449
	16	395	593	332	498	264	397	400	600	346	520	290	436
	17	385	577	323	484	256	385	387	581	336	504	281	422
	18	374	561	313	470	248	372	375	562	325	487	272	408
	19	363	544	304	455	240	360	361	542	313	470	262	393
	20	351	526	294	440	231	347	348	522	302	453	252	378
	21	339	509	284	425	223	334	335	502	290	435	242	364
	22	327	491	273	410	214	321	321	481	279	418	232	349
	23	315	473	263	395	205	308	307	461	267	400	222	334
	24	303	455	253	379	197	295	294	441	255	383	212	319
	25	291	437	242	363	188	282	280	420	244	365	203	304
	26	279	419	232	348	179	269	267	400	232	348	193	289
	27	267	401	222	333	171	256	253	380	220	331	183	274
	28	255	383	212	317	162	243	240	360	209	314	173	260
	29	244	365	202	302	154	231	228	343	198	297	164	246
	30	232	348	192	287	146	219	217	326	187	281	155	232
	32	209	314	172	259	130	195	195	293	166	249	137	205
	34	187	281	154	231	115	173	173	260	147	221	121	182
	36	167	251	137	206	103	154	155	232	131	197	108	162
	38	150	225	123	185	921	138	139	208	118	177	97.1	146
	40	135	203	111	167	83.1	125	125	188	106	159	87.6	131
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	124	187	98	147	69.7	105	127	191	106	160	83.8	126
$P_e(L_c)^2/10^4$, kip-in.^{2}		7110		5840		4370		6400		5580		4600	
r_m, in.		3.64		3.68		3.72		3.34		3.38		3.41	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

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④
COMPOSITE
HSS10.000–
HSS9.625

Table IV-3A (continued)

Available Strength in Axial Compression, kips

Filled Round HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS10.000×						HSS9.625×					
		0.312		0.250		0.188		0.500		0.375		0.312	
t_{des}, in,		0.291		0.233		0.174		0.465		0.349		0.291	
Steel, lb/ft		32.3		26.1		19.7		48.8		37.1		31.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	337	505	300	450	263	394	421	631	353	530	318	477
	1	336	504	300	450	262	393	421	631	353	530	318	477
	2	335	503	299	448	261	392	419	629	352	528	317	475
	3	334	500	297	446	260	390	417	626	350	525	315	473
	4	331	497	295	443	258	387	414	621	348	521	313	469
	5	328	492	292	438	255	383	410	615	344	517	310	465
	6	325	487	289	433	252	378	406	608	340	511	306	459
	7	320	481	285	428	248	373	400	600	336	504	302	453
	8	316	474	281	421	244	366	394	591	331	496	297	446
	9	310	466	276	414	240	360	387	581	325	487	292	438
	10	305	457	270	405	235	352	380	569	318	478	286	429
	11	298	447	264	397	229	344	371	557	312	467	279	419
	12	291	437	258	387	223	335	363	544	304	456	273	409
	13	284	426	251	377	217	326	353	530	296	444	265	398
	14	277	415	244	367	211	316	344	516	288	432	258	387
	15	269	403	237	356	204	306	334	500	280	419	250	375
	16	261	391	229	344	197	295	323	485	271	406	242	363
	17	252	378	222	333	190	285	312	468	261	392	233	350
	18	243	365	214	321	182	274	301	452	252	378	225	337
	19	235	352	206	308	175	262	290	435	243	364	216	324
	20	226	338	197	296	167	251	278	418	233	349	207	311
	21	217	325	189	284	160	240	267	400	223	335	198	297
	22	207	311	181	271	152	229	255	383	213	320	189	284
	23	198	297	172	259	145	217	244	365	204	305	180	270
	24	189	284	164	246	137	206	232	348	194	291	171	257
	25	180	270	156	234	130	195	221	331	184	276	163	244
	26	171	257	148	222	123	184	209	314	175	262	154	231
	27	162	243	140	210	116	174	198	297	165	248	145	218
	28	154	230	132	198	109	163	187	281	156	234	137	206
	29	145	218	124	186	102	153	176	265	147	220	129	193
	30	137	205	117	175	95.3	143	166	249	138	207	121	181
	32	121	181	103	154	83.8	126	146	219	121	182	106	159
	34	107	160	90.9	136	74.2	111	129	194	108	161	94.0	141
	36	95.3	143	81.1	122	66.2	99.3	115	173	96.0	144	83.9	126
	38	85.5	128	72.8	109	59.4	89.1	103	155	86.1	129	75.3	113
	40	77.2	116	65.7	98.6	53.6	80.4	93.4	140	77.7	117	67.9	102
Properties													
M_n/Ω_b / $\phi_b M_n$	kip-ft	72.0	108	59.7	89.7	46.5	69.8	97.8	147	77.1	116	66.3	99.6
$P_e(L_c)^2/10^4$, kip-in.^{2}		4060		3450		2820		4910		4080		3570	
r_m, in.		3.43		3.45		3.47		3.24		3.28		3.30	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

4

COMPOSITE
HSS9.625–
HSS8.625

Table IV-3A (continued)

Available Strength in Axial Compression, kips

Filled Round HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS9.625×				HSS8.625×							
		0.250		0.188		0.625		0.500		0.375		0.322	
t_{des}, in,		0.233		0.174		0.581		0.465		0.349		0.300	
Steel, lb/ft		25.1		19.0		53.5		43.4		33.1		28.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	283	425	247	371	421	632	362	543	302	454	277	415
	1	283	424	247	371	421	631	362	542	302	453	276	414
	2	282	423	246	369	419	628	360	540	301	451	275	413
	3	280	421	245	367	416	624	358	537	299	448	273	410
	4	278	417	243	364	412	619	355	532	296	444	271	406
	5	275	413	240	360	408	611	351	526	293	439	268	402
	6	272	408	237	355	402	603	346	519	289	433	264	396
	7	268	402	233	350	395	593	340	510	284	426	259	389
	8	264	395	229	344	387	581	334	500	278	418	254	382
	9	259	388	224	337	379	568	326	489	272	409	249	373
	10	253	380	219	329	370	554	318	478	266	399	243	364
	11	247	371	214	321	359	539	310	465	259	388	236	354
	12	241	361	208	312	349	523	301	451	251	377	229	344
	13	234	351	202	303	338	506	291	437	243	365	222	333
	14	227	341	195	293	326	489	281	422	235	352	214	321
	15	220	330	189	283	314	471	271	407	226	339	206	309
	16	213	319	182	273	301	452	261	391	217	326	198	297
	17	205	307	175	262	289	433	250	375	208	313	190	284
	18	197	295	168	251	276	414	239	358	199	299	181	272
	19	189	283	160	240	263	396	228	342	190	285	172	259
	20	181	271	153	229	251	378	217	325	181	271	164	246
	21	173	259	146	218	239	360	205	308	171	257	155	233
	22	165	247	138	207	227	342	194	292	162	243	147	220
	23	157	235	131	197	215	324	183	275	153	229	138	208
	24	148	223	124	186	204	306	173	259	144	216	130	195
	25	141	211	117	175	192	289	162	243	135	203	122	183
	26	133	199	110	165	181	272	152	228	127	190	114	171
	27	125	188	103	155	170	255	142	213	118	177	106	160
	28	118	176	96.5	145	159	239	132	198	110	165	99.0	148
	29	110	165	89.9	135	148	223	123	185	102	154	92.2	138
	30	103	155	84.0	126	138	208	115	173	95.7	144	86.2	129
	32	90.6	136	73.8	111	122	183	101	152	84.1	126	75.8	114
	34	80.2	120	65.4	98.1	108	162	89.7	135	74.5	112	67.1	101
	36	71.6	107	58.3	87.5	96.2	145	80.0	120	66.4	99.7	59.9	89.8
	38	64.2	96.3	52.4	78.5	86.3	130	71.8	108	59.6	89.5	53.7	80.6
	40	58.0	86.9	47.2	70.9	77.9	117	64.8	97.5	53.8	80.7	48.5	72.7
Properties													
M_n/Ω_b $\phi_b M_n$	kip-ft	54.9	82.6	42.8	64.4	91.9	138	76.9	116	60.8	91.4	53.6	80.6
$P_e(L_c)^2/10^4$, kip-in.^{2}		3050		2480		3880		3400		2830		2550	
r_m, in.		3.32		3.34		2.85		2.89		2.93		2.95	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS8.625–
HSS7.500

Table IV-3A (continued)
Available Strength in Axial Compression, kips
Filled Round HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape	HSS8.625×				HSS7.625×				HSS7.500×			
	0.250		0.188		0.375		0.328		0.500		0.375	
t_{des}, in.	0.233		0.174		0.349		0.305		0.465		0.349	
Steel, lb/ft	22.4		17.0		29.1		25.6		37.4		28.6	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	241	361	208	313	255	383	235	352	301	452	249	374
1	240	360	208	312	255	382	234	351	301	451	249	373
2	239	359	207	311	253	380	233	350	299	449	248	372
3	238	356	206	309	251	377	231	347	297	445	246	368
4	235	353	204	306	248	373	229	343	293	440	243	364
5	232	349	201	302	245	367	225	338	289	433	239	358
6	229	344	198	297	240	361	221	332	283	425	235	352
7	225	337	194	291	235	353	216	325	277	415	229	344
8	220	331	190	285	230	344	211	317	270	405	224	335
9	215	323	185	278	223	335	205	308	262	393	217	326
10	210	315	180	270	216	325	199	299	254	381	210	315
11	204	306	175	262	209	313	192	288	245	367	203	304
12	198	296	169	254	201	302	185	278	235	353	195	293
13	191	286	163	245	193	290	178	266	225	338	187	280
14	184	276	157	235	185	277	170	255	215	323	179	268
15	177	265	150	225	176	264	162	243	205	307	170	255
16	170	254	144	216	167	251	154	231	194	291	161	242
17	162	243	137	205	158	238	146	219	183	275	152	229
18	155	232	130	195	150	224	138	206	173	259	144	215
19	147	220	123	185	141	211	129	194	162	243	135	202
20	139	209	117	175	132	198	121	182	152	228	126	189
21	132	198	110	165	123	185	113	170	142	214	118	176
22	124	186	103	155	115	172	106	159	133	200	109	164
23	117	175	96.6	145	107	160	98.2	147	124	187	101	152
24	109	164	90.2	135	98.6	148	90.8	136	115	173	93.2	140
25	102	154	84.0	126	90.9	136	83.6	125	107	160	85.9	129
26	95.5	143	77.8	117	84.0	126	77.3	116	98.6	148	79.4	119
27	88.6	133	72.2	108	77.9	117	71.7	108	91.4	137	73.6	110
28	82.4	124	67.1	101	72.4	109	66.7	100	85.0	128	68.5	103
29	76.8	115	62.6	93.8	67.5	101	62.2	93.2	79.3	119	63.8	95.7
30	71.8	108	58.5	87.7	63.1	94.6	58.1	87.1	74.1	111	59.6	89.5
32	63.1	94.7	51.4	77.1	55.5	83.2	51.1	76.6	65.1	97.8	52.4	78.6
34	55.9	83.9	45.5	68.3	49.1	73.7	45.2	67.8	57.7	86.7	46.4	69.7
36	49.9	74.8	40.6	60.9	43.8	65.7	40.3	60.5	51.4	77.3	41.4	62.1
38	44.8	67.1	36.4	54.7	39.3	59.0	36.2	54.3	46.2	69.4	37.2	55.8
40	40.4	60.6	32.9	49.3	35.5	53.2	32.7	49.0	41.7	62.6	33.5	50.3
Properties												
M_n/Ω_b / $\phi_b M_n$, kip-ft	43.4	65.2	33.9	50.9	46.5	69.9	41.6	62.5	56.6	85.1	44.9	67.4
$P_e(L_c)^2/10^4$, kip-in.^{2}	2120		1730		1860		1720		2110		1760	
r_m, in.	2.97		2.99		2.58		2.59		2.49		2.53	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS7.500–
HSS7.000

Table IV-3A (continued)

Available Strength in Axial Compression, kips

Filled Round HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape	HSS7.500×						HSS7.000×					
	0.312		0.250		0.188		0.500		0.375		0.312	
t_{des}, in,	0.291		0.233		0.174		0.465		0.349		0.291	
Steel, lb/ft	24.0		19.4		14.7		34.7		26.6		22.3	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r												
0	223	334	196	294	168	253	275	412	227	340	202	304
1	223	334	196	294	168	252	274	411	226	340	202	303
2	221	332	195	292	167	251	272	409	225	338	201	301
3	220	329	193	290	166	248	270	405	223	334	199	298
4	217	325	191	286	163	245	266	399	220	330	196	294
5	214	321	188	282	161	241	261	392	216	324	193	289
6	210	315	184	276	157	236	256	384	211	317	189	283
7	205	308	180	270	154	230	249	374	206	309	184	276
8	200	300	175	263	149	224	242	363	200	300	179	268
9	194	291	170	255	145	217	234	351	194	290	173	259
10	188	282	165	247	140	210	225	338	187	280	167	250
11	181	272	159	238	134	202	216	324	179	269	160	240
12	174	262	152	228	129	193	207	310	171	257	153	229
13	167	251	146	219	123	184	197	295	163	244	146	218
14	160	239	139	208	117	175	186	279	155	232	138	207
15	152	228	132	198	111	166	176	264	146	219	130	196
16	144	216	125	188	104	157	166	249	137	206	123	184
17	136	204	118	177	98.3	147	156	235	129	193	115	173
18	128	192	111	166	92.1	138	147	221	120	180	107	161
19	120	181	104	156	85.9	129	137	206	112	168	100	150
20	113	169	97.0	146	79.9	120	128	192	104	155	92.6	139
21	105	158	90.3	135	74.0	111	119	179	95.7	143	85.5	128
22	97.7	146	83.7	126	68.3	103	110	165	87.9	132	78.6	118
23	90.4	136	77.3	116	62.7	94.1	101	152	80.4	121	71.9	108
24	83.3	125	71.0	107	57.6	86.4	93.1	140	73.8	111	66.0	99.0
25	76.8	115	65.5	98.2	53.1	79.6	85.8	129	68.0	102	60.8	91.3
26	71.0	106	60.5	90.8	49.1	73.6	79.4	119	62.9	94.4	56.2	84.4
27	65.8	98.7	56.1	84.2	45.5	68.3	73.6	111	58.3	87.5	52.2	78.2
28	61.2	91.8	52.2	78.3	42.3	63.5	68.4	103	54.2	81.4	48.5	72.7
29	57.1	85.6	48.6	73.0	39.4	59.2	63.8	95.9	50.6	75.8	45.2	67.8
30	53.3	80.0	45.5	68.2	36.9	55.3	59.6	89.6	47.2	70.9	42.2	63.4
32	46.9	70.3	40.0	59.9	32.4	48.6	52.4	78.8	41.5	62.3	37.1	55.7
34	41.5	62.3	35.4	53.1	28.7	43.0	46.4	69.8	36.8	55.2	32.9	49.3
36	37.0	55.5	31.6	47.4	25.6	38.4	41.4	62.2	32.8	49.2	29.3	44.0
38	33.2	49.8	28.3	42.5	23.0	34.5	37.2	55.8	29.4	44.2	26.3	39.5
40	30.0	45.0	25.6	38.4	20.7	31.1						
Properties												
M_n/Ω_b / $\phi_b M_n$, kip-ft	38.6	58.0	32.1	48.2	25.1	37.7	48.6	73.1	38.6	58.0	33.2	50.0
$P_e(L_c)^2/10^4$, kip-in.^{2}	1580		1340		1090		1670		1400		1250	
r_m, in.	2.55		2.57		2.59		2.32		2.35		2.37	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_c = 2.00$	$\phi_c = 0.75$

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS7.000–
HSS6.875

Table IV-3A (continued)

Available Strength in Axial Compression, kips

Filled Round HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS7.000×						HSS6.875×					
		0.250		0.188		0.125		0.500		0.375		0.312	
t_{des}, in,		0.233		0.174		0.116		0.465		0.349		0.291	
Steel, lb/ft		18.0		13.7		9.19		34.1		26.1		21.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	178	266	152	228	126	189	268	402	222	332	198	296
	1	177	266	152	227	126	189	267	401	221	332	197	296
	2	176	264	151	226	125	187	266	399	220	330	196	294
	3	174	262	149	223	123	185	263	395	218	326	194	291
	4	172	258	147	220	121	182	259	389	215	322	191	287
	5	169	253	144	216	119	178	255	382	211	316	188	282
	6	165	248	141	211	116	174	249	373	206	309	184	275
	7	161	241	137	205	113	169	242	364	201	301	179	268
	8	156	234	133	199	109	163	235	353	195	292	174	260
	9	151	227	128	192	104	157	227	340	188	282	168	251
	10	145	218	123	184	100	150	218	327	181	271	161	242
	11	139	209	118	176	95.2	143	209	314	173	260	155	232
	12	133	200	112	168	90.3	135	199	299	165	248	148	221
	13	127	190	106	159	85.2	128	189	284	157	236	140	210
	14	120	180	100	151	80	120	179	269	149	223	133	199
	15	113	170	94.4	142	74.8	112	169	254	140	210	125	188
	16	106	160	88.4	133	69.7	104	159	239	132	198	118	176
	17	99.6	149	82.5	124	64.5	96.8	150	225	123	185	110	165
	18	92.9	139	76.6	115	59.5	89.2	140	211	115	172	102	154
	19	86.3	129	70.8	106	54.6	81.9	131	197	106	160	95.0	143
	20	79.8	120	65.2	97.8	49.9	74.8	122	183	98.3	147	87.8	132
	21	73.5	110	59.8	89.7	45.3	67.9	113	169	90.5	136	80.8	121
	22	67.4	101	54.5	81.8	41.3	61.9	104	156	82.7	124	73.9	111
	23	61.6	92.4	49.9	74.8	37.7	56.6	95.2	143	75.7	114	67.6	101
	24	56.6	84.9	45.8	68.7	34.7	52.0	87.4	131	69.5	104	62.1	93.2
	25	52.2	78.2	42.2	63.3	31.9	47.9	80.6	121	64.1	96.1	57.2	85.9
	26	48.2	72.3	39.0	58.5	29.5	44.3	74.5	112	59.2	88.9	52.9	79.4
	27	44.7	67.1	36.2	54.3	27.4	41.1	69.1	104	54.9	82.4	49.1	73.6
	28	41.6	62.4	33.7	50.5	25.5	38.2	64.2	96.5	51.1	76.6	45.6	68.4
	29	38.8	58.1	31.4	47.1	23.7	35.6	59.9	90.0	47.6	71.4	42.5	63.8
	30	36.2	54.3	29.3	44.0	22.2	33.3	55.9	84.1	44.5	66.7	39.7	59.6
	32	31.8	47.8	25.8	38.7	19.5	29.2	49.2	73.9	39.1	58.7	34.9	52.4
	34	28.2	42.3	22.8	34.2	17.3	25.9	43.5	65.5	34.6	52.0	30.9	46.4
	36	25.2	37.7	20.4	30.5	15.4	23.1	38.8	58.4	30.9	46.3	27.6	41.4
	38	22.6	33.9	18.3	27.4	13.8	20.7			27.7	41.6	24.8	37.2
	40			16.5	24.7	12.5	18.7						
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	27.6	41.5	21.6	32.5	15.2	22.9	46.7	70.2	37.1	55.8	32.0	48.0
$P_e(L_c)^2/10^4$, kip-in.^{2}		1070		866		656		1570		1320		1170	
r_m, in.		2.39		2.41		2.43		2.27		2.31		2.33	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS6.875–
HSS6.625

Table IV-3A (continued)
Available Strength in Axial Compression, kips
Filled Round HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape	HSS6.875×				HSS6.625×							
	0.250		0.188		0.500		0.432		0.375		0.312	
t_{des}, in.	0.233		0.174		0.465		0.402		0.349		0.291	
Steel, lb/ft	17.7		13.4		32.7		28.6		25.1		21.1	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r: 0	173	260	148	222	255	383	231	347	211	316	188	281
1	173	259	147	221	255	382	231	346	210	315	187	281
2	172	257	146	220	253	380	229	344	209	313	186	279
3	170	255	145	217	250	376	227	340	207	310	184	276
4	167	251	143	214	246	370	223	335	203	305	181	272
5	164	247	140	210	242	362	219	328	199	299	178	267
6	161	241	137	205	236	354	214	321	195	292	173	260
7	156	235	133	199	229	344	208	312	189	284	169	253
8	152	227	129	193	222	332	201	301	183	275	163	245
9	146	220	124	186	213	320	194	290	176	265	157	236
10	141	211	119	178	205	307	186	279	169	254	151	226
11	135	202	114	170	195	293	177	266	162	243	144	216
12	128	193	108	162	186	278	169	253	154	231	137	206
13	122	183	102	153	176	264	160	239	146	218	130	195
14	115	173	96.5	145	166	250	150	226	137	206	122	183
15	109	163	90.6	136	157	236	141	212	129	193	115	172
16	102	153	84.7	127	147	221	132	198	120	181	107	161
17	95.1	143	78.8	118	138	207	123	184	112	168	99.8	150
18	88.5	133	73.0	110	128	193	113	170	104	156	92.5	139
19	82.0	123	67.4	101	119	179	105	158	95.7	143	85.3	128
20	75.6	113	61.9	92.8	110	165	97.2	146	87.8	132	78.4	118
21	69.5	104	56.5	84.8	101	152	89.6	135	80.2	120	71.5	107
22	63.4	95.1	51.5	77.3	92.2	139	82.0	123	73.1	110	65.2	97.8
23	58.0	87.0	47.1	70.7	84.4	127	75.1	113	66.9	101	59.6	89.4
24	53.3	79.9	43.3	64.9	77.5	116	68.9	104	61.4	92.4	54.8	82.2
25	49.1	73.6	39.9	59.8	71.4	107	63.5	95.5	56.6	85.1	50.5	75.7
26	45.4	68.1	36.9	55.3	66.0	99.3	58.7	88.3	52.4	78.7	46.7	70.0
27	42.1	63.1	34.2	51.3	61.2	92.0	54.5	81.9	48.5	73.0	43.3	64.9
28	39.1	58.7	31.8	47.7	56.9	85.6	50.6	76.1	45.1	67.9	40.2	60.4
29	36.5	54.7	29.6	44.5	53.1	79.8	47.2	71.0	42.1	63.3	37.5	56.3
30	34.1	51.1	27.7	41.5	49.6	74.6	44.1	66.3	39.3	59.1	35.1	52.6
32	30.0	44.9	24.3	36.5	43.6	65.5	38.8	58.3	34.6	51.9	30.8	46.2
34	26.5	39.8	21.6	32.3	38.6	58.0	34.4	51.6	30.6	46.0	27.3	40.9
36	23.7	35.5	19.2	28.9	34.4	51.8	30.6	46.1	27.3	41.0	24.3	36.5
38	21.2	31.9	17.3	25.9								
Properties												
M_n/Ω_b / $\phi_b M_n$, kip-ft	26.6	39.9	20.8	31.2	43.0	64.7	38.3	57.6	34.2	51.4	29.5	44.3
$P_e(L_c)^2/10^4$, kip-in.^{2}	1010		819		1390		1270		1160		1040	
r_m, in.	2.35		2.37		2.18		2.20		2.22		2.24	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_c = 2.00$	$\phi_c = 0.75$

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS6.625–
HSS6.000

Table IV-3A (continued)

Available Strength in Axial Compression, kips

Filled Round HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape	HSS6.625×								HSS6.000×			
	0.280		0.250		0.188		0.125		0.500		0.375	
t_{des}, in.	0.260		0.233		0.174		0.116		0.465		0.349	
Steel, lb/ft	19.0		17.0		12.9		8.69		29.4		22.6	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r												
0	175	263	164	246	140	210	116	173	224	337	185	277
1	175	262	164	246	140	209	115	173	224	336	184	276
2	174	261	163	244	139	208	114	171	222	333	183	274
3	172	258	161	241	137	206	113	169	219	328	180	270
4	169	254	159	238	135	202	111	166	215	322	177	265
5	166	249	155	233	132	198	108	163	210	314	173	259
6	162	243	152	227	129	193	105	158	204	306	168	251
7	157	236	147	221	125	187	102	153	197	296	162	243
8	152	229	143	214	121	181	98.1	147	190	285	155	233
9	147	220	137	206	116	174	94.0	141	182	273	149	223
10	141	211	132	197	111	166	89.5	134	173	260	141	212
11	135	202	126	188	106	158	84.8	127	164	247	133	200
12	128	192	119	179	100	150	80.0	120	155	233	125	188
13	121	182	113	169	94.5	142	75.1	113	146	219	117	176
14	114	171	106	160	88.7	133	70.1	105	136	204	109	164
15	107	161	99.7	150	82.9	124	65.1	97.6	126	190	101	151
16	100	150	93.1	140	77.2	116	60.1	90.2	117	176	92.9	139
17	93.2	140	86.5	130	71.5	107	55.3	82.9	108	162	85.1	128
18	86.4	130	80.1	120	65.9	98.8	50.6	75.8	98.4	148	77.9	117
19	79.6	119	73.8	111	60.5	90.7	46.0	69.0	89.7	135	71.2	107
20	73.1	110	67.6	101	55.2	82.8	41.6	62.4	81.1	122	64.7	97.3
21	66.8	100	61.6	92.5	50.1	75.1	37.7	56.6	73.6	111	58.7	88.2
22	60.8	91.3	56.2	84.3	45.6	68.5	34.4	51.5	67.0	101	53.5	80.4
23	55.7	83.5	51.4	77.1	41.8	62.6	31.4	47.2	61.3	92.2	48.9	73.5
24	51.1	76.7	47.2	70.8	38.4	57.5	28.9	43.3	56.3	84.6	44.9	67.5
25	47.1	70.7	43.5	65.2	35.3	53.0	26.6	39.9	51.9	78	41.4	62.3
26	43.6	65.3	40.2	60.3	32.7	49.0	24.6	36.9	48.0	72.1	38.3	57.6
27	40.4	60.6	37.3	55.9	30.3	45.5	22.8	34.2	44.5	66.9	35.5	53.4
28	37.6	56.3	34.7	52.0	28.2	42.3	21.2	31.8	41.4	62.2	33.0	49.6
29	35.0	52.5	32.3	48.5	26.3	39.4	19.8	29.7	38.6	58	30.8	46.3
30	32.7	49.1	30.2	45.3	24.5	36.8	18.5	27.7	36.0	54.2	28.8	43.2
32	28.8	43.1	26.5	39.8	21.6	32.4	16.2	24.4	31.7	47.6	25.3	38.0
34	25.5	38.2	23.5	35.3	19.1	28.7	14.4	21.6				
36	22.7	34.1	21.0	31.5	17.0	25.6	12.8	19.2				
38					15.3	22.9	11.5	17.3				
Properties												
M_n/Ω_b | $\phi_b M_n$ | kip-ft	26.9	40.4	24.5	36.9	19.2	28.9	13.6	20.4	34.5	51.9	27.5	41.4
$P_e(L_c)^2/10^4$, kip-in.^{2}	967		893		726		546		994		830	
r_m, in.	2.25		2.26		2.28		2.30		1.96		2.00	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS6.000–
HSS5.563

Table IV-3A (continued)
Available Strength in Axial Compression, kips
Filled Round HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS6.000×										HSS5.563×	
		0.312		0.280		0.250		0.188		0.125		0.500	
t_{des}, in,		0.291		0.260		0.233		0.174		0.116		0.465	
Steel, lb/ft		19.0		17.1		15.4		11.7		7.85		27.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	164	246	153	229	143	214	121	181	98.9	148	205	308
	1	163	245	152	228	142	214	120	181	98.6	148	205	308
	2	162	243	151	227	141	212	119	179	97.7	147	203	305
	3	160	240	149	224	139	209	118	177	96.2	144	200	300
	4	157	236	146	219	137	205	116	173	94.2	141	196	294
	5	153	230	143	214	134	200	113	169	91.6	137	191	286
	6	149	223	139	208	130	195	109	164	88.6	133	184	277
	7	144	216	134	201	125	188	105	158	85.2	128	178	267
	8	138	207	129	193	120	181	101	152	81.4	122	170	255
	9	132	198	123	185	115	173	96.4	145	77.3	116	162	243
	10	126	188	117	175	109	164	91.4	137	73.0	109	153	229
	11	119	178	111	166	103	155	86.2	129	68.5	103	143	216
	12	112	167	104	156	97.3	146	80.9	121	63.8	95.8	134	201
	13	104	157	97.3	146	91.0	137	75.4	113	59.2	88.8	125	187
	14	97.2	146	90.5	136	84.7	127	70.0	105	54.5	81.8	115	173
	15	89.9	135	83.8	126	78.4	118	64.5	96.8	49.9	74.9	106	159
	16	82.8	124	77.1	116	72.2	108	59.2	88.8	45.4	68.2	96.3	145
	17	75.8	114	70.6	106	66.1	99.2	54.0	81.0	41.1	61.7	87.3	131
	18	69.1	104	64.3	96.5	60.2	90.4	49.0	73.4	36.9	55.3	78.6	118
	19	62.5	93.7	58.2	87.3	54.5	81.8	44.1	66.1	33.1	49.7	70.6	106
	20	56.4	84.6	52.5	78.8	49.2	73.8	39.8	59.6	29.9	44.8	63.7	95.7
	21	51.2	76.7	47.6	71.4	44.6	66.9	36.1	54.1	27.1	40.6	57.8	86.8
	22	46.6	69.9	43.4	65.1	40.7	61.0	32.9	49.3	24.7	37.0	52.6	79.1
	23	42.6	64.0	39.7	59.6	37.2	55.8	30.1	45.1	22.6	33.9	48.2	72.4
	24	39.2	58.7	36.5	54.7	34.2	51.2	27.6	41.4	20.7	31.1	44.2	66.5
	25	36.1	54.1	33.6	50.4	31.5	47.2	25.4	38.2	19.1	28.7	40.8	61.3
	26	33.4	50.1	31.1	46.6	29.1	43.7	23.5	35.3	17.7	26.5	37.7	56.6
	27	30.9	46.4	28.8	43.2	27.0	40.5	21.8	32.7	16.4	24.6	34.9	52.5
	28	28.8	43.2	26.8	40.2	25.1	37.6	20.3	30.4	15.2	22.9	32.5	48.8
	29	26.8	40.2	25.0	37.5	23.4	35.1	18.9	28.4	14.2	21.3	30.3	45.5
	30	25.1	37.6	23.3	35.0	21.9	32.8	17.7	26.5	13.3	19.9	28.3	42.5
	32	22.0	33.0	20.5	30.8	19.2	28.8	15.5	23.3	11.7	17.5		
	34					17.0	25.5	13.8	20.6	10.3	15.5		
Properties													
M_n/Ω_b / $\phi_b M_n$	kip-ft	23.8	35.7	21.7	32.6	19.8	29.7	15.5	23.3	11.0	16.5	29.2	43.9
$P_e(L_c)^2/10^4$, kip-in.^{2}		741		690		646		522		392		769	
r_m, in.		2.02		2.03		2.04		2.06		2.08		1.81	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS5.563–
HSS5.500

Table IV-3A (continued)
Available Strength in Axial Compression, kips
Filled Round HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS5.563×								HSS5.500×			
		0.375		0.258		0.188		0.134		0.500		0.375	
t_{des}, in.		0.349		0.240		0.174		0.124		0.465		0.349	
Steel, lb/ft		20.8		14.6		10.8		7.78		26.7		20.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	167	250	131	196	108	163	90.9	136	203	305	164	247
	1	166	250	130	196	108	162	90.6	136	202	304	164	246
	2	165	247	129	194	107	161	89.7	135	200	301	162	243
	3	162	243	127	191	105	158	88.2	132	197	297	160	240
	4	159	238	124	187	103	155	86.1	129	193	290	156	234
	5	154	231	121	182	100	150	83.5	125	188	283	152	228
	6	149	224	117	175	96.6	145	80.4	121	182	273	146	220
	7	143	215	112	169	92.7	139	76.9	115	175	263	141	211
	8	137	205	107	161	88.3	132	73.0	110	167	251	134	201
	9	130	194	102	153	83.7	125	68.9	103	159	239	127	190
	10	122	183	95.9	144	78.7	118	64.6	96.9	150	225	119	179
	11	114	171	89.8	135	73.6	110	60.1	90.2	141	211	112	167
	12	106	160	83.7	125	68.4	103	55.6	83.3	131	197	104	156
	13	98.4	148	77.4	116	63.1	94.7	51.0	76.5	122	183	95.7	144
	14	90.5	136	71.2	107	57.9	86.8	46.5	69.8	112	168	88.3	133
	15	83.3	125	65.1	97.6	52.7	79.1	42.1	63.2	103	154	81.2	122
	16	76.3	115	59.1	88.6	47.8	71.6	37.9	56.8	93.5	141	74.2	112
	17	69.5	105	53.3	80.0	43.0	64.5	33.8	50.7	84.6	127	67.5	101
	18	63.0	94.7	47.8	71.6	38.4	57.5	30.1	45.2	76.0	114	61.0	91.6
	19	56.6	85.1	42.9	64.3	34.4	51.6	27.0	40.5	68.2	102	54.7	82.2
	20	51.1	76.8	38.7	58.0	31.1	46.6	24.4	36.6	61.5	92.5	49.4	74.2
	21	46.3	69.6	35.1	52.6	28.2	42.3	22.1	33.2	55.8	83.9	44.8	67.3
	22	42.2	63.5	32.0	47.9	25.7	38.5	20.2	30.2	50.9	76.4	40.8	61.3
	23	38.6	58.1	29.2	43.9	23.5	35.2	18.4	27.7	46.5	69.9	37.3	56.1
	24	35.5	53.3	26.9	40.3	21.6	32.4	16.9	25.4	42.7	64.2	34.3	51.5
	25	32.7	49.1	24.8	37.1	19.9	29.8	15.6	23.4	39.4	59.2	31.6	47.5
	26	30.2	45.4	22.9	34.3	18.4	27.6	14.4	21.7	36.4	54.7	29.2	43.9
	27	28.0	42.1	21.2	31.8	17.0	25.6	13.4	20.1	33.8	50.7	27.1	40.7
	28	26.1	39.2	19.7	29.6	15.9	23.8	12.4	18.7	31.4	47.2	25.2	37.9
	29	24.3	36.5	18.4	27.6	14.8	22.2	11.6	17.4	29.3	44.0	23.5	35.3
	30	22.7	34.1	17.2	25.8	13.8	20.7	10.8	16.3			21.9	33.0
	32							9.53	14.3				

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	23.3	35.0	17.2	25.9	13.2	19.8	9.89	14.9	28.4	42.8	22.7	34.2
$P_e(L_c)^2/10^4$, kip-in.^{2}			643		508		408		320		739		619	
r_m, in.			1.85		1.88		1.91		1.92		1.79		1.83	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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4

COMPOSITE
HSS5.500–
HSS5.000

Table IV-3A (continued)

Available Strength in Axial Compression, kips

Filled Round HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape	HSS5.500×		HSS5.000×									
	0.258		0.500		0.375		0.312		0.258		0.250	
t_{des}, in.	0.240		0.465		0.349		0.291		0.240		0.233	
Steel, lb/ft	14.5		24.1		18.5		15.6		13.1		12.7	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r												
0	129	193	182	274	145	217	128	192	113	170	111	166
1	128	193	182	273	144	217	128	191	113	169	111	166
2	127	191	180	270	143	214	126	189	111	167	109	164
3	125	188	176	265	140	210	124	186	109	164	107	161
4	123	184	172	258	136	204	120	181	106	159	104	157
5	119	179	166	250	132	197	116	174	103	154	101	151
6	115	172	159	240	126	189	111	167	98.5	148	96.6	145
7	110	165	152	228	120	180	106	159	93.7	141	91.9	138
8	105	158	144	216	113	170	100	150	88.5	133	86.8	130
9	99.6	149	135	202	106	159	93.8	141	82.9	124	81.3	122
10	93.8	141	125	189	98.4	148	87.2	131	77.1	116	75.6	113
11	87.7	132	116	174	91.3	137	80.4	121	71.1	107	69.8	105
12	81.6	122	106	160	84.2	126	73.6	110	65.1	97.6	63.9	95.8
13	75.3	113	97.0	146	77.0	116	66.9	100	59.1	88.7	58.0	87.1
14	69.1	104	87.7	132	69.9	105	60.3	90.4	53.3	80.0	52.3	78.5
15	63.0	94.6	78.7	118	63.1	94.8	54.2	81.5	47.7	71.6	46.8	70.2
16	57.1	85.7	70.0	105	56.5	84.9	48.7	73.2	42.3	63.4	41.5	62.2
17	51.4	77.2	62.0	93.2	50.1	75.4	43.3	65.1	37.5	56.2	36.8	55.1
18	45.9	68.9	55.3	83.1	44.7	67.2	38.6	58.1	33.4	50.1	32.8	49.2
19	41.2	61.8	49.6	74.6	40.1	60.3	34.7	52.1	30.0	45.0	29.4	44.1
20	37.2	55.8	44.8	67.3	36.2	54.5	31.3	47.0	27.1	40.6	26.6	39.8
21	33.8	50.6	40.6	61.0	32.9	49.4	28.4	42.7	24.5	36.8	24.1	36.1
22	30.8	46.1	37.0	55.6	29.9	45.0	25.9	38.9	22.4	33.5	21.9	32.9
23	28.1	42.2	33.9	50.9	27.4	41.2	23.7	35.6	20.5	30.7	20.1	30.1
24	25.8	38.8	31.1	46.7	25.2	37.8	21.7	32.7	18.8	28.2	18.4	27.7
25	23.8	35.7	28.7	43.1	23.2	34.9	20.0	30.1	17.3	26.0	17.0	25.5
26	22.0	33.0	26.5	39.8	21.4	32.2	18.5	27.8	16.0	24.0	15.7	23.6
27	20.4	30.6			19.9	29.9	17.2	25.8	14.8	22.3	14.6	21.9
28	19.0	28.5							13.8	20.7	13.5	20.3
29	17.7	26.5										
30	16.5	24.8										

Properties

	HSS5.500×0.258		HSS5.000×0.500		HSS5.000×0.375		HSS5.000×0.312		HSS5.000×0.258		HSS5.000×0.250	
M_n/Ω_b | $\phi_b M_n$ | kip-ft	16.8	25.2	23.0	34.5	18.4	27.7	15.9	24.0	13.6	20.5	13.3	20.0
$P_e(L_c)^2/10^4$, kip-in.^{2}	489		534		450		401		355		349	
r_m, in.	1.86		1.61		1.65		1.67		1.69		1.69	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

4
COMPOSITE
HSS5.000–
HSS4.500

Table IV-3A (continued)
Available Strength in Axial Compression, kips
Filled Round HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape	HSS5.000×				HSS4.500×							
	0.188		0.125		0.375		0.337		0.237		0.188	
t_{des}, in.	0.174		0.116		0.349		0.313		0.220		0.174	
Steel, lb/ft	9.67		6.51		16.5		15.0		10.8		8.67	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r												
0	93.0	140	74.9	112	126	189	117	176	92.7	139	80.0	120
1	92.6	139	74.6	112	126	188	117	175	92.2	138	79.6	119
2	91.6	137	73.6	110	124	186	115	172	90.9	136	78.5	118
3	89.8	135	72.1	108	121	181	112	168	88.8	133	76.7	115
4	87.4	131	70.0	105	117	175	108	163	85.9	129	74.2	111
5	84.4	127	67.4	101	112	168	104	156	82.3	123	71.1	107
6	80.8	121	64.4	96.6	107	160	98.5	148	78.1	117	67.5	101
7	76.8	115	61.0	91.4	101	151	92.6	139	73.5	110	63.4	95.2
8	72.5	109	57.2	85.9	94.1	141	86.1	129	68.4	103	59.1	88.6
9	67.8	102	53.3	79.9	87.2	131	79.4	119	63.1	94.7	54.5	81.8
10	63.0	94.5	49.2	73.8	80.1	120	72.9	110	57.7	86.5	49.8	74.7
11	58.0	87.1	45.1	67.6	72.9	110	66.5	99.9	52.2	78.3	45.1	67.7
12	53.1	79.6	40.9	61.4	65.7	98.8	60.0	90.2	46.8	70.3	40.4	60.7
13	48.1	72.2	36.8	55.3	58.8	88.3	53.7	80.8	41.6	62.4	35.9	53.9
14	43.3	65.0	32.9	49.3	52.1	78.2	47.7	71.7	36.6	54.9	31.6	47.4
15	38.7	58.1	29.1	43.7	45.6	68.6	41.9	62.9	31.9	47.9	27.6	41.3
16	34.2	51.3	25.6	38.4	40.1	60.3	36.8	55.3	28.0	42.1	24.2	36.3
17	30.3	45.5	22.7	34.0	35.5	53.4	32.6	49.0	24.8	37.3	21.5	32.2
18	27.0	40.6	20.2	30.3	31.7	47.6	29.1	43.7	22.2	33.2	19.1	28.7
19	24.3	36.4	18.1	27.2	28.4	42.7	26.1	39.2	19.9	29.8	17.2	25.8
20	21.9	32.9	16.4	24.6	25.7	38.6	23.5	35.4	17.9	26.9	15.5	23.2
21	19.9	29.8	14.9	22.3	23.3	35.0	21.4	32.1	16.3	24.4	14.1	21.1
22	18.1	27.2	13.5	20.3	21.2	31.9	19.5	29.3	14.8	22.2	12.8	19.2
23	16.6	24.8	12.4	18.6	19.4	29.2	17.8	26.8	13.6	20.4	11.7	17.6
24	15.2	22.8	11.4	17.1	17.8	26.8	16.4	24.6	12.5	18.7	10.8	16.1
25	14.0	21.0	10.5	15.7					11.5	17.2	9.92	14.9
26	13.0	19.4	9.69	14.5								
27	12.0	18.0	8.99	13.5								
28	11.2	16.8	8.36	12.5								
Properties												
M_n/Ω_b | $\phi_b M_n$ kip-ft	10.5	15.7	7.43	11.2	14.6	21.9	13.4	20.1	10.1	15.2	8.32	12.5
$P_e(L_c)^2/10^4$, kip-in.^{2}	288		215		314		294		236		204	
r_m, in.	1.71		1.73		1.47		1.48		1.52		1.53	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS4.500–
HSS4.000

Table IV-3A (continued)
Available Strength in Axial Compression, kips
Filled Round HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape	HSS4.500×		HSS4.000×									
	0.125		0.313		0.250		0.237		0.226		0.220	
t_{des}, in.	0.116		0.291		0.233		0.220		0.210		0.205	
Steel, lb/ft	5.85		12.3		10.0		9.53		9.12		8.89	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r: 0	64.0	96.0	95.4	143	82.1	123	78.9	118	76.6	115	75.4	113
1	63.7	95.5	94.8	142	81.6	122	78.5	118	76.2	114	74.9	112
2	62.7	94.0	93.1	140	80.2	120	77.1	116	74.8	112	73.6	110
3	61.1	91.7	90.3	135	77.8	117	74.8	112	72.6	109	71.4	107
4	59.0	88.5	86.5	130	74.6	112	71.7	108	69.6	104	68.5	103
5	56.4	84.5	81.9	123	70.6	106	67.9	102	65.9	98.9	64.9	97.3
6	53.3	80.0	76.6	115	66.1	99.2	63.6	95.4	61.7	92.6	60.7	91.1
7	49.9	74.9	71.1	107	61.1	91.7	58.8	88.2	57.1	85.6	56.2	84.3
8	46.3	69.4	65.4	98.3	55.8	83.8	53.7	80.6	52.2	78.3	51.3	77.0
9	42.4	63.6	59.5	89.5	50.4	75.6	48.5	72.8	47.1	70.7	46.4	69.6
10	38.5	57.8	53.6	80.5	45.0	67.4	43.3	64.9	42.0	63.0	41.4	62.1
11	34.6	52.0	47.7	71.6	39.6	59.4	38.2	57.2	37.1	55.6	36.5	54.7
12	30.8	46.2	41.9	63.0	34.6	51.9	33.2	49.9	32.3	48.4	31.8	47.7
13	27.2	40.7	36.5	54.8	30.1	45.3	28.9	43.4	27.7	41.6	27.3	41.0
14	23.6	35.4	31.5	47.3	26.0	39.1	25.0	37.5	23.9	35.9	23.5	35.3
15	20.6	30.9	27.4	41.2	22.6	34.0	21.7	32.7	20.8	31.3	20.5	30.8
16	18.1	27.1	24.1	36.2	19.9	29.9	19.1	28.7	18.3	27.5	18.0	27.0
17	16.0	24.0	21.3	32.1	17.6	26.5	16.9	25.4	16.2	24.4	16.0	23.9
18	14.3	21.4	19.0	28.6	15.7	23.6	15.1	22.7	14.5	21.7	14.2	21.4
19	12.8	19.2	17.1	25.7	14.1	21.2	13.6	20.4	13.0	19.5	12.8	19.2
20	11.6	17.4	15.4	23.2	12.7	19.1	12.2	18.4	11.7	17.6	11.5	17.3
21	10.5	15.8	14.0	21.0	11.6	17.4	11.1	16.7	10.6	16.0	10.5	15.7
22	9.57	14.4	12.7	19.1	10.5	15.8	10.1	15.2	9.68	14.6	9.53	14.3
23	8.76	13.1										
24	8.04	12.1										
25	7.41	11.1										
Properties												
M_n/Ω_b / $\phi_b M_n$, kip-ft	5.92	8.90	9.74	14.6	8.17	12.3	7.80	11.7	7.51	11.3	7.36	11.1
$P_e(L_c)^2/10^4$, kip-in.^{2}	152		189		164		158		154		152	
r_m, in.	1.55		1.32		1.33		1.34		1.34		1.34	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.

Dashed line indicates the L_c beyond which the bare steel strength controls.

4

COMPOSITE
HSS4.000

Table IV-3A (continued)
Available Strength in Axial Compression, kips
Filled Round HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS4.000×			
		0.188		0.125	
t_{des}, in,		0.174		0.116	
Steel, lb/ft		7.66		5.18	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	68.0	102	53.8	80.8
	1	67.6	101	53.5	80.3
	2	66.4	99.5	52.5	78.8
	3	64.4	96.6	50.9	76.3
	4	61.8	92.6	48.7	73.0
	5	58.5	87.8	46.0	69.0
	6	54.8	82.2	42.9	64.4
	7	50.7	76.0	39.6	59.4
	8	46.3	69.5	36.0	54.0
	9	41.8	62.7	32.4	48.6
	10	37.3	56.0	28.7	43.1
	11	32.9	49.4	25.2	37.8
	12	28.7	43.0	21.8	32.7
	13	24.6	36.9	18.6	27.9
	14	21.2	31.9	16.1	24.1
	15	18.5	27.8	14.0	21.0
	16	16.3	24.4	12.3	18.4
	17	14.4	21.6	10.9	16.3
	18	12.8	19.3	9.71	14.6
	19	11.5	17.3	8.72	13.1
	20	10.4	15.6	7.87	11.8
	21	9.44	14.2	7.13	10.7
	22	8.60	12.9	6.50	9.75

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	6.44	9.68	4.59	6.90
$P_e(L_c)^2/10^4$, kip-in.^{2}			137		103	
r_m, in.			1.35		1.37	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Heavy line indicates L_c/r_m equal to or greater than 200.

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COMPOSITE
HSS20.000–
HSS16.000

Table IV-3B
Available Strength in Axial Compression, kips
Filled Round HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS20.000×				HSS18.000×				HSS16.000×			
		0.500		0.375		0.500		0.375		0.625		0.500	
t_{des}, in.		0.465		0.349		0.465		0.349		0.581		0.465	
Steel, lb/ft		104		78.7		93.5		70.7		103		82.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	1330	2000	1190	1780	1130	1700	1000	1510	1060	1590	946	1420
	1	1330	2000	1190	1780	1130	1700	1000	1510	1060	1580	945	1420
	2	1330	2000	1190	1780	1130	1700	1000	1500	1060	1580	944	1420
	3	1330	2000	1190	1780	1130	1690	1000	1500	1050	1580	942	1410
	4	1330	1990	1180	1780	1130	1690	999	1500	1050	1580	939	1410
	5	1320	1990	1180	1770	1120	1680	995	1490	1050	1570	936	1400
	6	1320	1980	1180	1760	1120	1680	991	1490	1040	1560	932	1400
	7	1320	1970	1170	1760	1110	1670	987	1480	1040	1550	927	1390
	8	1310	1970	1170	1750	1110	1660	981	1470	1030	1550	921	1380
	9	1300	1960	1160	1740	1100	1650	975	1460	1020	1540	914	1370
	10	1300	1950	1150	1730	1090	1640	968	1450	1020	1520	907	1360
	11	1290	1930	1150	1720	1090	1630	961	1440	1010	1510	899	1350
	12	1280	1920	1140	1710	1080	1620	953	1430	998	1500	891	1340
	13	1270	1910	1130	1690	1070	1600	944	1420	988	1480	881	1320
	14	1260	1890	1120	1680	1060	1590	935	1400	978	1470	871	1310
	15	1250	1880	1110	1670	1050	1570	925	1390	966	1450	861	1290
	16	1240	1860	1100	1650	1040	1560	915	1370	955	1430	850	1270
	17	1230	1840	1090	1630	1030	1540	904	1360	942	1410	838	1260
	18	1220	1830	1080	1620	1010	1520	892	1340	929	1390	826	1240
	19	1200	1810	1070	1600	1000	1500	880	1320	915	1370	813	1220
	20	1190	1790	1050	1580	989	1480	868	1300	901	1350	800	1200
	21	1180	1770	1040	1560	975	1460	855	1280	887	1330	787	1180
	22	1160	1750	1030	1540	961	1440	841	1260	872	1310	773	1160
	23	1150	1720	1010	1520	946	1420	828	1240	856	1280	758	1140
	24	1130	1700	998	1500	932	1400	814	1220	840	1260	744	1120
	25	1120	1680	983	1470	916	1370	799	1200	824	1240	728	1090
	26	1100	1650	968	1450	900	1350	784	1180	807	1210	713	1070
	27	1090	1630	952	1430	884	1330	769	1150	790	1190	698	1050
	28	1070	1600	936	1400	868	1300	754	1130	773	1160	682	1020
	29	1050	1580	920	1380	851	1280	738	1110	756	1130	666	998
	30	1030	1550	904	1360	835	1250	723	1080	738	1110	649	974
	32	999	1500	870	1310	800	1200	691	1040	703	1050	617	925
	34	963	1440	836	1250	765	1150	658	987	667	1000	584	875
	36	925	1390	801	1200	730	1090	625	938	630	946	550	826
	38	888	1330	765	1150	694	1040	592	888	594	892	517	776
	40	849	1270	730	1090	658	987	559	839	559	838	485	727
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	487	731	381	572	388	583	304	457	361	543	300	452
$P_e(L_c)^2/10^4$, kip-in.^{2}		57000		47000		40300		33100		32000		27300	
r_m, in.		6.91		6.95		6.20		6.24		5.46		5.49	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

COMPOSITE
HSS16.000–
HSS14.000

Table IV-3B (continued)
Available Strength in Axial Compression, kips
Filled Round HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS16.000×								HSS14.000×			
		0.438		0.375		0.312		0.250		0.625		0.500	
t_{des}, in,		0.407		0.349		0.291		0.233		0.581		0.465	
Steel, lb/ft		72.9		62.6		52.3		42.1		89.4		72.2	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	888	1330	832	1250	775	1160	715	1070	871	1310	774	1160
	1	888	1330	832	1250	774	1160	714	1070	870	1310	774	1160
	2	886	1330	831	1250	773	1160	713	1070	869	1300	772	1160
	3	885	1330	829	1240	771	1160	711	1070	867	1300	770	1160
	4	882	1320	826	1240	769	1150	709	1060	864	1300	767	1150
	5	878	1320	823	1230	765	1150	706	1060	860	1290	764	1150
	6	874	1310	819	1230	761	1140	702	1050	855	1280	759	1140
	7	869	1300	814	1220	757	1140	697	1050	849	1270	754	1130
	8	864	1300	809	1210	751	1130	692	1040	843	1260	748	1120
	9	858	1290	803	1200	745	1120	686	1030	835	1250	741	1110
	10	851	1280	796	1190	739	1110	680	1020	827	1240	734	1100
	11	843	1260	788	1180	732	1100	672	1010	818	1230	726	1090
	12	835	1250	780	1170	724	1090	665	997	809	1210	717	1080
	13	826	1240	772	1160	715	1070	656	985	798	1200	708	1060
	14	816	1220	762	1140	706	1060	648	971	787	1180	698	1050
	15	806	1210	752	1130	697	1040	638	957	776	1160	687	1030
	16	795	1190	742	1110	686	1030	628	942	764	1150	676	1010
	17	784	1180	731	1100	676	1010	618	927	751	1130	664	996
	18	772	1160	720	1080	665	997	607	911	737	1110	652	978
	19	760	1140	708	1060	653	980	596	894	723	1090	639	959
	20	747	1120	696	1040	641	962	584	877	709	1060	626	939
	21	734	1100	683	1020	629	944	572	859	694	1040	612	919
	22	721	1080	670	1000	616	925	560	840	679	1020	599	898
	23	707	1060	657	985	603	905	548	822	664	995	585	877
	24	693	1040	643	964	590	885	535	802	648	972	570	855
	25	678	1020	629	943	577	865	522	783	632	947	555	833
	26	664	996	615	922	563	844	509	763	615	923	541	811
	27	649	973	600	900	549	824	495	743	599	898	526	788
	28	634	950	586	879	535	803	482	723	582	873	511	766
	29	618	927	571	856	521	781	468	702	565	848	495	743
	30	603	904	556	834	507	760	454	682	548	823	480	720
	32	571	857	526	789	478	717	427	640	514	772	449	674
	34	540	810	496	744	449	673	399	599	481	721	419	629
	36	508	762	466	698	420	630	372	558	447	671	389	584
	38	477	715	436	654	392	588	346	518	415	622	360	540
	40	446	669	406	609	364	546	320	479	383	574	331	497

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	269	404	236	355	202	304	165	247	270	406	225	338
$P_e(L_c)^2/10^4$, kip-in.^{2}			24900		22400		19800		17100		20400		17600	
r_m, in.			5.51		5.53		5.55		5.58		4.75		4.79	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

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COMPOSITE
HSS14.000–
HSS12.750

Table IV-3B (continued)
Available Strength in Axial Compression, kips
Filled Round HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS14.000×						HSS12.750×					
		0.375		0.312		0.250		0.500		0.375		0.250	
t_{des}, in.		0.349		0.291		0.233		0.465		0.349		0.233	
Steel, lb/ft		54.6		45.7		36.8		65.5		49.6		33.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	675	1010	623	935	574	861	672	1010	584	876	492	738
	1	675	1010	623	935	574	860	672	1010	583	875	492	738
	2	673	1010	622	933	572	859	671	1010	582	873	491	736
	3	672	1010	620	930	571	856	669	1000	580	870	489	733
	4	669	1000	618	926	568	852	666	999	578	866	486	730
	5	666	998	614	921	565	847	662	993	574	861	483	725
	6	661	992	610	915	561	842	657	986	570	855	479	719
	7	657	985	606	908	556	835	652	978	565	847	475	712
	8	651	976	600	900	551	827	646	969	559	839	469	704
	9	645	967	594	891	545	818	639	959	553	830	464	695
	10	638	957	588	881	539	808	632	947	546	819	457	686
	11	630	945	580	870	532	797	623	935	539	808	450	675
	12	622	933	572	859	524	786	614	921	530	796	442	664
	13	613	920	564	846	516	774	605	907	522	782	434	651
	14	604	906	555	833	507	761	595	892	512	768	426	639
	15	594	891	546	819	498	747	584	876	503	754	417	625
	16	584	876	536	804	488	732	573	859	492	738	407	611
	17	573	860	525	788	478	717	561	841	482	722	397	596
	18	562	843	515	772	468	701	549	823	470	706	387	581
	19	550	825	503	755	457	685	536	804	459	689	377	565
	20	538	807	492	738	446	669	523	785	447	671	366	549
	21	526	789	480	720	434	651	510	765	435	653	355	533
	22	513	770	468	702	423	634	496	744	423	634	344	516
	23	500	751	456	684	411	616	482	724	410	616	333	499
	24	487	731	443	665	399	598	468	703	398	597	321	482
	25	474	711	431	646	387	580	454	681	385	578	310	465
	26	461	691	418	627	374	562	440	660	372	558	298	448
	27	447	670	405	607	362	543	426	638	359	539	287	430
	28	433	650	392	588	350	524	411	617	346	520	276	413
	29	419	629	379	568	337	506	397	595	333	500	264	396
	30	406	609	366	549	325	487	382	573	321	481	253	379
	32	378	567	340	510	300	451	354	530	295	443	231	346
	34	351	527	314	472	276	415	326	488	270	406	209	314
	36	324	486	289	434	253	379	298	447	246	370	188	283
	38	298	447	265	398	230	346	272	408	223	334	169	254
	40	273	410	242	362	208	312	246	369	201	302	153	229
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	177	266	152	228	125	188	184	276	145	217	103	154
$P_e(L_c)^2/10^4$, kip-in.^{2}		14400		12700		10900		12900		10600		8020	
r_m, in.		4.83		4.85		4.87		4.35		4.39		4.43	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

5

COMPOSITE
HSS10.750–
HSS10.000

Table IV-3B (continued)

Available Strength in Axial Compression, kips

Filled Round HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS10.750×						HSS10.000×					
		0.500		0.375		0.250		0.625		0.500		0.375	
t_{des}, in.		0.465		0.349		0.233		0.581		0.465		0.349	
Steel, lb/ft		54.8		41.6		28.1		62.6		50.8		38.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	525	787	451	676	374	562	541	812	473	710	405	608
	1	524	787	450	675	374	561	541	811	473	709	405	607
	2	523	785	449	674	373	559	539	809	471	707	403	605
	3	521	781	447	671	371	557	536	805	469	703	401	602
	4	518	777	444	666	368	553	533	799	466	699	399	598
	5	514	770	441	661	365	548	528	792	462	692	395	592
	6	509	763	436	654	361	542	522	783	456	685	390	586
	7	503	755	431	647	357	535	515	773	451	676	385	578
	8	497	745	425	638	351	527	508	761	444	666	379	569
	9	489	734	419	628	345	518	499	749	436	655	373	559
	10	481	722	412	618	339	508	490	734	428	642	365	548
	11	473	709	404	606	332	498	479	719	419	629	358	536
	12	463	695	396	593	324	486	468	703	410	615	349	524
	13	453	680	387	580	316	474	457	685	400	599	340	510
	14	443	664	377	566	308	462	445	667	389	583	331	496
	15	432	648	368	552	299	449	432	648	378	567	321	482
	16	420	630	358	536	290	435	419	628	366	549	311	467
	17	408	612	347	520	281	421	405	608	354	532	301	451
	18	396	594	336	504	271	406	391	587	342	513	290	435
	19	384	575	325	488	261	392	377	565	330	495	279	419
	20	371	556	314	471	251	377	362	544	317	476	268	402
	21	358	537	302	454	241	362	348	522	304	457	257	385
	22	345	517	291	436	231	347	333	500	292	437	246	369
	23	331	497	279	419	221	331	318	478	279	418	235	352
	24	318	477	268	401	211	316	304	456	266	399	224	335
	25	305	457	256	384	201	301	289	434	253	380	213	319
	26	292	437	244	367	191	286	275	412	241	361	202	302
	27	279	418	233	350	181	271	261	391	228	342	191	286
	28	266	398	222	333	171	257	247	370	216	324	180	271
	29	253	379	211	316	162	243	233	349	204	306	170	255
	30	240	360	200	299	153	229	219	329	192	289	160	240
	32	216	323	178	268	135	202	195	293	170	254	141	211
	34	192	288	158	237	119	179	173	260	150	225	125	187
	36	171	257	141	212	106	159	155	232	134	201	111	167
	38	153	230	127	190	95.4	143	139	208	120	180	99.8	150
	40	139	208	114	171	86.1	129	125	188	109	163	90.0	135
Properties													
M_n/Ω_b / $\phi_b M_n$	kip-ft	126	190	99.9	150	71.2	107	129	194	108	162	85.4	128
$P_e(L_c)^2/10^4$, kip-in.^{2}		7280		6000		4520		6510		5700		4730	
r_m, in.		3.64		3.68		3.72		3.34		3.38		3.41	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS10.000–
HSS9.625

Table IV-3B (continued)
Available Strength in Axial Compression, kips
Filled Round HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS10.000×						HSS9.625×					
		0.312		0.250		0.188		0.500		0.375		0.312	
t_{des}, in,		0.291		0.233		0.174		0.465		0.349		0.291	
Steel, lb/ft		32.3		26.1		19.7		48.8		37.1		31.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	370	555	334	501	297	446	449	674	383	575	349	523
	1	369	554	334	500	297	445	449	673	383	574	348	523
	2	368	552	332	499	296	444	447	671	381	572	347	521
	3	366	549	331	496	294	441	445	667	379	569	345	518
	4	363	545	328	492	292	437	441	662	376	565	342	514
	5	360	540	325	487	288	433	437	656	373	559	339	508
	6	356	534	321	481	284	427	432	648	368	552	335	502
	7	351	526	316	474	280	420	426	639	363	545	330	495
	8	345	518	311	466	275	412	419	629	357	536	324	486
	9	339	509	305	457	269	404	412	617	350	526	318	477
	10	332	498	298	448	263	395	403	605	343	515	311	467
	11	325	487	291	437	256	385	394	591	335	503	304	456
	12	317	476	284	426	249	374	384	577	327	490	296	444
	13	309	463	276	414	242	363	374	561	318	477	287	431
	14	300	450	268	402	234	351	363	545	309	463	279	418
	15	291	436	259	389	226	339	352	528	299	449	270	404
	16	281	422	250	375	217	326	341	511	289	434	260	390
	17	272	407	241	362	209	313	329	493	279	418	251	376
	18	262	393	232	348	200	300	316	475	268	402	241	361
	19	252	377	222	334	191	287	304	456	257	386	231	346
	20	241	362	213	319	182	273	291	437	247	370	221	331
	21	231	347	203	305	173	260	279	418	236	354	211	316
	22	221	331	194	290	164	247	266	399	225	337	201	301
	23	210	315	184	276	156	234	254	380	214	321	190	286
	24	200	300	174	262	147	220	241	361	203	305	180	271
	25	190	285	165	248	138	208	229	343	193	289	171	256
	26	180	270	156	234	130	195	216	324	182	273	161	241
	27	170	255	147	220	122	183	204	306	172	258	152	227
	28	160	240	138	207	114	171	192	289	162	242	142	213
	29	151	226	129	194	106	159	181	271	152	228	133	200
	30	141	212	121	181	99.1	149	169	254	142	213	124	187
	32	124	186	106	159	87.1	131	149	223	125	187	109	164
	34	110	165	94.1	141	77.2	116	132	198	110	166	96.8	145
	36	98.2	147	83.9	126	68.8	103	118	177	98.5	148	86.4	130
	38	88.1	132	75.3	113	61.8	92.7	106	158	88.4	133	77.5	116
	40	79.5	119	68.0	102	55.8	83.7	95.3	143	79.8	120	70.0	105
Properties													
M_n/Ω_b $\phi_b M_n$	kip-ft	73.4	110	60.9	91.6	47.5	71.4	99.3	149	78.6	118	67.6	102
$P_e(L_c)^2/10^4$, kip-in.^{2}		4180		3570		2930		5010		4190		3680	
r_m, in.		3.43		3.45		3.47		3.24		3.28		3.30	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

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⑤

COMPOSITE
HSS9.625–
HSS8.625

Table IV-3B (continued)

Available Strength in Axial Compression, kips

Filled Round HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS9.625×				HSS8.625×							
		0.250		0.188		0.625		0.500		0.375		0.322	
t_{des}, in.		0.233		0.174		0.581		0.465		0.349		0.300	
Steel, lb/ft		25.1		19.0		53.5		43.4		33.1		28.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	314	472	279	419	442	663	384	576	326	489	301	451
	1	314	471	279	419	441	662	384	576	325	488	300	450
	2	313	469	278	417	440	659	382	573	324	486	299	449
	3	311	467	276	414	437	655	380	569	322	483	297	445
	4	308	463	274	410	432	649	376	564	319	478	294	441
	5	305	458	270	406	427	641	372	557	315	472	290	436
	6	301	452	267	400	421	631	366	549	310	465	286	429
	7	296	445	262	393	414	620	360	540	305	457	281	421
	8	291	437	257	386	405	608	353	529	299	448	275	413
	9	285	428	251	377	396	594	345	517	292	438	269	403
	10	279	418	245	368	386	579	336	504	285	427	262	393
	11	272	408	239	358	375	563	327	490	277	415	254	381
	12	264	397	232	347	364	545	317	475	268	402	246	369
	13	257	385	224	336	352	527	306	460	259	389	238	357
	14	248	373	216	325	339	508	296	443	250	375	229	344
	15	240	360	208	312	326	489	284	426	240	360	220	330
	16	231	347	200	300	312	469	273	409	230	346	211	316
	17	222	333	192	287	299	448	261	391	220	330	202	302
	18	213	319	183	275	285	427	249	373	210	315	192	288
	19	204	305	174	262	271	407	237	355	200	300	182	274
	20	194	291	166	249	257	386	225	337	190	284	173	259
	21	185	277	157	236	243	365	213	319	179	269	163	245
	22	176	263	149	223	229	344	201	301	169	254	154	231
	23	166	250	140	210	216	324	189	284	159	239	145	217
	24	157	236	132	198	204	306	178	267	149	224	135	203
	25	148	222	124	185	192	289	166	250	140	210	127	190
	26	139	209	116	173	181	272	155	233	130	196	118	177
	27	131	196	108	162	170	255	144	217	121	182	109	164
	28	122	183	100	150	159	239	134	202	113	169	102	153
	29	114	171	93.4	140	148	223	125	188	105	157	94.8	142
	30	107	160	87.3	131	138	208	117	176	98.1	147	88.6	133
	32	93.6	140	76.7	115	122	183	103	154	86.2	129	77.9	117
	34	82.9	124	68.0	102	108	162	91.1	137	76.4	115	69.0	103
	36	74.0	111	60.6	91.0	96.2	145	81.3	122	68.1	102	61.5	92.3
	38	66.4	99.6	54.4	81.6	86.3	130	72.9	109	61.1	91.7	55.2	82.8
	40	59.9	89.9	49.1	73.7	77.9	117	65.8	98.7	55.2	82.8	49.8	74.7
Properties													
M_n/Ω_b $\phi_b M_n$	kip-ft	56.1	84.3	43.8	65.8	93.1	140	78.0	117	61.9	93.0	54.6	82.1
$P_e(L_c)^2/10^4$, kip-in.^{2}		3150		2580		3930		3460		2900		2620	
r_m, in.		3.32		3.34		2.85		2.89		2.93		2.95	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

5

COMPOSITE
HSS8.625–
HSS7.500

Table IV-3B (continued)
Available Strength in Axial Compression, kips
Filled Round HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS8.625×				HSS7.625×				HSS7.500×			
		0.250		0.188		0.375		0.328		0.500		0.375	
t_{des}, in,		0.233		0.174		0.349		0.305		0.465		0.349	
Steel, lb/ft		22.4		17.0		29.1		25.6		37.4		28.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	265	398	234	351	273	410	253	380	317	476	267	400
	1	265	398	234	351	273	409	253	379	317	475	266	399
	2	264	396	233	349	271	407	251	377	315	473	265	397
	3	262	393	231	346	269	403	249	374	312	468	262	394
	4	259	389	228	342	266	398	246	369	308	463	259	389
	5	256	384	225	338	262	392	242	363	304	455	255	382
	6	252	378	221	332	257	385	238	357	298	446	250	375
	7	247	371	217	325	251	376	232	349	291	436	244	366
	8	242	363	212	317	244	367	226	340	283	425	238	357
	9	236	354	206	309	237	356	220	330	275	412	231	346
	10	229	344	200	300	230	345	213	319	266	398	223	335
	11	222	334	193	290	222	332	205	308	256	384	215	322
	12	215	323	187	280	213	319	197	296	246	368	206	309
	13	207	311	179	269	204	306	189	283	235	352	197	296
	14	199	299	172	258	195	292	180	270	224	336	188	282
	15	191	287	164	246	185	278	171	257	213	319	178	268
	16	183	274	156	235	175	263	162	244	201	302	169	253
	17	174	261	148	223	166	249	153	230	190	284	159	239
	18	165	248	141	211	156	234	144	217	178	267	150	224
	19	157	235	133	199	146	220	135	203	167	250	140	210
	20	148	222	125	187	137	205	127	190	156	233	131	196
	21	139	209	117	175	127	191	118	177	145	217	121	182
	22	131	196	109	164	118	178	109	164	134	201	112	169
	23	123	184	102	153	110	164	101	152	124	187	104	156
	24	114	172	94.5	142	101	151	93.1	140	115	173	95.2	143
	25	107	160	87.3	131	92.9	139	85.8	129	107	160	87.8	132
	26	98.6	148	80.7	121	85.9	129	79.3	119	98.6	148	81.1	122
	27	91.5	137	74.9	112	79.6	119	73.5	110	91.4	137	75.2	113
	28	85.1	128	69.6	104	74.0	111	68.4	103	85.0	128	70.0	105
	29	79.3	119	64.9	97.3	69.0	104	63.7	95.6	79.3	119	65.2	97.8
	30	74.1	111	60.6	91.0	64.5	96.7	59.6	89.3	74.1	111	60.9	91.4
	32	65.1	97.7	53.3	79.9	56.7	85.0	52.3	78.5	65.1	97.8	53.6	80.3
	34	57.7	86.5	47.2	70.8	50.2	75.3	46.4	69.6	57.7	86.7	47.4	71.2
	36	51.5	77.2	42.1	63.2	44.8	67.2	41.4	62.0	51.4	77.3	42.3	63.5
	38	46.2	69.3	37.8	56.7	40.2	60.3	37.1	55.7	46.2	69.4	38.0	57.0
	40	41.7	62.5	34.1	51.2	36.3	54.4	33.5	50.3	41.7	62.6	34.3	51.4
Properties													
M_n/Ω_b $\phi_b M_n$	kip-ft	44.3	66.5	34.6	52.0	47.3	71.0	42.3	63.6	57.3	86.2	45.6	68.5
$P_e(L_c)^2/10^4$, kip-in.^{2}		2190		1790		1910		1760		2150		1800	
r_m, in.		2.97		2.99		2.58		2.59		2.49		2.53	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

5

COMPOSITE
HSS7.500–
HSS7.000

Table IV-3B (continued)

Available Strength in Axial Compression, kips

Filled Round HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS7.500×						HSS7.000×					
		0.312		0.250		0.188		0.500		0.375		0.312	
t_{des}, in.		0.291		0.233		0.174		0.465		0.349		0.291	
Steel, lb/ft		24.0		19.4		14.7		34.7		26.6		22.3	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	241	361	215	322	187	281	288	433	242	363	218	327
	1	240	361	214	321	187	281	288	432	241	362	217	326
	2	239	359	213	320	186	279	286	429	240	360	216	324
	3	237	355	211	317	184	276	283	425	237	356	214	321
	4	234	351	208	312	182	272	279	419	234	351	211	316
	5	230	345	205	307	178	267	274	411	230	345	207	310
	6	226	339	201	301	174	262	268	402	225	337	202	303
	7	221	331	196	294	170	255	261	391	219	328	197	295
	8	215	322	190	286	165	247	253	379	212	318	191	287
	9	208	312	184	277	159	239	244	366	205	307	184	277
	10	201	302	178	267	153	230	235	352	197	296	177	266
	11	194	291	171	257	147	221	225	338	189	283	170	255
	12	186	279	164	246	140	211	215	322	180	270	162	243
	13	178	267	156	235	134	200	204	306	171	257	154	231
	14	169	254	149	223	127	190	193	289	162	243	146	219
	15	161	241	141	211	119	179	182	273	153	229	137	206
	16	152	228	133	199	112	168	171	256	143	215	129	193
	17	143	215	125	188	105	158	160	239	134	201	120	181
	18	135	202	117	176	97.9	147	148	223	125	187	112	168
	19	126	189	109	164	90.9	136	138	206	116	173	104	156
	20	117	176	102	152	84.1	126	128	192	107	160	95.8	144
	21	109	164	94.1	141	77.4	116	119	179	98.1	147	88.1	132
	22	101	151	86.8	130	70.9	106	110	165	89.6	134	80.5	121
	23	93.1	140	79.6	119	64.9	97.3	101	152	82.0	123	73.6	110
	24	85.5	128	73.1	110	59.6	89.4	93.1	140	75.3	113	67.6	101
	25	78.8	118	67.4	101	54.9	82.4	85.8	129	69.4	104	62.3	93.5
	26	72.8	109	62.3	93.5	50.8	76.2	79.4	119	64.2	96.3	57.6	86.4
	27	67.5	101	57.8	86.7	47.1	70.6	73.6	111	59.5	89.3	53.4	80.1
	28	62.8	94.2	53.7	80.6	43.8	65.7	68.4	103	55.3	83.0	49.7	74.5
	29	58.5	87.8	50.1	75.1	40.8	61.2	63.8	95.9	51.6	77.4	46.3	69.5
	30	54.7	82.1	46.8	70.2	38.1	57.2	59.6	89.6	48.2	72.3	43.3	64.9
	32	48.1	72.1	41.1	61.7	33.5	50.3	52.4	78.8	42.4	63.6	38.0	57.1
	34	42.6	63.9	36.4	54.7	29.7	44.5	46.4	69.8	37.5	56.3	33.7	50.5
	36	38.0	57.0	32.5	48.7	26.5	39.7	41.4	62.2	33.5	50.2	30.1	45.1
	38	34.1	51.1	29.2	43.8	23.8	35.7	37.2	55.8	30.0	45.1	27.0	40.5
	40	30.8	46.2	26.3	39.5	21.5	32.2						
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	39.3	59.1	32.7	49.2	25.6	38.5	49.2	74.0	39.2	58.9	33.8	50.8
$P_e(L_c)^2/10^4$, kip-in.^{2}		1620		1380		1130		1700		1420		1280	
r_m, in.		2.55		2.57		2.59		2.32		2.35		2.37	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_c = 2.00$	$\phi_c = 0.75$

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

COMPOSITE
HSS7.000–
HSS6.875

Table IV-3B (continued)

Available Strength in Axial Compression, kips

Filled Round HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape	HSS7.000×						HSS6.875×					
	0.250		0.188		0.125		0.500		0.375		0.312	
t_{des}, in.	0.233		0.174		0.116		0.465		0.349		0.291	
Steel, lb/ft	18.0		13.7		9.19		34.1		26.1		21.9	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r: 0	193	290	168	252	143	215	281	422	236	354	212	318
1	193	290	168	252	143	214	281	421	235	353	212	318
2	192	288	167	250	142	213	279	418	234	351	211	316
3	190	285	165	247	140	210	276	414	231	347	208	312
4	187	281	162	244	137	206	272	408	228	342	205	308
5	184	275	159	239	134	202	267	400	224	336	201	302
6	179	269	155	233	131	196	261	391	219	328	197	295
7	174	262	151	226	126	190	253	380	213	319	191	287
8	169	253	146	218	122	183	245	368	206	309	185	278
9	163	244	140	210	117	175	237	355	199	298	179	268
10	157	235	134	201	111	167	227	341	191	286	172	257
11	150	225	128	192	105	158	217	326	182	274	164	246
12	143	214	121	182	99.5	149	207	311	174	261	156	234
13	135	203	115	172	93.4	140	196	295	165	247	148	222
14	128	192	108	162	87.2	131	185	278	156	233	140	210
15	120	180	101	152	81.1	122	174	261	146	220	132	197
16	112	169	94.2	141	75.0	112	163	245	137	206	123	185
17	105	157	87.4	131	69.0	103	152	228	128	192	115	172
18	97.4	146	80.8	121	63.1	94.7	141	212	119	178	107	160
19	90.1	135	74.3	111	57.5	86.2	131	197	110	165	98.5	148
20	82.9	124	68.0	102	52.0	78.0	122	183	101	152	90.7	136
21	76.0	114	61.8	92.7	47.1	70.7	113	169	92.6	139	83.1	125
22	69.3	104	56.3	84.5	43.0	64.4	104	156	84.4	127	75.7	114
23	63.4	95.1	51.5	77.3	39.3	58.9	95.2	143	77.2	116	69.2	104
24	58.2	87.3	47.3	71.0	36.1	54.1	87.4	131	70.9	106	63.6	95.4
25	53.6	80.5	43.6	65.4	33.3	49.9	80.6	121	65.3	98.0	58.6	87.9
26	49.6	74.4	40.3	60.5	30.8	46.1	74.5	112	60.4	90.6	54.2	81.3
27	46.0	69.0	37.4	56.1	28.5	42.8	69.1	104	56.0	84.0	50.2	75.4
28	42.8	64.1	34.8	52.2	26.5	39.8	64.2	96.5	52.1	78.1	46.7	70.1
29	39.9	59.8	32.4	48.6	24.7	37.1	59.9	90.0	48.6	72.8	43.6	65.3
30	37.3	55.9	30.3	45.4	23.1	34.6	55.9	84.1	45.4	68.1	40.7	61.1
32	32.7	49.1	26.6	39.9	20.3	30.5	49.2	73.9	39.9	59.8	35.8	53.7
34	29.0	43.5	23.6	35.4	18.0	27.0	43.5	65.5	35.3	53.0	31.7	47.5
36	25.9	38.8	21.0	31.6	16.0	24.1	38.8	58.4	31.5	47.3	28.3	42.4
38	23.2	34.8	18.9	28.3	14.4	21.6			28.3	42.4	25.4	38.1
40			17.0	25.6	13.0	19.5						
Properties												
M_n/Ω_b | $\phi_b M_n$ | kip-ft	28.2	42.3	22.1	33.2	15.6	23.4	47.3	71.1	37.7	56.6	32.5	48.9
$P_e(L_c)^2/10^4$, kip-in.^{2}	1100		895		683		1600		1340		1200	
r_m, in.	2.39		2.41		2.43		2.27		2.31		2.33	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

⑤

COMPOSITE
HSS6.875–
HSS6.625

Table IV-3B (continued)
Available Strength in Axial Compression, kips
Filled Round HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS6.875×				HSS6.625×							
		0.250		0.188		0.500		0.432		0.375		0.312	
t_{des}, in,		0.233		0.174		0.465		0.402		0.349		0.291	
Steel, lb/ft		17.7		13.4		32.7		28.6		25.1		21.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	188	283	164	245	267	401	244	366	224	336	201	302
	1	188	282	163	245	267	400	243	365	223	335	201	301
	2	187	280	162	243	265	398	242	363	222	333	199	299
	3	185	277	160	240	262	393	239	359	219	329	197	296
	4	182	273	158	236	258	387	235	353	216	324	194	291
	5	178	268	154	232	253	379	230	346	211	317	190	285
	6	174	261	150	226	246	369	225	337	206	309	185	278
	7	169	254	146	219	239	359	218	327	200	300	180	270
	8	164	246	141	211	231	347	211	316	193	290	174	261
	9	158	237	135	203	222	333	203	304	186	279	167	251
	10	151	227	130	194	213	319	194	291	178	267	160	240
	11	145	217	123	185	203	304	185	278	170	255	153	229
	12	137	206	117	175	192	288	176	263	161	242	145	217
	13	130	195	110	165	182	272	166	249	152	228	137	205
	14	123	184	104	155	171	256	156	234	143	215	129	193
	15	115	173	96.8	145	160	240	146	219	134	201	120	180
	16	107	161	90.1	135	149	223	136	204	125	187	112	168
	17	100	150	83.4	125	138	207	126	189	116	174	104	156
	18	92.6	139	76.9	115	128	193	116	175	107	160	95.9	144
	19	85.4	128	70.5	106	119	179	107	161	98.3	147	88.1	132
	20	78.4	118	64.4	96.6	110	165	97.9	147	89.9	135	80.6	121
	21	71.5	107	58.4	87.6	101	152	89.6	135	81.7	123	73.2	110
	22	65.2	97.8	53.2	79.8	92.2	139	82.0	123	74.4	112	66.7	100
	23	59.6	89.5	48.7	73.0	84.4	127	75.1	113	68.1	102	61.0	91.5
	24	54.8	82.2	44.7	67.1	77.5	116	68.9	104	62.6	93.8	56.0	84.1
	25	50.5	75.7	41.2	61.8	71.4	107	63.5	95.5	57.6	86.5	51.6	77.5
	26	46.7	70.0	38.1	57.1	66.0	99.3	58.7	88.3	53.3	80.0	47.7	71.6
	27	43.3	64.9	35.3	53.0	61.2	92.0	54.5	81.9	49.4	74.1	44.3	66.4
	28	40.2	60.4	32.8	49.3	56.9	85.6	50.6	76.1	46.0	68.9	41.2	61.8
	29	37.5	56.3	30.6	45.9	53.1	79.8	47.2	71.0	42.8	64.3	38.4	57.6
	30	35.1	52.6	28.6	42.9	49.6	74.6	44.1	66.3	40.0	60.1	35.9	53.8
	32	30.8	46.2	25.1	37.7	43.6	65.5	38.8	58.3	35.2	52.8	31.5	47.3
	34	27.3	40.9	22.3	33.4	38.6	58.0	34.4	51.6	31.2	46.8	27.9	41.9
	36	24.3	36.5	19.9	29.8	34.4	51.8	30.6	46.1	27.8	41.7	24.9	37.4
	38	21.8	32.8	17.8	26.7								
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	27.1	40.7	21.2	31.9	43.5	65.5	38.9	58.4	34.7	52.2	30.0	45.1
$P_e(L_c)^2/10^4$, kip-in.^{2}		1040		846		1410		1290		1180		1060	
r_m, in.		2.35		2.37		2.18		2.20		2.22		2.24	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
HSS6.625–
HSS6.000

Table IV-3B (continued)

Available Strength in Axial Compression, kips

Filled Round HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS6.625×								HSS6.000×			
		0.280		0.250		0.188		0.125		0.500		0.375	
t_{des}, in.		0.260		0.233		0.174		0.116		0.465		0.349	
Steel, lb/ft		19.0		17.0		12.9		8.69		29.4		22.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	189	284	178	268	155	232	131	196	234	351	195	293
	1	189	283	178	267	154	231	130	196	233	350	194	292
	2	187	281	177	265	153	230	129	194	231	347	193	289
	3	185	278	175	262	151	227	128	191	228	342	190	285
	4	182	273	172	258	149	223	125	188	224	336	187	280
	5	179	268	168	252	145	218	122	183	218	327	182	273
	6	174	261	164	246	141	212	118	177	212	317	176	265
	7	169	253	159	239	137	205	114	171	204	306	170	255
	8	163	245	154	230	132	198	109	164	196	293	163	245
	9	157	236	148	221	126	190	104	157	186	280	156	233
	10	150	225	141	212	121	181	99.1	149	177	265	148	221
	11	143	215	134	202	114	172	93.5	140	167	250	139	209
	12	136	204	127	191	108	162	87.7	132	156	234	131	196
	13	128	192	120	180	102	152	81.8	123	146	219	122	183
	14	121	181	113	169	95.0	142	75.9	114	136	204	113	170
	15	113	169	105	158	88.3	133	70.0	105	126	190	104	156
	16	105	158	97.9	147	81.8	123	64.3	96.4	117	176	95.6	143
	17	97.3	146	90.6	136	75.3	113	58.6	88.0	108	162	87.2	131
	18	89.8	135	83.5	125	69.0	104	53.2	79.8	98.4	148	79.1	119
	19	82.5	124	76.5	115	63.0	94.5	47.9	71.9	89.7	135	71.2	107
	20	75.4	113	69.9	105	57.0	85.5	43.2	64.9	81.1	122	64.7	97.3
	21	68.5	103	63.3	95.0	51.7	77.6	39.2	58.8	73.6	111	58.7	88.2
	22	62.4	93.6	57.7	86.6	47.1	70.7	35.7	53.6	67.0	101	53.5	80.4
	23	57.1	85.7	52.8	79.2	43.1	64.7	32.7	49.0	61.3	92.2	48.9	73.5
	24	52.4	78.7	48.5	72.7	39.6	59.4	30.0	45.0	56.3	84.6	44.9	67.5
	25	48.3	72.5	44.7	67.0	36.5	54.7	27.7	41.5	51.9	78.0	41.4	62.3
	26	44.7	67.0	41.3	62.0	33.7	50.6	25.6	38.4	48.0	72.1	38.3	57.6
	27	41.4	62.2	38.3	57.5	31.3	46.9	23.7	35.6	44.5	66.9	35.5	53.4
	28	38.5	57.8	35.6	53.4	29.1	43.6	22.1	33.1	41.4	62.2	33.0	49.6
	29	35.9	53.9	33.2	49.8	27.1	40.7	20.6	30.8	38.6	58.0	30.8	46.3
	30	33.6	50.4	31.0	46.6	25.3	38.0	19.2	28.8	36.0	54.2	28.8	43.2
	32	29.5	44.3	27.3	40.9	22.3	33.4	16.9	25.3	31.7	47.6	25.3	38.0
	34	26.1	39.2	24.2	36.2	19.7	29.6	15.0	22.4				
	36	23.3	35.0	21.6	32.3	17.6	26.4	13.3	20.0				
	38					15.8	23.7	12.0	18.0				
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	27.3	41.1	25.0	37.6	19.6	29.4	13.9	20.9	34.9	52.5	27.9	42.0
$P_e(L_c)^2/10^4$, kip-in.^{2}		992		917		749		568		1010		844	
r_m, in.		2.25		2.26		2.28		2.30		1.96		2.00	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

⑤

COMPOSITE
HSS6.000–
HSS5.563

Table IV-3B (continued)

Available Strength in Axial Compression, kips

Filled Round HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS6.000×										HSS5.563×	
		0.312		0.280		0.250		0.188		0.125		0.500	
t_{des}, in,		0.291		0.260		0.233		0.174		0.116		0.465	
Steel, lb/ft		19.0		17.1		15.4		11.7		7.85		27.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	175	262	164	246	154	231	133	199	111	167	211	317
	1	174	261	163	245	154	231	132	199	111	166	211	316
	2	173	259	162	243	152	229	131	197	110	165	209	313
	3	170	256	160	240	150	226	129	194	108	162	205	308
	4	167	251	157	235	147	221	127	190	106	158	201	301
	5	163	245	153	229	144	216	123	185	102	154	195	292
	6	158	237	148	222	139	209	119	179	98.8	148	188	282
	7	153	229	143	214	134	202	115	172	94.7	142	180	270
	8	146	219	137	206	129	193	110	165	90.1	135	171	257
	9	139	209	131	196	123	184	104	157	85.2	128	162	243
	10	132	198	124	186	116	175	98.6	148	80.1	120	153	229
	11	125	187	117	175	110	165	92.6	139	74.7	112	143	216
	12	117	176	110	164	103	154	86.5	130	69.3	104	134	201
	13	109	164	102	153	96.0	144	80.3	120	63.8	95.7	125	187
	14	101	152	94.7	142	89.0	134	74.1	111	58.4	87.5	115	173
	15	93.4	140	87.3	131	82.1	123	68.0	102	53.0	79.6	106	159
	16	85.7	129	80.1	120	75.2	113	62.0	93.0	47.9	71.8	96.3	145
	17	78.1	117	73.0	109	68.6	103	56.2	84.3	42.9	64.4	87.3	131
	18	70.9	106	66.2	99.3	62.2	93.3	50.6	75.9	38.3	57.4	78.6	118
	19	63.8	95.7	59.5	89.3	55.9	83.9	45.4	68.1	34.4	51.5	70.6	106
	20	57.6	86.4	53.7	80.6	50.5	75.7	41.0	61.5	31.0	46.5	63.7	95.7
	21	52.2	78.3	48.7	73.1	45.8	68.7	37.2	55.8	28.1	42.2	57.8	86.8
	22	47.6	71.4	44.4	66.6	41.7	62.6	33.9	50.8	25.6	38.4	52.6	79.1
	23	43.5	65.3	40.6	61.0	38.2	57.2	31.0	46.5	23.4	35.2	48.2	72.4
	24	40.0	60.0	37.3	56.0	35.1	52.6	28.5	42.7	21.5	32.3	44.2	66.5
	25	36.9	55.3	34.4	51.6	32.3	48.5	26.2	39.3	19.8	29.8	40.8	61.3
	26	34.1	51.1	31.8	47.7	29.9	44.8	24.3	36.4	18.4	27.5	37.7	56.6
	27	31.6	47.4	29.5	44.2	27.7	41.5	22.5	33.7	17.0	25.5	34.9	52.5
	28	29.4	44.1	27.4	41.1	25.8	38.6	20.9	31.4	15.8	23.7	32.5	48.8
	29	27.4	41.1	25.6	38.3	24.0	36.0	19.5	29.2	14.8	22.1	30.3	45.5
	30	25.6	38.4	23.9	35.8	22.4	33.6	18.2	27.3	13.8	20.7	28.3	42.5
	32	22.5	33.7	21.0	31.5	19.7	29.6	16.0	24.0	12.1	18.2		
	34					17.5	26.2	14.2	21.3	10.7	16.1		

Properties														
M_n/Ω_b	$\phi_b M_n$	kip-ft	24.1	36.3	22.0	33.1	20.1	30.3	15.8	23.8	11.2	16.9	29.5	44.3
$P_e(L_c)^2/10^4$, kip-in.^{2}			756		706		663		538		407		777	
r_m, in.			2.02		2.03		2.04		2.06		2.08		1.81	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

⑤

COMPOSITE
HSS5.563–
HSS5.500

Table IV-3B (continued)
Available Strength in Axial Compression, kips
Filled Round HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS5.563×								HSS5.500×			
		0.375		0.258		0.188		0.134		0.500		0.375	
t_{des}, in,		0.349		0.240		0.174		0.124		0.465		0.349	
Steel, lb/ft		20.8		14.6		10.8		7.78		26.7		20.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	176	264	140	211	119	178	101	152	208	312	173	259
	1	175	263	140	210	118	177	101	152	208	311	172	259
	2	173	260	139	208	117	175	100	150	205	308	171	256
	3	171	256	136	205	115	172	98.1	147	202	303	168	252
	4	167	250	133	200	112	168	95.6	143	197	296	164	246
	5	162	243	129	194	109	163	92.5	139	191	287	159	239
	6	156	235	125	187	105	157	88.8	133	184	277	153	230
	7	150	225	120	180	100	151	84.7	127	177	265	147	221
	8	143	214	114	171	95.4	143	80.1	120	168	252	140	210
	9	135	203	108	162	90.0	135	75.3	113	159	239	132	198
	10	127	191	101	152	84.4	127	70.2	105	150	225	124	186
	11	119	178	94.7	142	78.6	118	65.0	97.4	141	211	116	174
	12	110	165	87.9	132	72.6	109	59.7	89.5	131	197	107	161
	13	102	152	81.0	122	66.7	100	54.4	81.6	122	183	98.7	148
	14	93.1	140	74.2	111	60.9	91.3	49.3	73.9	112	168	90.3	135
	15	84.7	127	67.5	101	55.2	82.7	44.3	66.4	103	154	82.0	123
	16	76.6	115	61.0	91.6	49.6	74.4	39.5	59.2	93.5	141	74.2	112
	17	69.5	105	54.8	82.2	44.3	66.4	35.0	52.4	84.6	127	67.5	101
	18	63.0	94.7	48.9	73.3	39.5	59.2	31.2	46.8	76.0	114	61.0	91.6
	19	56.6	85.1	43.9	65.8	35.4	53.2	28.0	42.0	68.2	102	54.7	82.2
	20	51.1	76.8	39.6	59.4	32.0	48.0	25.3	37.9	61.5	92.5	49.4	74.2
	21	46.3	69.6	35.9	53.9	29.0	43.5	22.9	34.4	55.8	83.9	44.8	67.3
	22	42.2	63.5	32.7	49.1	26.4	39.7	20.9	31.3	50.9	76.4	40.8	61.3
	23	38.6	58.1	29.9	44.9	24.2	36.3	19.1	28.7	46.5	69.9	37.3	56.1
	24	35.5	53.3	27.5	41.2	22.2	33.3	17.5	26.3	42.7	64.2	34.3	51.5
	25	32.7	49.1	25.3	38.0	20.5	30.7	16.2	24.3	39.4	59.2	31.6	47.5
	26	30.2	45.4	23.4	35.1	18.9	28.4	14.9	22.4	36.4	54.7	29.2	43.9
	27	28.0	42.1	21.7	32.6	17.6	26.3	13.9	20.8	33.8	50.7	27.1	40.7
	28	26.1	39.2	20.2	30.3	16.3	24.5	12.9	19.3	31.4	47.2	25.2	37.9
	29	24.3	36.5	18.8	28.2	15.2	22.8	12.0	18.0	29.3	44.0	23.5	35.3
	30	22.7	34.1	17.6	26.4	14.2	21.3	11.2	16.8			21.9	33.0
	32							9.87	14.8				
Properties													
M_n/Ω_b $\phi_b M_n$	kip-ft	23.6	35.5	17.5	26.3	13.4	20.2	10.1	15.2	28.7	43.2	23.0	34.6
$P_e(L_c)^2/10^4$, kip-in.^{2}		653		520		420		332		747		629	
r_m, in.		1.85		1.88		1.91		1.92		1.79		1.83	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

5

COMPOSITE
HSS5.500–
HSS5.000

Table IV-3B (continued)
Available Strength in Axial Compression, kips
Filled Round HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS5.500×		HSS5.000×									
		0.258		0.500		0.375		0.312		0.258		0.250	
t_{des}, in.		0.240		0.465		0.349		0.291		0.240		0.233	
Steel, lb/ft		14.5		24.1		18.5		15.6		13.1		12.7	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	138	207	183	275	152	228	135	203	121	181	119	178
	1	138	207	182	274	151	227	135	202	120	180	118	177
	2	136	205	180	270	149	224	133	200	119	178	117	175
	3	134	201	177	265	146	220	131	196	116	175	114	172
	4	131	197	172	258	142	214	127	190	113	170	111	167
	5	127	191	166	250	137	206	122	184	109	164	107	161
	6	123	184	159	240	131	197	117	176	104	157	103	154
	7	117	176	152	228	125	187	111	167	99.2	149	97.4	146
	8	112	168	144	216	117	176	105	157	93.4	140	91.8	138
	9	106	158	135	202	110	165	97.9	147	87.2	131	85.7	129
	10	99.1	149	125	189	102	152	90.8	136	80.8	121	79.4	119
	11	92.4	139	116	174	93.4	140	83.5	125	74.3	111	73.0	110
	12	85.6	128	106	160	85.2	128	76.1	114	67.8	102	66.6	99.9
	13	78.7	118	97.0	146	77.1	116	68.9	103	61.3	92.0	60.2	90.3
	14	72.0	108	87.7	132	69.9	105	61.8	92.8	55.0	82.5	54.0	81.1
	15	65.3	98.0	78.7	118	63.1	94.8	55.1	82.6	49.0	73.5	48.1	72.2
	16	58.9	88.4	70.0	105	56.5	84.9	48.7	73.2	43.2	64.8	42.4	63.6
	17	52.7	79.1	62.0	93.2	50.1	75.4	43.3	65.1	38.2	57.4	37.6	56.3
	18	47.0	70.5	55.3	83.1	44.7	67.2	38.6	58.1	34.1	51.2	33.5	50.3
	19	42.2	63.3	49.6	74.6	40.1	60.3	34.7	52.1	30.6	45.9	30.1	45.1
	20	38.1	57.1	44.8	67.3	36.2	54.5	31.3	47.0	27.6	41.5	27.1	40.7
	21	34.5	51.8	40.6	61.0	32.9	49.4	28.4	42.7	25.1	37.6	24.6	36.9
	22	31.5	47.2	37.0	55.6	29.9	45.0	25.9	38.9	22.8	34.3	22.4	33.6
	23	28.8	43.2	33.9	50.9	27.4	41.2	23.7	35.6	20.9	31.3	20.5	30.8
	24	26.4	39.7	31.1	46.7	25.2	37.8	21.7	32.7	19.2	28.8	18.8	28.3
	25	24.4	36.6	28.7	43.1	23.2	34.9	20.0	30.1	17.7	26.5	17.4	26.0
	26	22.5	33.8	26.5	39.8	21.4	32.2	18.5	27.8	16.4	24.5	16.1	24.1
	27	20.9	31.3			19.9	29.9	17.2	25.8	15.2	22.7	14.9	22.3
	28	19.4	29.1							14.1	21.1	13.8	20.8
	29	18.1	27.2										
	30	16.9	25.4										
Properties													
M_n/Ω_b / $\phi_b M_n$	kip-ft	17.1	25.6	23.2	34.8	18.6	28.0	16.2	24.3	13.8	20.8	13.5	20.3
$P_e(L_c)^2/10^4$, kip-in.^{2}		500		539		456		408		363		356	
r_m, in.		1.86		1.61		1.65		1.67		1.69		1.69	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

5

COMPOSITE
HSS5.000–
HSS4.500

Table IV-3B (continued)

Available Strength in Axial Compression, kips

Filled Round HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS5.000×				HSS4.500×							
		0.188		0.125		0.375		0.337		0.237		0.188	
t_{des}, in.		0.174		0.116		0.349		0.313		0.220		0.174	
Steel, lb/ft		9.67		6.51		16.5		15.0		10.8		8.67	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	101	152	83.3	125	132	197	123	184	98.8	148	86.4	130
	1	101	151	83.0	124	131	196	122	183	98.3	148	86.0	129
	2	99.4	149	81.9	123	129	194	120	181	96.9	145	84.7	127
	3	97.4	146	80.0	120	126	189	117	176	94.5	142	82.7	124
	4	94.6	142	77.5	116	122	182	113	170	91.3	137	79.8	120
	5	91.2	137	74.5	112	116	174	108	163	87.3	131	76.3	114
	6	87.1	131	70.8	106	110	165	103	154	82.7	124	72.2	108
	7	82.6	124	66.8	100	103	155	96.2	144	77.5	116	67.7	102
	8	77.7	116	62.4	93.6	95.6	143	89.3	134	72.0	108	62.8	94.2
	9	72.4	109	57.8	86.7	87.9	132	82.1	123	66.2	99.3	57.7	86.6
	10	66.9	100	53.1	79.6	80.1	120	74.7	112	60.2	90.3	52.5	78.7
	11	61.4	92.1	48.3	72.4	72.9	110	67.3	101	54.3	81.4	47.3	70.9
	12	55.8	83.8	43.5	65.3	65.7	98.8	60.0	90.2	48.4	72.7	42.1	63.2
	13	50.4	75.6	38.9	58.3	58.8	88.3	53.7	80.8	42.8	64.2	37.2	55.8
	14	45.1	67.6	34.4	51.6	52.1	78.2	47.7	71.7	37.4	56.1	32.5	48.7
	15	40.0	60.0	30.1	45.2	45.6	68.6	41.9	62.9	32.6	48.8	28.3	42.4
	16	35.2	52.8	26.5	39.7	40.1	60.3	36.8	55.3	28.6	42.9	24.8	37.3
	17	31.2	46.7	23.5	35.2	35.5	53.4	32.6	49.0	25.4	38.0	22.0	33.0
	18	27.8	41.7	20.9	31.4	31.7	47.6	29.1	43.7	22.6	33.9	19.6	29.4
	19	24.9	37.4	18.8	28.2	28.4	42.7	26.1	39.2	20.3	30.4	17.6	26.4
	20	22.5	33.8	16.9	25.4	25.7	38.6	23.5	35.4	18.3	27.5	15.9	23.9
	21	20.4	30.6	15.4	23.1	23.3	35.0	21.4	32.1	16.6	24.9	14.4	21.6
	22	18.6	27.9	14.0	21.0	21.2	31.9	19.5	29.3	15.1	22.7	13.1	19.7
	23	17.0	25.5	12.8	19.2	19.4	29.2	17.8	26.8	13.8	20.8	12.0	18.0
	24	15.6	23.4	11.8	17.6	17.8	26.8	16.4	24.6	12.7	19.1	11.0	16.6
	25	14.4	21.6	10.8	16.3					11.7	17.6	10.2	15.3
	26	13.3	20.0	10.0	15.0								
	27	12.4	18.5	9.30	13.9								
	28	11.5	17.2	8.64	13.0								
Properties													
M_n/Ω_b / $\phi_b M_n$	kip-ft	10.6	16.0	7.59	11.4	14.7	22.2	13.6	20.4	10.2	15.4	8.47	12.7
$P_e(L_c)^2/10^4$, kip-in.^{2}		296		223		318		298		241		209	
r_m, in.		1.71		1.73		1.47		1.48		1.52		1.53	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_c = 2.00$	$\phi_c = 0.75$

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

5

COMPOSITE
HSS4.500–
HSS4.000

Table IV-3B (continued)
Available Strength in Axial Compression, kips
Filled Round HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape	HSS4.500×		HSS4.000×									
	0.125		0.313		0.250		0.237		0.226		0.220	
t_{des}, in,	0.116		0.291		0.233		0.220		0.210		0.205	
Steel, lb/ft	5.85		12.3		10.0		9.53		9.12		8.89	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r												
0	70.8	106	99.8	150	86.8	130	83.7	126	81.4	122	80.2	120
1	70.4	106	99.1	149	86.2	129	83.2	125	80.9	121	79.7	120
2	69.3	104	97.3	146	84.6	127	81.6	122	79.4	119	78.2	117
3	67.4	101	94.3	141	82.0	123	79.1	119	76.9	115	75.8	114
4	64.9	97.3	90.2	135	78.5	118	75.7	114	73.6	110	72.5	109
5	61.8	92.7	85.3	128	74.2	111	71.6	107	69.6	104	68.6	103
6	58.2	87.3	79.6	119	69.3	104	66.8	100	65.0	97.5	64.0	96.0
7	54.2	81.4	73.3	110	63.8	95.8	61.6	92.4	59.9	89.9	59.0	88.5
8	50.0	75.0	66.7	100	58.1	87.2	56.1	84.1	54.5	81.8	53.7	80.6
9	45.6	68.4	59.9	89.9	52.3	78.4	50.4	75.6	49.0	73.5	48.3	72.5
10	41.1	61.7	53.6	80.5	46.4	69.6	44.8	67.2	43.5	65.3	42.9	64.3
11	36.7	55.0	47.7	71.6	40.7	61.0	39.3	58.9	38.2	57.3	37.6	56.4
12	32.4	48.6	41.9	63.0	35.2	52.8	34.0	51.0	33.1	49.6	32.6	48.9
13	28.3	42.4	36.5	54.8	30.1	45.3	29.1	43.6	28.2	42.4	27.8	41.8
14	24.4	36.6	31.5	47.3	26.0	39.1	25.1	37.6	24.4	36.5	24.0	36.0
15	21.3	31.9	27.4	41.2	22.6	34.0	21.8	32.7	21.2	31.8	20.9	31.4
16	18.7	28.0	24.1	36.2	19.9	29.9	19.2	28.8	18.6	28.0	18.4	27.6
17	16.6	24.8	21.3	32.1	17.6	26.5	17.0	25.5	16.5	24.8	16.3	24.4
18	14.8	22.1	19.0	28.6	15.7	23.6	15.2	22.7	14.7	22.1	14.5	21.8
19	13.3	19.9	17.1	25.7	14.1	21.2	13.6	20.4	13.2	19.8	13.0	19.5
20	12.0	17.9	15.4	23.2	12.7	19.1	12.3	18.4	11.9	17.9	11.8	17.6
21	10.8	16.3	14.0	21.0	11.6	17.4	11.1	16.7	10.8	16.2	10.7	16.0
22	9.88	14.8	12.7	19.1	10.5	15.8	10.1	15.2	9.86	14.8	9.72	14.6
23	9.04	13.6										
24	8.31	12.5										
25	7.65	11.5										

Properties												
M_n/Ω_b | $\phi_b M_n$ | kip-ft	6.04	9.09	9.85	14.8	8.28	12.4	7.91	11.9	7.62	11.5	7.48	11.2
$P_e(L_c)^2/10^4$, kip-in.^{2}	157		191		167		161		157		154	
r_m, in.	1.55		1.32		1.33		1.34		1.34		1.34	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r_m equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

⑤

COMPOSITE
HSS4.000

Table IV-3B (continued)
Available Strength in Axial Compression, kips
Filled Round HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS4.000×			
		0.188		0.125	
t_{des}, in,		0.174		0.116	
Steel, lb/ft		7.66		5.18	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	73.0	109	59.1	88.7
	1	72.5	109	58.7	88.1
	2	71.1	107	57.6	86.4
	3	68.9	103	55.7	83.5
	4	66.0	99.0	53.1	79.7
	5	62.3	93.5	50.0	75.0
	6	58.2	87.3	46.5	69.7
	7	53.6	80.4	42.6	63.9
	8	48.8	73.2	38.5	57.8
	9	43.8	65.8	34.4	51.6
	10	38.9	58.4	30.3	45.4
	11	34.1	51.1	26.3	39.5
	12	29.5	44.3	22.5	33.8
	13	25.2	37.8	19.2	28.8
	14	21.7	32.6	16.5	24.8
	15	18.9	28.4	14.4	21.6
	16	16.6	25.0	12.7	19.0
	17	14.7	22.1	11.2	16.8
	18	13.1	19.7	10.0	15.0
	19	11.8	17.7	8.98	13.5
	20	10.7	16.0	8.11	12.2
	21	9.66	14.5	7.35	11.0
	22	8.80	13.2	6.70	10.1

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	6.55	9.84	4.69	7.04
$P_e(L_c)^2/10^4$, kip-in.^{2}			140		107	
r_m, in.			1.35		1.37	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Heavy line indicates L_c/r_m equal to or greater than 200.

④

COMPOSITE
PIPE 12–PIPE 8

Table IV-4A
Available Strength in Axial Compression, kips
Filled Pipe

F_y = 35 ksi
f'_c = 4 ksi

Shape		Pipe 12				Pipe 10				Pipe 8			
		XS		STD		XS		STD		XXS		XS	
t_{des}, in,		0.465		0.349		0.465		0.340		0.816		0.465	
Steel, lb/ft		65.5		49.6		54.8		40.5		72.5		43.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	517	776	458	687	410	614	353	530	423	635	297	445
	1	517	776	458	687	409	614	353	530	423	634	296	445
	2	516	775	457	686	408	613	352	529	421	632	296	443
	3	515	773	456	684	407	611	351	527	419	629	294	441
	4	513	770	454	681	405	608	349	524	416	624	292	438
	5	511	767	452	678	403	604	347	521	412	618	289	434
	6	508	763	449	674	400	599	344	516	407	611	286	429
	7	505	758	446	669	396	594	341	511	402	602	282	423
	8	501	752	443	664	392	588	337	506	395	593	277	416
	9	497	746	439	658	387	581	333	500	388	583	273	409
	10	492	739	434	651	382	573	328	493	381	573	267	401
	11	487	731	429	644	377	565	323	485	373	561	261	392
	12	482	723	424	636	371	556	318	477	365	549	255	383
	13	476	714	418	628	364	547	312	468	357	536	248	373
	14	470	704	412	619	358	537	306	459	348	523	241	362
	15	463	694	406	609	351	526	300	449	338	508	234	351
	16	456	684	399	599	343	515	293	439	328	494	227	340
	17	448	673	392	589	335	503	286	429	318	478	219	328
	18	441	661	385	578	327	491	278	418	308	463	211	316
	19	433	649	378	566	319	479	271	406	297	447	203	304
	20	424	637	370	555	311	466	263	395	286	430	195	292
	21	416	624	362	542	302	453	255	383	275	414	187	280
	22	407	611	353	530	293	440	248	371	264	397	178	267
	23	398	597	345	517	284	426	239	359	253	380	170	255
	24	389	583	336	505	275	413	231	347	242	364	162	243
	25	380	569	328	491	266	399	223	335	231	347	154	231
	26	370	555	319	478	257	385	215	322	220	331	146	218
	27	361	541	310	465	247	371	207	310	209	314	138	207
	28	351	526	301	451	238	357	198	298	198	298	130	195
	29	341	512	292	438	229	343	190	285	188	283	122	184
	30	331	497	283	424	220	330	182	273	178	267	115	172
	32	312	467	264	397	202	303	166	250	158	237	101	152
	34	292	438	246	370	184	276	151	226	140	210	89.7	135
	36	272	408	229	343	167	251	136	204	124	187	80.0	120
	38	253	379	211	317	151	226	122	183	112	168	71.8	108
	40	234	351	194	291	136	204	110	165	101	152	64.8	97.5

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	141	213	111	168	97.6	147	75.5	113	92.0	138	59.7	89.7
$P_e(L_c)^2/10^4$, kip-in.^{2}			12600		10300		7140		5790		4770		3400	
r_m, in.			4.35		4.39		3.64		3.68		2.78		2.89	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
PIPE 8–PIPE 5

Table IV-4A (continued)
Available Strength in Axial Compression, kips
Filled Pipe

F_y = 35 ksi
f'_c = 4 ksi

Shape		Pipe 8		Pipe 6						Pipe 5			
		STD		XXS		XS		STD		XXS		XS	
t_{des}, in,		0.300		0.805		0.403		0.261		0.699		0.349	
Steel, lb/ft		28.6		53.2		28.6		19		38.6		20.8	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	234	350	308	463	188	282	147	220	224	337	136	203
	1	233	350	308	462	187	281	146	220	224	336	135	203
	2	233	349	306	460	186	280	146	218	222	334	134	201
	3	231	347	303	456	185	277	144	216	219	330	132	199
	4	230	344	300	451	182	274	142	214	216	324	130	195
	5	227	341	295	444	180	269	140	210	211	317	127	191
	6	224	337	290	436	176	264	137	206	205	309	124	186
	7	221	332	283	426	172	258	134	201	199	299	120	179
	8	218	326	276	415	168	251	131	196	192	288	115	173
	9	214	320	268	403	163	244	127	190	184	277	110	165
	10	209	314	260	391	157	236	122	183	176	264	105	158
	11	204	307	251	377	151	227	118	177	167	251	99.7	149
	12	199	299	241	362	145	218	113	169	158	237	94.0	141
	13	194	291	231	347	139	208	108	162	149	223	88.2	132
	14	188	282	221	332	132	199	103	154	139	209	82.4	124
	15	182	273	210	316	126	189	97.4	146	130	195	76.5	115
	16	176	264	199	299	119	179	92.0	138	120	181	70.7	106
	17	170	255	188	283	112	168	86.6	130	111	167	65.0	97.6
	18	163	245	177	267	105	158	81.3	122	102	153	59.8	89.8
	19	157	235	167	250	98.7	148	76.0	114	93.1	140	55.2	83.0
	20	150	225	156	234	92.1	138	70.8	106	84.5	127	50.7	76.3
	21	144	215	145	218	85.6	128	65.7	98.5	76.7	115	46.4	69.8
	22	137	205	135	203	79.3	119	60.7	91.1	69.9	105	42.3	63.6
	23	130	195	125	188	73.3	110	55.9	83.8	63.9	96.1	38.7	58.2
	24	124	185	115	173	68.3	103	51.3	77.0	58.7	88.2	35.5	53.4
	25	117	176	106	160	63.3	95.1	47.3	70.9	54.1	81.3	32.8	49.2
	26	111	166	98.2	148	58.5	88.0	43.7	65.6	50.0	75.2	30.3	45.5
	27	104	157	91.1	137	54.3	81.6	40.5	60.8	46.4	69.7	28.1	42.2
	28	98.2	147	84.7	127	50.5	75.8	37.7	56.5	43.1	64.8	26.1	39.2
	29	92.3	138	78.9	119	47.0	70.7	35.1	52.7	40.2	60.4	24.3	36.6
	30	86.2	129	73.8	111	44.0	66.1	32.8	49.3			22.7	34.2
	32	75.8	114	64.8	97.4	38.6	58.1	28.9	43.3				
	34	67.1	101	57.4	86.3	34.2	51.4	25.6	38.3				
	36	59.9	89.8			30.5	45.9	22.8	34.2				
	38	53.8	80.6										
	40	48.5	72.8										
Properties													
M_n/Ω_b $\phi_b M_n$	kip-ft	41.8	62.8	49.8	74.8	29.8	44.7	21.0	31.5	30.1	45.2	18.0	27.1
$P_e(L_c)^2/10^4$, kip-in.^{2}		2550		1910		1270		970		967		643	
r_m, in.		2.95		2.08		2.20		2.25		1.74		1.85	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
PIPE 5–PIPE 3½

Table IV-4A (continued)
Available Strength in Axial Compression, kips
Filled Pipe

F_y = 35 ksi
f'_c = 4 ksi

Shape		Pipe 5		Pipe 4						Pipe 3½			
		STD		XXS		XS		STD		XS		STD	
t_{des}, in.		0.241		0.628		0.315		0.221		0.296		0.211	
Steel, lb/ft		14.6		27.6		15		10.8		12.5		9.12	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	109	163	161	241	94.8	142	76.4	115	77.4	116	62.9	94.3
	1	108	163	160	240	94.4	142	76.1	114	77.0	116	62.6	93.9
	2	108	161	158	238	93.3	140	75.2	113	75.9	114	61.7	92.5
	3	106	159	155	233	91.6	137	73.8	111	74.0	111	60.2	90.2
	4	104	156	151	227	89.1	134	71.8	108	71.6	107	58.1	87.2
	5	102	153	146	219	86.1	129	69.3	104	68.5	103	55.6	83.4
	6	99.1	149	140	210	82.5	124	66.4	99.6	64.9	97.3	52.7	79.0
	7	95.8	144	133	200	78.4	118	63.1	94.7	60.9	91.3	49.4	74.1
	8	92.2	138	126	189	74.0	111	59.5	89.3	56.6	84.9	45.9	68.9
	9	88.3	132	118	177	69.3	104	55.7	83.6	52.1	78.1	42.2	63.4
	10	84.1	126	110	165	64.4	96.6	51.8	77.6	47.4	71.1	38.5	57.7
	11	79.7	120	101	152	59.3	89.0	47.7	71.5	42.8	64.3	34.7	52.1
	12	75.1	113	92.7	139	54.3	81.4	43.6	65.4	38.7	58.2	31.0	46.5
	13	70.4	106	84.3	127	49.3	73.9	39.6	59.3	34.8	52.3	27.4	41.1
	14	65.7	98.6	76.0	114	44.9	67.4	35.6	53.4	31.0	46.6	24.0	36.0
	15	61.0	91.5	68.1	102	40.7	61.2	31.8	47.7	27.3	41.0	20.9	31.3
	16	56.3	84.5	60.3	90.7	36.7	55.1	28.1	42.2	24.0	36.1	18.4	27.5
	17	51.8	77.7	53.5	80.3	32.8	49.2	24.9	37.4	21.3	32.0	16.3	24.4
	18	47.3	71.0	47.7	71.7	29.2	43.9	22.2	33.3	19.0	28.5	14.5	21.8
	19	43.0	64.6	42.8	64.3	26.2	39.4	19.9	29.9	17.0	25.6	13.0	19.5
	20	38.9	58.3	38.6	58.0	23.7	35.6	18.0	27.0	15.4	23.1	11.7	17.6
	21	35.3	52.9	35.0	52.6	21.5	32.3	16.3	24.5	13.9	20.9	10.7	16.0
	22	32.1	48.2	31.9	48.0	19.6	29.4	14.9	22.3			9.71	14.6
	23	29.4	44.1	29.2	43.9	17.9	26.9	13.6	20.4				
	24	27.0	40.5			16.4	24.7	12.5	18.8				
	25	24.9	37.3					11.5	17.3				
	26	23.0	34.5										
	27	21.3	32.0										
	28	19.8	29.7										
	29	18.5	27.7										
	30	17.3	25.9										

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	13.4	20.1	17.1	25.7	10.4	15.6	7.85	11.8	7.62	11.4	5.84	8.78
$P_e(L_c)^2/10^4$, kip-in.^{2}			511		438		295		236		191		154	
r_m, in.			1.88		1.39		1.48		1.51		1.31		1.34	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
PIPE 3

Table IV-4A (continued)
Available Strength in Axial Compression, kips
Filled Pipe

F_y = 35 ksi
f'_c = 4 ksi

Shape		Pipe 3					
		XXS		XS		STD	
t_{des}, in.		0.559		0.280		0.201	
Steel, lb/ft		18.6		10.3		7.58	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	108	163	62.4	93.6	50.6	75.9
	1	108	162	62.0	93.0	50.3	75.4
	2	106	159	60.8	91.3	49.3	73.9
	3	102	154	58.9	88.4	47.8	71.6
	4	97.6	147	56.3	84.5	45.7	68.5
	5	92.0	138	53.2	79.8	43.1	64.7
	6	85.6	129	49.6	74.3	40.2	60.3
	7	78.6	118	45.6	68.4	37.0	55.5
	8	71.2	107	41.4	62.1	33.6	50.4
	9	63.7	95.7	37.5	56.3	30.2	45.3
	10	56.2	84.5	33.6	50.6	26.7	40.1
	11	49.0	73.6	29.9	44.9	23.4	35.1
	12	42.1	63.3	26.2	39.4	20.2	30.3
	13	35.9	53.9	22.7	34.1	17.5	26.2
	14	30.9	46.5	19.6	29.4	15.1	22.7
	15	26.9	40.5	17.1	25.6	13.1	19.8
	16	23.7	35.6	15.0	22.5	11.6	17.4
	17	21.0	31.5	13.3	20.0	10.2	15.4
	18			11.8	17.8	9.13	13.7
	19			10.6	16.0	8.19	12.3
Properties							
M_n/Ω_b | $\phi_b M_n$	kip-ft	8.74	13.1	5.42	8.14	4.19	6.29
$P_e(L_c)^2/10^4$, kip-in.^{2}		171		117		95.6	
r_m, in.		1.06		1.14		1.17	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
PIPE 12–PIPE 8

Table IV-4B
Available Strength in Axial Compression, kips
Filled Pipe

F_y = 35 ksi
f'_c = 5 ksi

Shape		Pipe 12				Pipe 10				Pipe 8			
		XS		STD		XS		STD		XXS		XS	
t_{des}, in,		0.465		0.349		0.465		0.340		0.816		0.465	
Steel, lb/ft		65.5		49.6		54.8		40.5		72.5		43.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	570	855	513	769	446	669	392	587	441	662	319	478
	1	570	855	513	769	446	668	391	587	441	661	319	478
	2	569	854	512	767	445	667	390	585	439	659	317	476
	3	568	851	510	765	443	665	389	583	437	656	316	474
	4	565	848	508	762	441	661	387	580	434	651	313	470
	5	563	844	505	758	438	657	384	576	429	644	310	465
	6	559	839	502	753	434	651	381	571	424	636	306	460
	7	556	833	498	748	430	645	377	565	418	627	302	453
	8	551	827	494	741	425	638	372	558	411	617	297	446
	9	546	819	489	734	420	630	367	551	404	605	291	437
	10	541	811	484	726	414	622	362	543	395	593	285	428
	11	535	802	478	717	408	612	356	534	386	579	279	418
	12	528	792	472	707	401	602	349	524	376	565	272	408
	13	521	782	465	697	394	591	342	514	366	549	264	396
	14	514	771	458	686	386	579	335	503	355	533	257	385
	15	506	759	450	675	378	567	328	491	344	516	248	373
	16	498	747	442	663	369	554	320	480	333	499	240	360
	17	489	734	433	650	361	541	311	467	321	481	231	347
	18	480	720	425	637	351	527	303	454	309	463	223	334
	19	471	706	416	623	342	513	294	441	297	447	214	320
	20	461	692	406	609	332	498	285	428	286	430	205	307
	21	451	677	397	595	322	484	276	414	275	414	195	293
	22	441	661	387	580	312	469	267	400	264	397	186	279
	23	430	646	377	565	302	453	258	386	253	380	177	266
	24	420	630	367	550	292	438	248	372	242	364	168	252
	25	409	614	356	535	282	422	239	358	231	347	159	239
	26	398	597	346	519	271	407	229	344	220	331	151	226
	27	387	580	335	503	261	391	220	330	209	314	142	213
	28	376	564	325	487	251	376	210	316	198	298	133	200
	29	365	547	314	471	240	360	201	302	188	283	125	188
	30	353	530	304	456	230	345	192	288	178	267	117	176
	32	331	496	283	424	210	315	174	261	158	237	103	154
	34	308	463	262	393	191	286	157	235	140	210	912	137
	36	286	429	241	362	172	258	140	210	124	187	81.3	122
	38	265	397	221	332	154	231	126	189	112	168	73.0	109
	40	244	365	202	303	139	209	113	170	101	152	65.9	98.8

Properties

			Pipe 12 XS		Pipe 12 STD		Pipe 10 XS		Pipe 10 STD		Pipe 8 XXS		Pipe 8 XS	
M_n/Ω_b	$\phi_b M_n$	kip-ft	144	217	114	171	99.4	149	77	116	92.9	140	60.7	91.2
$P_e(L_c)^2/10^4$, kip-in.^{2}			12900		10600		7310		5960		4820		3460	
r_m, in.			4.35		4.39		3.64		3.68		2.78		2.89	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Note: Dashed line indicates the L_c beyond which the bare steel strength controls.

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COMPOSITE
PIPE 8–PIPE 5

Table IV-4B (continued)
Available Strength in Axial Compression, kips
Filled Pipe

F_y = 35 ksi
f'_c = 5 ksi

Shape		Pipe 8		Pipe 6						Pipe 5			
		STD		XXS		XS		STD		XXS		XS	
t_{des}, in,		0.300		0.805		0.403		0.261		0.699		0.349	
Steel, lb/ft		28.6		53.2		28.6		19		38.6		20.8	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	258	386	308	463	200	301	161	241	224	337	144	216
	1	257	386	308	462	200	300	160	240	224	336	144	216
	2	256	385	306	460	199	298	159	239	222	334	143	214
	3	255	382	303	456	197	296	158	237	219	330	141	211
	4	253	379	300	451	195	292	156	234	216	324	138	207
	5	250	375	295	444	191	287	153	230	211	317	135	203
	6	247	370	290	436	187	281	150	225	205	309	131	197
	7	243	365	283	426	183	274	146	219	199	299	127	190
	8	239	358	276	415	178	267	142	213	192	288	122	183
	9	234	351	268	403	172	258	137	206	184	277	116	174
	10	229	343	260	391	166	249	132	198	176	264	111	166
	11	223	335	251	377	160	240	127	190	167	251	105	157
	12	217	326	241	362	153	230	121	182	158	237	98.4	148
	13	211	316	231	347	146	219	116	173	149	223	92.0	138
	14	204	306	221	332	139	208	110	165	139	209	85.6	128
	15	197	296	210	316	132	197	104	156	130	195	79.3	119
	16	190	285	199	299	124	186	97.6	146	120	181	73.0	109
	17	183	274	188	283	117	175	91.6	137	111	167	66.8	100
	18	175	263	177	267	109	164	85.5	128	102	153	60.9	91.3
	19	168	252	167	250	102	153	79.6	119	93.1	140	55.2	83.0
	20	160	241	156	234	94.9	142	73.8	111	84.5	127	50.7	76.3
	21	153	229	145	218	87.9	132	68.1	102	76.7	115	46.4	69.8
	22	145	218	135	203	81.1	122	62.7	94.0	69.9	105	42.3	63.6
	23	138	206	125	188	74.4	112	57.3	86.0	63.9	96.1	38.7	58.2
	24	130	195	115	173	68.3	103	52.6	78.9	58.7	88.2	35.5	53.4
	25	123	184	106	160	63.3	95.1	48.5	72.7	54.1	81.3	32.8	49.2
	26	116	173	98.2	148	58.5	88.0	44.8	67.3	50.0	75.2	30.3	45.5
	27	109	163	91.1	137	54.3	81.6	41.6	62.4	46.4	69.7	28.1	42.2
	28	102	153	84.7	127	50.5	75.8	38.7	58.0	43.1	64.8	26.1	39.2
	29	94.9	142	78.9	119	47.0	70.7	36.0	54.1	40.2	60.4	24.3	36.6
	30	88.6	133	73.8	111	44.0	66.1	33.7	50.5			22.7	34.2
	32	77.9	117	64.8	97.4	38.6	58.1	29.6	44.4				
	34	69.0	104	57.4	86.3	34.2	51.4	26.2	39.3				
	36	61.6	92.3			30.5	45.9	23.4	35.1				
	38	55.2	82.9										
	40	49.9	74.8										
Properties													
M_n/Ω_b / $\phi_b M_n$	kip-ft	42.6	64.1	50.2	75.4	30.2	45.4	21.4	32.2	30.3	45.5	18.3	27.5
$P_e(L_c)^2/10^4$, kip-in.^{2}		2620		1930		1290		995		973		653	
r_m, in.		2.95		2.08		2.20		2.25		1.74		1.85	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

⑤

COMPOSITE
PIPE 5–PIPE 3½

Table IV-4B (continued)
Available Strength in Axial Compression, kips
Filled Pipe

F_y = 35 ksi
f'_c = 5 ksi

Shape		Pipe 5		Pipe 4						Pipe 3½			
		STD		XXS		XS		STD		XS		STD	
t_{des}, in,		0.241		0.628		0.315		0.221		0.296		0.211	
Steel, lb/ft		14.6		27.6		15		10.8		12.5		9.12	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	118	177	161	241	100	151	82.5	124	81.7	123	67.7	101
	1	118	177	160	240	100	150	82.2	123	81.3	122	67.3	101
	2	117	176	158	238	98.8	148	81.2	122	80.1	120	66.3	99.4
	3	115	173	155	233	96.8	145	79.5	119	78.1	117	64.6	96.9
	4	113	170	151	227	94.1	141	77.3	116	75.3	113	62.3	93.4
	5	111	166	146	219	90.7	136	74.5	112	71.9	108	59.4	89.1
	6	107	161	140	210	86.8	130	71.2	107	68.0	102	56.1	84.2
	7	104	155	133	200	82.3	124	67.4	101	63.6	95.5	52.5	78.7
	8	99.4	149	126	189	77.5	116	63.4	95.1	58.9	88.4	48.5	72.8
	9	94.8	142	118	177	72.3	109	59.1	88.7	54.0	81.1	44.5	66.7
	10	90.1	135	110	165	67.0	100	54.7	82.0	49.1	73.6	40.3	60.4
	11	85.0	128	101	152	61.5	92.3	50.1	75.2	44.1	66.1	36.1	54.2
	12	79.9	120	92.7	139	56.1	84.1	45.6	68.4	39.2	58.8	32.1	48.1
	13	74.6	112	84.3	127	50.7	76.0	41.1	61.7	34.8	52.3	28.2	42.2
	14	69.3	104	76.0	114	45.4	68.2	36.8	55.2	31.0	46.6	24.4	36.7
	15	64.0	96.0	68.1	102	40.7	61.2	32.6	49.0	27.3	41.0	21.3	31.9
	16	58.8	88.2	60.3	90.7	36.7	55.1	28.7	43.1	24.0	36.1	18.7	28.1
	17	53.8	80.6	53.5	80.3	32.8	49.2	25.4	38.1	21.3	32.0	16.6	24.9
	18	48.9	73.3	47.7	71.7	29.2	43.9	22.7	34.0	19.0	28.5	14.8	22.2
	19	44.1	66.1	42.8	64.3	26.2	39.4	20.4	30.5	17.0	25.6	13.3	19.9
	20	39.8	59.7	38.6	58.0	23.7	35.6	18.4	27.6	15.4	23.1	12.0	18.0
	21	36.1	54.1	35.0	52.6	21.5	32.3	16.7	25.0	13.9	20.9	10.9	16.3
	22	32.9	49.3	31.9	48.0	19.6	29.4	15.2	22.8			9.89	14.8
	23	30.1	45.1	29.2	43.9	17.9	26.9	13.9	20.8				
	24	27.6	41.4			16.4	24.7	12.8	19.1				
	25	25.5	38.2					11.8	17.6				
	26	23.5	35.3										
	27	21.8	32.7										
	28	20.3	30.4										
	29	18.9	28.4										
	30	17.7	26.5										

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	13.6	20.5	17.2	25.8	10.5	15.8	7.99	12.0	7.72	11.6	5.94	8.93
$P_e(L_c)^2/10^4$, kip-in.^{2}			522		440		299		241		193		157	
r_m, in.			1.88		1.39		1.48		1.51		1.31		1.34	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r equal to or greater than 200.

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⑤

COMPOSITE
PIPE 3

Table IV-4B (continued)
Available Strength in Axial Compression, kips
Filled Pipe

F_y = 35 ksi
f'_c = 5 ksi

Shape		Pipe 3					
		XXS		XS		STD	
t_{des}, in,		0.559		0.280		0.201	
Steel, lb/ft		18.6		10.3		7.58	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r	0	108	163	65.7	98.5	54.2	81.2
	1	108	162	65.2	97.8	53.8	80.7
	2	106	159	63.9	95.9	52.7	79.1
	3	102	154	61.8	92.7	51.0	76.5
	4	97.6	147	59.0	88.5	48.6	73.0
	5	92.0	138	55.6	83.4	45.8	68.7
	6	85.6	129	51.6	77.5	42.5	63.8
	7	78.6	118	47.3	71.0	39.0	58.5
	8	71.2	107	42.8	64.3	35.2	52.9
	9	63.7	95.7	38.2	57.4	31.4	47.2
	10	56.2	84.5	33.7	50.6	27.7	41.5
	11	49.0	73.6	29.9	44.9	24.0	36.1
	12	42.1	63.3	26.2	39.4	20.6	30.8
	13	35.9	53.9	22.7	34.1	17.5	26.3
	14	30.9	46.5	19.6	29.4	15.1	22.7
	15	26.9	40.5	17.1	25.6	13.2	19.8
	16	23.7	35.6	15.0	22.5	11.6	17.4
	17	21.0	31.5	13.3	20.0	10.2	15.4
	18			11.8	17.8	9.14	13.7
	19			10.6	16.0	8.20	12.3

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	8.79	13.2	5.48	8.24	4.25	6.39
$P_e(L_c)^2/10^4$, kip-in.^{2}			171		119		97.3	
r_m, in.			1.06		1.14		1.17	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 2.00	ϕ_c = 0.75

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Dashed line indicates the L_c beyond which the bare steel strength controls.

W44

Table IV-5
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W44×											
		335^c				290^c				262^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.345	0.229	0.220	0.146	0.416	0.277	0.253	0.168	0.473	0.315	0.281	0.187
	11	0.377	0.251	0.220	0.146	0.455	0.303	0.253	0.168	0.518	0.345	0.281	0.187
	12	0.384	0.256	0.220	0.146	0.463	0.308	0.253	0.168	0.527	0.351	0.281	0.187
	13	0.392	0.261	0.222	0.148	0.472	0.314	0.255	0.170	0.537	0.357	0.284	0.189
	14	0.402	0.267	0.225	0.150	0.482	0.320	0.259	0.173	0.548	0.365	0.289	0.192
	15	0.412	0.274	0.229	0.152	0.492	0.327	0.264	0.175	0.560	0.373	0.294	0.196
	16	0.423	0.281	0.232	0.155	0.504	0.335	0.268	0.178	0.574	0.382	0.299	0.199
	17	0.435	0.290	0.236	0.157	0.516	0.343	0.273	0.181	0.588	0.391	0.304	0.203
	18	0.449	0.299	0.240	0.160	0.530	0.353	0.277	0.184	0.604	0.402	0.310	0.206
	19	0.463	0.308	0.244	0.162	0.545	0.362	0.282	0.188	0.621	0.413	0.316	0.210
	20	0.479	0.319	0.248	0.165	0.561	0.373	0.287	0.191	0.640	0.426	0.322	0.214
	22	0.515	0.343	0.256	0.171	0.597	0.397	0.298	0.198	0.681	0.453	0.335	0.223
	24	0.558	0.371	0.266	0.177	0.643	0.428	0.309	0.206	0.730	0.486	0.348	0.232
	26	0.608	0.405	0.275	0.183	0.702	0.467	0.321	0.214	0.787	0.524	0.363	0.242
	28	0.668	0.444	0.286	0.190	0.770	0.512	0.335	0.223	0.859	0.571	0.379	0.252
	30	0.738	0.491	0.297	0.198	0.851	0.567	0.349	0.232	0.950	0.632	0.397	0.264
	32	0.822	0.547	0.310	0.206	0.948	0.631	0.365	0.243	1.06	0.705	0.417	0.277
	34	0.923	0.614	0.323	0.215	1.06	0.708	0.382	0.254	1.19	0.793	0.438	0.292
	36	1.03	0.689	0.338	0.225	1.19	0.794	0.401	0.267	1.34	0.889	0.465	0.310
	38	1.15	0.767	0.354	0.235	1.33	0.885	0.429	0.286	1.49	0.990	0.507	0.337
	40	1.28	0.850	0.377	0.251	1.47	0.980	0.464	0.309	1.65	1.10	0.549	0.365
	42	1.41	0.937	0.404	0.269	1.62	1.08	0.499	0.332	1.82	1.21	0.592	0.394
	44	1.55	1.03	0.431	0.287	1.78	1.19	0.534	0.355	2.00	1.33	0.635	0.423
	46	1.69	1.12	0.459	0.305	1.95	1.30	0.570	0.379	2.18	1.45	0.679	0.452
	48	1.84	1.22	0.486	0.323	2.12	1.41	0.605	0.403	2.37	1.58	0.722	0.481
	50	2.00	1.33	0.514	0.342	2.30	1.53	0.641	0.426	2.58	1.71	0.766	0.510

Other Constants and Properties

	335^c ASD	335^c LRFD	290^c ASD	290^c LRFD	262^c ASD	262^c LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	1.51	1.00	1.74	1.16	1.96	1.30
$t_y \times 10^3$, (kips)$^{-1}$	0.339	0.226	0.391	0.260	0.433	0.288
$t_r \times 10^3$, (kips)$^{-1}$	0.417	0.278	0.480	0.320	0.531	0.354
r_x/r_y	5.10		5.10		5.10	
r_y, in.	3.49		3.49		3.47	

^{c} Shape is slender for compression for F_y = 50 ksi.

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

W44–W40

F_y = 50 ksi

Shape		W44×				W40×							
		$230^{c,v}$				655^h				593^h			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.555	0.369	0.324	0.215	0.173	0.115	0.116	0.0770	0.192	0.128	0.129	0.0859
	11	0.609	0.405	0.324	0.215	0.189	0.125	0.116	0.0770	0.210	0.139	0.129	0.0859
	12	0.620	0.413	0.324	0.215	0.192	0.127	0.116	0.0770	0.213	0.142	0.129	0.0859
	13	0.632	0.421	0.329	0.219	0.195	0.130	0.116	0.0770	0.217	0.144	0.129	0.0859
	14	0.646	0.430	0.335	0.223	0.199	0.132	0.116	0.0772	0.221	0.147	0.130	0.0863
	15	0.660	0.439	0.341	0.227	0.203	0.135	0.117	0.0777	0.226	0.150	0.131	0.0870
	16	0.676	0.450	0.347	0.231	0.207	0.138	0.118	0.0783	0.231	0.154	0.132	0.0877
	17	0.694	0.461	0.354	0.235	0.212	0.141	0.119	0.0789	0.237	0.158	0.133	0.0884
	18	0.712	0.474	0.360	0.240	0.218	0.145	0.119	0.0795	0.243	0.162	0.134	0.0892
	19	0.733	0.488	0.367	0.244	0.223	0.149	0.120	0.0801	0.250	0.166	0.135	0.0899
	20	0.755	0.503	0.375	0.249	0.230	0.153	0.121	0.0807	0.257	0.171	0.136	0.0907
	22	0.806	0.536	0.390	0.260	0.244	0.162	0.123	0.0820	0.273	0.182	0.139	0.0923
	24	0.865	0.575	0.407	0.271	0.260	0.173	0.125	0.0833	0.292	0.194	0.141	0.0939
	26	0.934	0.621	0.425	0.283	0.279	0.186	0.127	0.0846	0.314	0.209	0.144	0.0956
	28	1.01	0.675	0.446	0.296	0.301	0.200	0.129	0.0860	0.340	0.226	0.146	0.0973
	30	1.11	0.738	0.468	0.311	0.327	0.217	0.131	0.0874	0.370	0.246	0.149	0.0991
	32	1.23	0.820	0.492	0.327	0.357	0.237	0.134	0.0889	0.405	0.269	0.152	0.101
	34	1.39	0.924	0.519	0.346	0.392	0.261	0.136	0.0904	0.446	0.297	0.155	0.103
	36	1.56	1.04	0.568	0.378	0.432	0.288	0.138	0.0920	0.494	0.329	0.158	0.105
	38	1.73	1.15	0.621	0.413	0.481	0.320	0.141	0.0936	0.551	0.366	0.161	0.107
	40	1.92	1.28	0.674	0.449	0.533	0.355	0.143	0.0953	0.610	0.406	0.164	0.109
	42	2.12	1.41	0.729	0.485	0.588	0.391	0.146	0.0971	0.673	0.448	0.168	0.112
	44	2.33	1.55	0.784	0.522	0.645	0.429	0.149	0.0989	0.738	0.491	0.171	0.114
	46	2.54	1.69	0.840	0.559	0.705	0.469	0.152	0.101	0.807	0.537	0.175	0.116
	48	2.77	1.84	0.897	0.597	0.768	0.511	0.154	0.103	0.879	0.585	0.179	0.119
	50	3.00	2.00	0.954	0.634	0.833	0.554	0.158	0.105	0.953	0.634	0.183	0.122
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		2.27		1.51		0.657		0.437		0.741		0.493	
$t_y \times 10^3$, (kips)$^{-1}$		0.493		0.328		0.173		0.115		0.192		0.128	
$t_r \times 10^3$, (kips)$^{-1}$		0.605		0.403		0.213		0.142		0.236		0.157	
r_x/r_y		5.10				4.43				4.47			
r_y, in.		3.43				3.86				3.80			

[c] Shape is slender for compression for F_y = 50 ksi.

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(1) with F_y = 50 ksi; therefore, ϕ_v = 0.90 and Ω_v = 1.67.

W40

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W40×											
		503^h				431^h				397^h			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.226	0.150	0.154	0.102	0.263	0.175	0.182	0.121	0.285	0.190	0.198	0.132
	11	0.247	0.165	0.154	0.102	0.289	0.193	0.182	0.121	0.314	0.209	0.198	0.132
	12	0.252	0.168	0.154	0.102	0.295	0.196	0.182	0.121	0.320	0.213	0.198	0.132
	13	0.257	0.171	0.154	0.102	0.301	0.200	0.182	0.121	0.327	0.217	0.198	0.132
	14	0.262	0.174	0.155	0.103	0.307	0.204	0.184	0.122	0.334	0.222	0.201	0.133
	15	0.268	0.178	0.156	0.104	0.314	0.209	0.186	0.124	0.341	0.227	0.203	0.135
	16	0.274	0.182	0.158	0.105	0.322	0.214	0.188	0.125	0.350	0.233	0.205	0.137
	17	0.281	0.187	0.159	0.106	0.330	0.220	0.190	0.127	0.359	0.239	0.208	0.138
	18	0.289	0.192	0.161	0.107	0.340	0.226	0.193	0.128	0.369	0.246	0.211	0.140
	19	0.297	0.198	0.163	0.108	0.350	0.233	0.195	0.130	0.380	0.253	0.213	0.142
	20	0.306	0.204	0.164	0.109	0.361	0.240	0.197	0.131	0.392	0.261	0.216	0.144
	22	0.326	0.217	0.168	0.112	0.386	0.257	0.202	0.134	0.419	0.279	0.221	0.147
	24	0.350	0.233	0.171	0.114	0.415	0.276	0.207	0.138	0.451	0.300	0.227	0.151
	26	0.377	0.251	0.175	0.117	0.449	0.299	0.212	0.141	0.488	0.325	0.234	0.155
	28	0.410	0.273	0.179	0.119	0.489	0.325	0.218	0.145	0.532	0.354	0.240	0.160
	30	0.448	0.298	0.183	0.122	0.536	0.356	0.224	0.149	0.584	0.388	0.247	0.164
	32	0.492	0.327	0.187	0.125	0.591	0.393	0.230	0.153	0.644	0.429	0.255	0.169
	34	0.544	0.362	0.192	0.128	0.656	0.436	0.236	0.157	0.715	0.476	0.262	0.175
	36	0.606	0.403	0.197	0.131	0.734	0.488	0.243	0.162	0.801	0.533	0.271	0.180
	38	0.675	0.449	0.201	0.134	0.818	0.544	0.251	0.167	0.892	0.594	0.280	0.186
	40	0.748	0.498	0.207	0.138	0.906	0.603	0.259	0.172	0.989	0.658	0.289	0.192
	42	0.825	0.549	0.212	0.141	0.999	0.665	0.267	0.178	1.09	0.725	0.299	0.199
	44	0.906	0.603	0.218	0.145	1.10	0.729	0.276	0.184	1.20	0.796	0.310	0.206
	46	0.990	0.659	0.224	0.149	1.20	0.797	0.285	0.190	1.31	0.870	0.322	0.214
	48	1.08	0.717	0.230	0.153	1.30	0.868	0.295	0.197	1.42	0.947	0.338	0.225
	50	1.17	0.778	0.237	0.158	1.42	0.942	0.308	0.205	1.55	1.03	0.356	0.237
Other Constants and Properties													
$b_y \times 10^3$, $(\text{kip-ft})^{-1}$		0.904		0.602		1.09		0.723		1.19		0.790	
$t_y \times 10^3$, $(\text{kips})^{-1}$		0.226		0.150		0.263		0.175		0.285		0.190	
$t_r \times 10^3$, $(\text{kips})^{-1}$		0.277		0.185		0.323		0.215		0.351		0.234	
r_x/r_y		4.52				4.55				4.56			
r_y, in.		3.72				3.65				3.64			

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

W40

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W40×											
		392^h				372^h				362^h			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.288	0.192	0.208	0.139	0.304	0.202	0.212	0.141	0.315	0.210	0.217	0.145
	11	0.346	0.230	0.213	0.142	0.335	0.223	0.212	0.141	0.348	0.231	0.217	0.145
	12	0.358	0.238	0.217	0.144	0.341	0.227	0.212	0.141	0.354	0.236	0.217	0.145
	13	0.372	0.247	0.220	0.146	0.348	0.232	0.213	0.142	0.361	0.240	0.218	0.145
	14	0.387	0.258	0.223	0.148	0.356	0.237	0.215	0.143	0.369	0.246	0.221	0.147
	15	0.404	0.269	0.227	0.151	0.365	0.243	0.218	0.145	0.378	0.252	0.224	0.149
	16	0.424	0.282	0.23	0.153	0.374	0.249	0.221	0.147	0.388	0.258	0.227	0.151
	17	0.446	0.296	0.234	0.156	0.384	0.255	0.224	0.149	0.398	0.265	0.230	0.153
	18	0.470	0.313	0.238	0.158	0.395	0.263	0.227	0.151	0.410	0.273	0.233	0.155
	19	0.497	0.331	0.241	0.161	0.407	0.271	0.230	0.153	0.422	0.281	0.236	0.157
	20	0.527	0.351	0.245	0.163	0.420	0.280	0.233	0.155	0.436	0.290	0.239	0.159
	22	0.598	0.398	0.254	0.169	0.450	0.299	0.240	0.159	0.467	0.311	0.246	0.164
	24	0.687	0.457	0.263	0.175	0.485	0.323	0.246	0.164	0.503	0.335	0.253	0.168
	26	0.801	0.533	0.273	0.181	0.526	0.350	0.254	0.169	0.546	0.363	0.261	0.174
	28	0.929	0.618	0.283	0.188	0.574	0.382	0.261	0.174	0.596	0.396	0.269	0.179
	30	1.07	0.710	0.295	0.196	0.631	0.420	0.270	0.179	0.655	0.436	0.278	0.185
	32	1.21	0.807	0.307	0.204	0.698	0.464	0.278	0.185	0.724	0.482	0.287	0.191
	34	1.37	0.911	0.320	0.213	0.777	0.517	0.288	0.191	0.806	0.536	0.297	0.197
	36	1.54	1.02	0.335	0.223	0.871	0.579	0.298	0.198	0.904	0.601	0.307	0.204
	38	1.71	1.14	0.351	0.233	0.970	0.646	0.308	0.205	1.01	0.670	0.319	0.212
	40	1.90	1.26	0.372	0.248	1.08	0.715	0.320	0.213	1.12	0.742	0.331	0.220
	42	2.09	1.39	0.394	0.262	1.19	0.789	0.332	0.221	1.23	0.818	0.344	0.229
	44	2.29	1.53	0.415	0.276	1.30	0.866	0.345	0.230	1.35	0.898	0.358	0.238
	46					1.42	0.946	0.365	0.243	1.48	0.982	0.380	0.253
	48					1.55	1.03	0.385	0.256	1.61	1.07	0.401	0.267
	50					1.68	1.12	0.405	0.270	1.74	1.16	0.422	0.281
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		1.71		1.14		1.29		0.856		1.32		0.878	
$t_y \times 10^3$, (kips)$^{-1}$		0.288		0.192		0.304		0.202		0.315		0.210	
$t_r \times 10^3$, (kips)$^{-1}$		0.354		0.236		0.373		0.249		0.387		0.258	
r_x/r_y		6.10				4.58				4.58			
r_y, in.		2.64				3.60				3.60			

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W40

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W40×											
		331[h]				327[h]				324			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.342	0.227	0.249	0.166	0.348	0.232	0.253	0.168	0.350	0.233	0.244	0.162
	11	0.415	0.276	0.257	0.171	0.422	0.281	0.261	0.174	0.387	0.258	0.244	0.162
	12	0.430	0.286	0.262	0.174	0.437	0.291	0.265	0.177	0.394	0.262	0.244	0.162
	13	0.448	0.298	0.266	0.177	0.455	0.303	0.270	0.180	0.403	0.268	0.245	0.163
	14	0.467	0.311	0.271	0.18	0.475	0.316	0.275	0.183	0.412	0.274	0.249	0.165
	15	0.489	0.326	0.276	0.184	0.497	0.331	0.280	0.186	0.422	0.281	0.252	0.168
	16	0.514	0.342	0.281	0.187	0.522	0.347	0.285	0.190	0.433	0.288	0.256	0.170
	17	0.542	0.361	0.287	0.191	0.550	0.366	0.290	0.193	0.444	0.296	0.259	0.173
	18	0.573	0.381	0.292	0.194	0.581	0.387	0.296	0.197	0.457	0.304	0.263	0.175
	19	0.608	0.404	0.298	0.198	0.616	0.410	0.302	0.201	0.471	0.314	0.267	0.178
	20	0.647	0.430	0.304	0.202	0.656	0.436	0.308	0.205	0.487	0.324	0.271	0.180
	22	0.739	0.492	0.317	0.211	0.749	0.498	0.321	0.213	0.522	0.347	0.279	0.186
	24	0.856	0.570	0.331	0.220	0.866	0.576	0.335	0.223	0.563	0.374	0.288	0.192
	26	1.00	0.668	0.346	0.230	1.01	0.675	0.350	0.233	0.611	0.406	0.298	0.198
	28	1.16	0.774	0.362	0.241	1.18	0.783	0.367	0.244	0.667	0.444	0.308	0.205
	30	1.34	0.889	0.381	0.253	1.35	0.899	0.385	0.256	0.734	0.488	0.319	0.212
	32	1.52	1.01	0.401	0.267	1.54	1.02	0.406	0.270	0.813	0.541	0.330	0.22
	34	1.72	1.14	0.425	0.283	1.73	1.15	0.430	0.286	0.907	0.603	0.343	0.228
	36	1.92	1.28	0.456	0.304	1.95	1.29	0.462	0.307	1.02	0.676	0.357	0.237
	38	2.14	1.43	0.488	0.324	2.17	1.44	0.494	0.329	1.13	0.754	0.371	0.247
	40	2.38	1.58	0.519	0.345	2.40	1.60	0.526	0.350	1.25	0.835	0.387	0.258
	42	2.62	1.74	0.550	0.366	2.65	1.76	0.557	0.371	1.38	0.921	0.408	0.272
	44									1.52	1.01	0.435	0.289
	46									1.66	1.10	0.461	0.307
	48									1.81	1.20	0.488	0.324
	50									1.96	1.30	0.514	0.342

Other Constants and Properties

	331[h] ASD	331[h] LRFD	327[h] ASD	327[h] LRFD	324 ASD	324 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	2.10	1.40	2.12	1.41	1.49	0.992
$t_y \times 10^3$, (kips)$^{-1}$	0.342	0.227	0.348	0.232	0.350	0.233
$t_r \times 10^3$, (kips)$^{-1}$	0.420	0.280	0.428	0.285	0.430	0.287
r_x/r_y	6.19		6.20		4.58	
r_y, in.	2.57		2.58		3.58	

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

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W40

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W40×											
		297^c				294				278			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.385	0.256	0.268	0.178	0.387	0.258	0.281	0.187	0.406	0.270	0.299	0.199
	11	0.424	0.282	0.268	0.178	0.471	0.314	0.291	0.194	0.496	0.330	0.312	0.207
	12	0.432	0.287	0.268	0.178	0.489	0.325	0.296	0.197	0.515	0.343	0.318	0.211
	13	0.441	0.293	0.270	0.179	0.509	0.339	0.302	0.201	0.537	0.357	0.324	0.216
	14	0.451	0.300	0.274	0.182	0.532	0.354	0.308	0.205	0.562	0.374	0.331	0.220
	15	0.462	0.308	0.278	0.185	0.558	0.371	0.314	0.209	0.589	0.392	0.338	0.225
	16	0.474	0.316	0.282	0.188	0.586	0.390	0.321	0.214	0.620	0.413	0.345	0.229
	17	0.488	0.325	0.286	0.190	0.619	0.412	0.328	0.218	0.655	0.436	0.352	0.234
	18	0.502	0.334	0.291	0.193	0.655	0.436	0.335	0.223	0.694	0.462	0.360	0.240
	19	0.518	0.345	0.295	0.197	0.695	0.463	0.342	0.228	0.738	0.491	0.369	0.245
	20	0.535	0.356	0.300	0.200	0.740	0.493	0.350	0.233	0.788	0.524	0.377	0.251
	22	0.575	0.382	0.310	0.206	0.848	0.564	0.366	0.244	0.905	0.602	0.396	0.263
	24	0.621	0.413	0.321	0.213	0.985	0.655	0.384	0.256	1.06	0.702	0.416	0.277
	26	0.675	0.449	0.332	0.221	1.16	0.769	0.404	0.269	1.24	0.824	0.439	0.292
	28	0.739	0.492	0.344	0.229	1.34	0.892	0.426	0.284	1.44	0.956	0.464	0.309
	30	0.815	0.542	0.357	0.238	1.54	1.02	0.451	0.300	1.65	1.10	0.493	0.328
	32	0.904	0.602	0.372	0.247	1.75	1.16	0.482	0.320	1.88	1.25	0.535	0.356
	34	1.01	0.674	0.387	0.257	1.98	1.31	0.521	0.347	2.12	1.41	0.580	0.386
	36	1.13	0.755	0.404	0.269	2.22	1.47	0.561	0.373	2.38	1.58	0.624	0.415
	38	1.26	0.841	0.422	0.281	2.47	1.64	0.601	0.400	2.65	1.76	0.669	0.445
	40	1.40	0.932	0.446	0.297	2.73	1.82	0.640	0.426	2.93	1.95	0.714	0.475
	42	1.54	1.03	0.478	0.318	3.02	2.01	0.679	0.452	3.23	2.15	0.758	0.504
	44	1.70	1.13	0.509	0.339								
	46	1.85	1.23	0.541	0.360								
	48	2.02	1.34	0.573	0.381								
	50	2.19	1.46	0.605	0.403								

Other Constants and Properties

	297^c		294		278	
	ASD	LRFD	ASD	LRFD	ASD	LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	1.66	1.10	2.38	1.58	2.56	1.70
$t_y \times 10^3$, (kips)$^{-1}$	0.383	0.255	0.387	0.258	0.406	0.270
$t_r \times 10^3$, (kips)$^{-1}$	0.470	0.313	0.476	0.317	0.498	0.332
r_x/r_y	4.60		6.24		6.27	
r_y, in.	3.54		2.55		2.52	

^{c} Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W40

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W40×											
		277^c				264				249^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.424	0.282	0.285	0.190	0.432	0.287	0.315	0.210	0.482	0.320	0.318	0.212
	11	0.462	0.308	0.285	0.190	0.527	0.351	0.329	0.219	0.526	0.350	0.318	0.212
	12	0.470	0.313	0.285	0.190	0.548	0.365	0.335	0.223	0.535	0.356	0.318	0.212
	13	0.479	0.319	0.287	0.191	0.571	0.380	0.342	0.228	0.545	0.363	0.320	0.213
	14	0.488	0.325	0.291	0.193	0.597	0.397	0.349	0.233	0.556	0.370	0.325	0.217
	15	0.498	0.332	0.295	0.196	0.627	0.417	0.357	0.238	0.568	0.378	0.331	0.220
	16	0.510	0.339	0.300	0.199	0.660	0.439	0.365	0.243	0.581	0.387	0.336	0.224
	17	0.522	0.347	0.304	0.203	0.697	0.464	0.373	0.248	0.595	0.396	0.342	0.227
	18	0.535	0.356	0.309	0.206	0.738	0.491	0.382	0.254	0.611	0.406	0.347	0.231
	19	0.551	0.367	0.314	0.209	0.785	0.522	0.391	0.260	0.628	0.418	0.353	0.235
	20	0.569	0.379	0.320	0.213	0.838	0.557	0.401	0.267	0.646	0.430	0.359	0.239
	22	0.610	0.406	0.33	0.220	0.963	0.641	0.421	0.280	0.687	0.457	0.372	0.248
	24	0.658	0.438	0.342	0.228	1.12	0.747	0.444	0.295	0.735	0.489	0.386	0.257
	26	0.714	0.475	0.355	0.236	1.32	0.877	0.469	0.312	0.799	0.532	0.401	0.267
	28	0.780	0.519	0.368	0.245	1.53	1.02	0.498	0.331	0.875	0.582	0.417	0.278
	30	0.858	0.571	0.382	0.254	1.75	1.17	0.533	0.354	0.964	0.641	0.435	0.289
	32	0.950	0.632	0.398	0.265	2.00	1.33	0.582	0.387	1.07	0.711	0.454	0.302
	34	1.06	0.705	0.415	0.276	2.25	1.50	0.632	0.420	1.20	0.795	0.475	0.316
	36	1.19	0.791	0.434	0.289	2.53	1.68	0.681	0.453	1.34	0.892	0.498	0.331
	38	1.32	0.881	0.454	0.302	2.81	1.87	0.730	0.486	1.49	0.994	0.530	0.353
	40	1.47	0.976	0.484	0.322	3.12	2.07	0.780	0.519	1.65	1.10	0.573	0.381
	42	1.62	1.08	0.519	0.345	3.44	2.29	0.829	0.552	1.82	1.21	0.616	0.410
	44	1.78	1.18	0.555	0.369					2.00	1.33	0.659	0.438
	46	1.94	1.29	0.590	0.393					2.19	1.46	0.702	0.467
	48	2.11	1.41	0.625	0.416					2.38	1.59	0.746	0.496
	50	2.29	1.53	0.661	0.440					2.59	1.72	0.790	0.525
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		1.75		1.16		2.70		1.80		1.96		1.30	
$t_y \times 10^3$, (kips)$^{-1}$		0.410		0.273		0.432		0.287		0.454		0.302	
$t_r \times 10^3$, (kips)$^{-1}$		0.503		0.336		0.530		0.353		0.558		0.372	
r_x/r_y		4.58				6.27				4.59			
r_y, in.		3.58				2.52				3.55			

^{c} Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W40

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		**W40×**											
		235^c				**215^c**				**211^c**			
Design		**$p \times 10^3$ (kips)$^{-1}$**		**$b_x \times 10^3$ (kip-ft)$^{-1}$**		**$p \times 10^3$ (kips)$^{-1}$**		**$b_x \times 10^3$ (kip-ft)$^{-1}$**		**$p \times 10^3$ (kips)$^{-1}$**		**$b_x \times 10^3$ (kip-ft)$^{-1}$**	
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	**0**	0.503	0.335	0.353	0.235	0.577	0.384	0.370	0.246	0.576	0.383	0.393	0.262
	11	0.596	0.396	0.368	0.245	0.631	0.420	0.370	0.246	0.685	0.456	0.412	0.274
	12	0.615	0.409	0.376	0.250	0.642	0.427	0.370	0.246	0.708	0.471	0.422	0.281
	13	0.637	0.424	0.384	0.255	0.654	0.435	0.373	0.248	0.734	0.488	0.432	0.287
	14	0.666	0.443	0.393	0.261	0.667	0.444	0.379	0.252	0.763	0.507	0.442	0.294
	15	0.698	0.464	0.402	0.267	0.681	0.453	0.385	0.256	0.795	0.529	0.453	0.301
	16	0.734	0.488	0.411	0.274	0.697	0.464	0.392	0.261	0.831	0.553	0.464	0.309
	17	0.775	0.515	0.421	0.280	0.714	0.475	0.399	0.265	0.872	0.580	0.476	0.317
	18	0.820	0.546	0.431	0.287	0.733	0.488	0.406	0.270	0.924	0.615	0.489	0.325
	19	0.871	0.580	0.442	0.294	0.753	0.501	0.413	0.275	0.983	0.654	0.503	0.334
	20	0.928	0.618	0.454	0.302	0.775	0.516	0.421	0.280	1.05	0.698	0.517	0.344
	22	1.06	0.709	0.479	0.319	0.825	0.549	0.437	0.291	1.21	0.803	0.548	0.364
	24	1.24	0.823	0.507	0.337	0.883	0.588	0.455	0.302	1.41	0.938	0.582	0.388
	26	1.45	0.967	0.538	0.358	0.951	0.633	0.473	0.315	1.66	1.10	0.622	0.414
	28	1.68	1.12	0.573	0.381	1.03	0.685	0.494	0.329	1.92	1.28	0.679	0.452
	30	1.93	1.29	0.629	0.419	1.12	0.746	0.516	0.344	2.20	1.47	0.753	0.501
	32	2.20	1.46	0.690	0.459	1.24	0.827	0.541	0.360	2.51	1.67	0.827	0.550
	34	2.48	1.65	0.750	0.499	1.39	0.926	0.568	0.378	2.83	1.88	0.902	0.600
	36	2.79	1.85	0.811	0.540	1.56	1.04	0.603	0.401	3.17	2.11	0.978	0.650
	38	3.10	2.06	0.872	0.580	1.74	1.16	0.657	0.437	3.54	2.35	1.05	0.701
	40	3.44	2.29	0.932	0.620	1.93	1.28	0.712	0.474	3.92	2.61	1.13	0.751
	42	3.79	2.52	0.993	0.661	2.12	1.41	0.768	0.511				
	44					2.33	1.55	0.825	0.549				
	46					2.55	1.69	0.882	0.587				
	48					2.77	1.85	0.939	0.625				
	50					3.01	2.00	0.997	0.663				
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		3.02		2.01		2.28		1.52		3.39		2.26	
$t_y \times 10^3$, (kips)$^{-1}$		0.483		0.322		0.526		0.350		0.538		0.358	
$t_r \times 10^3$, (kips)$^{-1}$		0.594		0.396		0.646		0.431		0.661		0.440	
r_x/r_y		6.26				4.58				6.29			
r_y, in.		2.54				3.54				2.51			

[c] Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W40

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W40×											
		199^c				183^c				167^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.628	0.418	0.410	0.273	0.700	0.466	0.46	0.306	0.764	0.508	0.514	0.342
	11	0.689	0.458	0.410	0.273	0.835	0.555	0.485	0.323	0.921	0.613	0.547	0.364
	12	0.701	0.467	0.410	0.273	0.863	0.574	0.497	0.330	0.954	0.635	0.562	0.374
	13	0.715	0.476	0.416	0.277	0.895	0.595	0.509	0.339	0.991	0.660	0.577	0.384
	14	0.730	0.486	0.423	0.282	0.931	0.619	0.522	0.348	1.03	0.688	0.593	0.395
	15	0.747	0.497	0.431	0.287	0.971	0.646	0.536	0.357	1.08	0.719	0.610	0.406
	16	0.765	0.509	0.439	0.292	1.02	0.676	0.551	0.367	1.13	0.754	0.628	0.418
	17	0.784	0.522	0.447	0.297	1.07	0.709	0.567	0.377	1.19	0.793	0.647	0.431
	18	0.806	0.536	0.455	0.303	1.12	0.746	0.583	0.388	1.26	0.837	0.668	0.444
	19	0.829	0.551	0.464	0.309	1.18	0.787	0.6	0.399	1.33	0.886	0.689	0.459
	20	0.854	0.568	0.473	0.315	1.25	0.833	0.619	0.412	1.41	0.941	0.712	0.474
	22	0.911	0.606	0.493	0.328	1.43	0.948	0.659	0.439	1.64	1.09	0.763	0.508
	24	0.978	0.651	0.514	0.342	1.67	1.11	0.705	0.469	1.94	1.29	0.822	0.547
	26	1.06	0.702	0.537	0.357	1.96	1.30	0.763	0.507	2.28	1.52	0.919	0.611
	28	1.15	0.763	0.562	0.374	2.27	1.51	0.859	0.571	2.65	1.76	1.04	0.690
	30	1.26	0.838	0.590	0.393	2.61	1.74	0.957	0.636	3.04	2.02	1.16	0.771
	32	1.41	0.935	0.621	0.413	2.97	1.98	1.06	0.702	3.45	2.30	1.28	0.853
	34	1.58	1.05	0.655	0.436	3.35	2.23	1.16	0.769	3.90	2.59	1.41	0.937
	36	1.77	1.18	0.716	0.476	3.76	2.50	1.26	0.837	4.37	2.91	1.53	1.02
	38	1.98	1.32	0.782	0.520	4.19	2.79	1.36	0.905	4.87	3.24	1.66	1.11
	40	2.19	1.46	0.849	0.565	4.64	3.09	1.46	0.973	5.40	3.59	1.79	1.19
	42	2.41	1.61	0.918	0.610								
	44	2.65	1.76	0.987	0.657								
	46	2.90	1.93	1.06	0.703								
	48	3.15	2.10	1.13	0.750								
	50	3.42	2.28	1.20	0.797								
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		2.60		1.73		4.03		2.68		4.69		3.12	
$t_y \times 10^3$, (kips)$^{-1}$		0.568		0.378		0.627		0.417		0.677		0.451	
$t_r \times 10^3$, (kips)$^{-1}$		0.698		0.465		0.770		0.513		0.832		0.555	
r_x/r_y		4.64				6.31				6.38			
r_y, in.		3.45				2.49				2.40			

^{c} Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

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W40–W36

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W40×				W36×							
		$149^{c,v}$				925^{h}				853^{h}			
Design		$p \times 10^3$ $(\text{kips})^{-1}$		$b_x \times 10^3$ $(\text{kip-ft})^{-1}$		$p \times 10^3$ $(\text{kips})^{-1}$		$b_x \times 10^3$ $(\text{kip-ft})^{-1}$		$p \times 10^3$ $(\text{kips})^{-1}$		$b_x \times 10^3$ $(\text{kip-ft})^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.879	0.585	0.596	0.396	0.123	0.0817	0.0863	0.0574	0.133	0.0885	0.0909	0.0605
	11	1.08	0.716	0.644	0.429	0.132	0.0876	0.0863	0.0574	0.143	0.0949	0.0909	0.0605
	12	1.12	0.744	0.663	0.441	0.133	0.0888	0.0863	0.0574	0.145	0.0962	0.0909	0.0605
	13	1.17	0.775	0.682	0.454	0.135	0.0901	0.0863	0.0574	0.147	0.0976	0.0909	0.0605
	14	1.22	0.811	0.703	0.468	0.138	0.0915	0.0863	0.0574	0.149	0.0991	0.0909	0.0605
	15	1.28	0.851	0.725	0.483	0.140	0.0931	0.0863	0.0574	0.151	0.101	0.0909	0.0605
	16	1.35	0.896	0.749	0.498	0.142	0.0948	0.0866	0.0576	0.154	0.103	0.0913	0.0607
	17	1.42	0.946	0.774	0.515	0.145	0.0966	0.0871	0.0579	0.157	0.105	0.0917	0.0610
	18	1.51	1.00	0.801	0.533	0.148	0.0986	0.0875	0.0582	0.160	0.107	0.0922	0.0613
	19	1.60	1.07	0.830	0.552	0.151	0.101	0.0879	0.0585	0.164	0.109	0.0927	0.0617
	20	1.71	1.14	0.861	0.573	0.155	0.103	0.0883	0.0587	0.167	0.111	0.0931	0.0620
	22	2.02	1.34	0.930	0.619	0.163	0.108	0.0891	0.0593	0.176	0.117	0.0941	0.0626
	24	2.40	1.60	1.03	0.683	0.172	0.114	0.0900	0.0599	0.185	0.123	0.0951	0.0632
	26	2.82	1.88	1.18	0.783	0.182	0.121	0.0909	0.0605	0.196	0.131	0.0961	0.0639
	28	3.27	2.18	1.33	0.887	0.194	0.129	0.0918	0.0611	0.209	0.139	0.0971	0.0646
	30	3.75	2.50	1.49	0.993	0.207	0.138	0.0927	0.0617	0.223	0.149	0.0981	0.0653
	32	4.27	2.84	1.66	1.10	0.222	0.148	0.0936	0.0623	0.240	0.159	0.0992	0.0660
	34	4.82	3.21	1.82	1.21	0.240	0.160	0.0946	0.0629	0.259	0.172	0.100	0.0667
	36	5.41	3.60	1.99	1.33	0.260	0.173	0.0956	0.0636	0.280	0.186	0.101	0.0674
	38	6.02	4.01	2.16	1.44	0.284	0.189	0.0966	0.0642	0.305	0.203	0.102	0.0682
	40					0.311	0.207	0.0976	0.0649	0.334	0.222	0.104	0.0689
	42					0.342	0.228	0.0986	0.0656	0.368	0.245	0.105	0.0697
	44					0.376	0.250	0.100	0.0663	0.403	0.268	0.106	0.0705
	46					0.411	0.273	0.101	0.0670	0.441	0.293	0.107	0.0714
	48					0.447	0.298	0.102	0.0678	0.480	0.319	0.109	0.0722
	50					0.485	0.323	0.103	0.0685	0.521	0.347	0.110	0.0731

Other Constants and Properties

	W40×149		W36×925		W36×853	
$b_y \times 10^3$, $(\text{kip-ft})^{-1}$	5.74	3.82	0.419	0.279	0.443	0.294
$t_y \times 10^3$, $(\text{kips})^{-1}$	0.763	0.507	0.123	0.0817	0.133	0.0885
$t_r \times 10^3$, $(\text{kips})^{-1}$	0.937	0.624	0.151	0.101	0.163	0.109
r_x/r_y	6.55		3.85		3.90	
r_y, in.	2.29		4.26		4.28	

[c] Shape is slender for compression for F_y = 50 ksi.

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(1) with F_y = 50 ksi; therefore, ϕ_v = 0.90 and Ω_v = 1.67.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W36

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W36×											
		802^h				723^h				652^h			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.142	0.0942	0.0973	0.0648	0.157	0.104	0.109	0.0725	0.174	0.116	0.122	0.0815
	11	0.152	0.101	0.0973	0.0648	0.169	0.112	0.109	0.0725	0.188	0.125	0.122	0.0815
	12	0.154	0.103	0.0973	0.0648	0.171	0.114	0.109	0.0725	0.190	0.127	0.122	0.0815
	13	0.156	0.104	0.0973	0.0648	0.174	0.116	0.109	0.0725	0.193	0.129	0.122	0.0815
	14	0.159	0.106	0.0973	0.0648	0.177	0.117	0.109	0.0725	0.197	0.131	0.122	0.0815
	15	0.162	0.108	0.0974	0.0648	0.180	0.120	0.109	0.0726	0.200	0.133	0.123	0.0817
	16	0.165	0.110	0.0979	0.0651	0.183	0.122	0.110	0.0730	0.204	0.136	0.124	0.0823
	17	0.168	0.112	0.0984	0.0655	0.187	0.124	0.110	0.0735	0.208	0.139	0.124	0.0828
	18	0.171	0.114	0.0990	0.0658	0.191	0.127	0.111	0.0739	0.213	0.142	0.125	0.0833
	19	0.175	0.117	0.100	0.0662	0.195	0.130	0.112	0.0743	0.218	0.145	0.126	0.0839
	20	0.179	0.119	0.100	0.0665	0.200	0.133	0.112	0.0748	0.223	0.149	0.127	0.0845
	22	0.188	0.125	0.101	0.0673	0.210	0.140	0.114	0.0757	0.236	0.157	0.129	0.0856
	24	0.199	0.132	0.102	0.0680	0.222	0.148	0.115	0.0766	0.250	0.166	0.130	0.0868
	26	0.211	0.140	0.103	0.0688	0.236	0.157	0.117	0.0776	0.266	0.177	0.132	0.0880
	28	0.225	0.150	0.105	0.0696	0.252	0.168	0.118	0.0786	0.284	0.189	0.134	0.0892
	30	0.241	0.160	0.106	0.0703	0.270	0.180	0.120	0.0796	0.306	0.203	0.136	0.0905
	32	0.259	0.173	0.107	0.0712	0.292	0.194	0.121	0.0806	0.330	0.220	0.138	0.0918
	34	0.280	0.187	0.108	0.0720	0.316	0.210	0.123	0.0817	0.359	0.239	0.140	0.0932
	36	0.305	0.203	0.109	0.0728	0.344	0.229	0.124	0.0828	0.392	0.261	0.142	0.0946
	38	0.332	0.221	0.111	0.0737	0.376	0.250	0.126	0.0839	0.430	0.286	0.144	0.0960
	40	0.365	0.243	0.112	0.0746	0.414	0.275	0.128	0.0850	0.475	0.316	0.147	0.0975
	42	0.402	0.268	0.114	0.0755	0.456	0.304	0.130	0.0862	0.524	0.348	0.149	0.0990
	44	0.441	0.294	0.115	0.0765	0.501	0.333	0.131	0.0874	0.575	0.382	0.151	0.101
	46	0.482	0.321	0.116	0.0774	0.547	0.364	0.133	0.0887	0.628	0.418	0.154	0.102
	48	0.525	0.349	0.118	0.0784	0.596	0.397	0.135	0.0900	0.684	0.455	0.156	0.104
	50	0.570	0.379	0.119	0.0794	0.647	0.430	0.137	0.0913	0.742	0.494	0.159	0.106

Other Constants and Properties

	802^h ASD	802^h LRFD	723^h ASD	723^h LRFD	652^h ASD	652^h LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	0.479	0.319	0.541	0.360	0.613	0.408
$t_y \times 10^3$, (kips)$^{-1}$	0.142	0.0942	0.157	0.104	0.174	0.116
$t_r \times 10^3$, (kips)$^{-1}$	0.174	0.116	0.193	0.128	0.214	0.142
r_x / r_y	3.93		3.93		3.95	
r_y, in.	4.22		4.17		4.10	

^h Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

W36

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W36×											
		529^h				487^h				441^h			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.214	0.142	0.153	0.102	0.234	0.155	0.167	0.111	0.257	0.171	0.187	0.124
	11	0.232	0.154	0.153	0.102	0.253	0.169	0.167	0.111	0.279	0.186	0.187	0.124
	12	0.235	0.157	0.153	0.102	0.257	0.171	0.167	0.111	0.284	0.189	0.187	0.124
	13	0.239	0.159	0.153	0.102	0.262	0.174	0.167	0.111	0.288	0.192	0.187	0.124
	14	0.244	0.162	0.153	0.102	0.266	0.177	0.167	0.111	0.294	0.196	0.187	0.124
	15	0.248	0.165	0.154	0.102	0.272	0.181	0.169	0.112	0.300	0.199	0.189	0.125
	16	0.253	0.169	0.155	0.103	0.277	0.185	0.170	0.113	0.306	0.204	0.190	0.127
	17	0.259	0.172	0.157	0.104	0.284	0.189	0.172	0.114	0.313	0.208	0.192	0.128
	18	0.265	0.176	0.158	0.105	0.290	0.193	0.173	0.115	0.321	0.213	0.194	0.129
	19	0.272	0.181	0.159	0.106	0.298	0.198	0.175	0.116	0.329	0.219	0.196	0.130
	20	0.279	0.185	0.160	0.107	0.306	0.203	0.176	0.117	0.338	0.225	0.198	0.132
	22	0.294	0.196	0.163	0.109	0.323	0.215	0.180	0.120	0.358	0.238	0.202	0.135
	24	0.313	0.208	0.166	0.110	0.344	0.229	0.183	0.122	0.381	0.254	0.206	0.137
	26	0.334	0.222	0.169	0.112	0.368	0.245	0.187	0.124	0.408	0.272	0.211	0.140
	28	0.359	0.239	0.172	0.114	0.395	0.263	0.190	0.127	0.440	0.293	0.215	0.143
	30	0.387	0.258	0.175	0.117	0.427	0.284	0.194	0.129	0.476	0.317	0.220	0.147
	32	0.420	0.279	0.178	0.119	0.465	0.309	0.198	0.132	0.518	0.345	0.225	0.150
	34	0.458	0.305	0.182	0.121	0.508	0.338	0.202	0.135	0.567	0.377	0.231	0.153
	36	0.502	0.334	0.185	0.123	0.558	0.371	0.207	0.138	0.624	0.415	0.236	0.157
	38	0.554	0.369	0.189	0.126	0.617	0.410	0.211	0.141	0.693	0.461	0.242	0.161
	40	0.614	0.409	0.193	0.128	0.684	0.455	0.216	0.144	0.767	0.511	0.248	0.165
	42	0.677	0.450	0.197	0.131	0.754	0.501	0.221	0.147	0.846	0.563	0.255	0.169
	44	0.743	0.494	0.201	0.134	0.827	0.550	0.226	0.150	0.928	0.618	0.261	0.174
	46	0.812	0.540	0.205	0.137	0.904	0.601	0.232	0.154	1.01	0.675	0.269	0.179
	48	0.884	0.588	0.210	0.140	0.984	0.655	0.237	0.158	1.10	0.735	0.276	0.184
	50	0.960	0.638	0.215	0.143	1.07	0.711	0.243	0.162	1.20	0.798	0.284	0.189
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		0.785		0.522		0.865		0.575		0.968		0.644	
$t_y \times 10^3$, (kips)$^{-1}$		0.214		0.142		0.234		0.155		0.257		0.171	
$t_r \times 10^3$, (kips)$^{-1}$		0.263		0.175		0.287		0.191		0.316		0.210	
r_x / r_y		4.00				3.99				4.01			
r_y, in.		4.00				3.96				3.92			

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

W36

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W36×											
		395[h]				361[h]				330			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.288	0.192	0.208	0.139	0.315	0.210	0.230	0.153	0.345	0.229	0.253	0.168
	11	0.313	0.208	0.208	0.139	0.343	0.228	0.230	0.153	0.376	0.250	0.253	0.168
	12	0.318	0.212	0.208	0.139	0.349	0.232	0.230	0.153	0.382	0.254	0.253	0.168
	13	0.324	0.216	0.208	0.139	0.355	0.236	0.230	0.153	0.389	0.259	0.253	0.168
	14	0.330	0.220	0.209	0.139	0.362	0.241	0.231	0.154	0.397	0.264	0.254	0.169
	15	0.337	0.224	0.211	0.141	0.370	0.246	0.234	0.155	0.405	0.270	0.257	0.171
	16	0.344	0.229	0.213	0.142	0.378	0.251	0.236	0.157	0.414	0.276	0.260	0.173
	17	0.352	0.234	0.216	0.144	0.387	0.257	0.239	0.159	0.424	0.282	0.264	0.175
	18	0.361	0.240	0.218	0.145	0.397	0.264	0.242	0.161	0.435	0.289	0.267	0.178
	19	0.371	0.247	0.221	0.147	0.407	0.271	0.245	0.163	0.447	0.297	0.270	0.180
	20	0.381	0.253	0.223	0.148	0.419	0.279	0.248	0.165	0.459	0.306	0.274	0.182
	22	0.404	0.269	0.228	0.152	0.444	0.296	0.254	0.169	0.488	0.325	0.281	0.187
	24	0.431	0.287	0.234	0.155	0.474	0.316	0.260	0.173	0.521	0.347	0.289	0.192
	26	0.462	0.307	0.239	0.159	0.509	0.339	0.267	0.178	0.560	0.373	0.297	0.198
	28	0.498	0.331	0.245	0.163	0.550	0.366	0.274	0.183	0.605	0.403	0.306	0.204
	30	0.540	0.359	0.251	0.167	0.597	0.397	0.282	0.188	0.658	0.438	0.315	0.210
	32	0.589	0.392	0.258	0.172	0.652	0.434	0.290	0.193	0.719	0.478	0.325	0.216
	34	0.646	0.430	0.265	0.176	0.716	0.477	0.299	0.199	0.790	0.526	0.335	0.223
	36	0.713	0.474	0.272	0.181	0.791	0.526	0.308	0.205	0.874	0.581	0.346	0.230
	38	0.792	0.527	0.280	0.186	0.880	0.586	0.317	0.211	0.973	0.648	0.358	0.238
	40	0.878	0.584	0.288	0.191	0.976	0.649	0.327	0.218	1.08	0.717	0.371	0.247
	42	0.968	0.644	0.296	0.197	1.08	0.716	0.338	0.225	1.19	0.791	0.384	0.256
	44	1.06	0.707	0.305	0.203	1.18	0.785	0.350	0.233	1.30	0.868	0.399	0.265
	46	1.16	0.772	0.315	0.210	1.29	0.858	0.362	0.241	1.43	0.949	0.417	0.277
	48	1.26	0.841	0.325	0.216	1.40	0.935	0.376	0.250	1.55	1.03	0.441	0.293
	50	1.37	0.913	0.336	0.224	1.52	1.01	0.395	0.263	1.69	1.12	0.465	0.309

Other Constants and Properties

	395[h] ASD	395[h] LRFD	361[h] ASD	361[h] LRFD	330 ASD	330 LRFD
$b_y \times 10^3$, $(\text{kip-ft})^{-1}$	1.10	0.729	1.22	0.809	1.34	0.894
$t_y \times 10^3$, $(\text{kips})^{-1}$	0.288	0.192	0.315	0.210	0.345	0.229
$t_r \times 10^3$, $(\text{kips})^{-1}$	0.354	0.236	0.387	0.258	0.423	0.282
r_x / r_y	4.05		4.05		4.05	
r_y, in.	3.88		3.85		3.83	

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

W36

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W36×											
		302				282^c				262^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.375	0.250	0.278	0.185	0.404	0.269	0.299	0.199	0.439	0.292	0.324	0.215
	11	0.410	0.272	0.278	0.185	0.440	0.293	0.299	0.199	0.475	0.316	0.324	0.215
	12	0.416	0.277	0.278	0.185	0.447	0.298	0.299	0.199	0.483	0.321	0.324	0.215
	13	0.424	0.282	0.278	0.185	0.456	0.303	0.299	0.199	0.491	0.326	0.324	0.215
	14	0.432	0.288	0.280	0.186	0.465	0.309	0.302	0.201	0.501	0.333	0.327	0.218
	15	0.441	0.294	0.284	0.189	0.475	0.316	0.306	0.203	0.512	0.340	0.332	0.221
	16	0.451	0.300	0.287	0.191	0.486	0.323	0.310	0.206	0.524	0.348	0.337	0.224
	17	0.462	0.308	0.291	0.194	0.497	0.331	0.314	0.209	0.537	0.357	0.342	0.227
	18	0.474	0.315	0.295	0.196	0.510	0.339	0.319	0.212	0.551	0.366	0.347	0.231
	19	0.487	0.324	0.299	0.199	0.524	0.349	0.323	0.215	0.566	0.377	0.352	0.234
	20	0.501	0.333	0.303	0.202	0.539	0.359	0.328	0.218	0.583	0.388	0.357	0.238
	22	0.532	0.354	0.312	0.208	0.573	0.382	0.338	0.225	0.620	0.413	0.369	0.245
	24	0.569	0.378	0.321	0.214	0.613	0.408	0.348	0.232	0.664	0.442	0.381	0.253
	26	0.611	0.407	0.331	0.220	0.660	0.439	0.359	0.239	0.716	0.476	0.394	0.262
	28	0.661	0.440	0.341	0.227	0.714	0.475	0.371	0.247	0.776	0.516	0.408	0.271
	30	0.718	0.478	0.352	0.234	0.777	0.517	0.384	0.255	0.846	0.563	0.423	0.281
	32	0.786	0.523	0.364	0.242	0.850	0.566	0.397	0.264	0.928	0.617	0.439	0.292
	34	0.864	0.575	0.376	0.250	0.936	0.623	0.412	0.274	1.02	0.681	0.456	0.303
	36	0.956	0.636	0.389	0.259	1.04	0.690	0.428	0.284	1.14	0.757	0.474	0.316
	38	1.07	0.709	0.404	0.269	1.16	0.769	0.444	0.296	1.27	0.843	0.495	0.329
	40	1.18	0.785	0.419	0.279	1.28	0.852	0.463	0.308	1.40	0.934	0.517	0.344
	42	1.30	0.866	0.436	0.290	1.41	0.939	0.482	0.321	1.55	1.03	0.551	0.367
	44	1.43	0.950	0.456	0.303	1.55	1.03	0.514	0.342	1.70	1.13	0.589	0.392
	46	1.56	1.04	0.484	0.322	1.69	1.13	0.547	0.364	1.86	1.24	0.628	0.418
	48	1.70	1.13	0.513	0.341	1.84	1.23	0.580	0.386	2.02	1.35	0.666	0.443
	50	1.84	1.23	0.541	0.360	2.00	1.33	0.612	0.407	2.19	1.46	0.705	0.469
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		1.48		0.984		1.60		1.06		1.75		1.16	
$t_y \times 10^3$, (kips)$^{-1}$		0.375		0.250		0.403		0.268		0.433		0.288	
$t_r \times 10^3$, (kips)$^{-1}$		0.461		0.307		0.495		0.330		0.531		0.354	
r_x/r_y		4.03				4.05				4.07			
r_y, in.		3.82				3.80				3.76			

^{c} Shape is slender for compression for F_y = 50 ksi.

W36

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W36×											
		256				247^c				232^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.444	0.295	0.343	0.228	0.473	0.315	0.346	0.230	0.497	0.330	0.381	0.253
	11	0.532	0.354	0.353	0.235	0.513	0.341	0.346	0.230	0.591	0.393	0.394	0.262
	12	0.550	0.366	0.360	0.239	0.521	0.346	0.346	0.230	0.613	0.408	0.402	0.267
	13	0.571	0.380	0.367	0.244	0.529	0.352	0.346	0.230	0.637	0.424	0.410	0.273
	14	0.595	0.396	0.374	0.249	0.539	0.359	0.350	0.233	0.663	0.441	0.419	0.278
	15	0.622	0.414	0.381	0.254	0.549	0.365	0.355	0.236	0.694	0.461	0.427	0.284
	16	0.651	0.433	0.389	0.259	0.561	0.373	0.360	0.240	0.727	0.484	0.437	0.291
	17	0.684	0.455	0.397	0.264	0.573	0.381	0.366	0.243	0.765	0.509	0.447	0.297
	18	0.721	0.480	0.406	0.270	0.588	0.391	0.372	0.247	0.807	0.537	0.457	0.304
	19	0.762	0.507	0.414	0.276	0.605	0.402	0.378	0.251	0.855	0.569	0.468	0.311
	20	0.808	0.538	0.424	0.282	0.623	0.414	0.384	0.255	0.907	0.604	0.479	0.319
	22	0.916	0.610	0.443	0.295	0.663	0.441	0.396	0.264	1.03	0.687	0.503	0.335
	24	1.05	0.700	0.465	0.309	0.711	0.473	0.410	0.273	1.19	0.791	0.530	0.352
	26	1.22	0.815	0.489	0.325	0.766	0.510	0.424	0.282	1.39	0.923	0.559	0.372
	28	1.42	0.945	0.515	0.343	0.831	0.553	0.440	0.293	1.61	1.07	0.592	0.394
	30	1.63	1.08	0.545	0.362	0.907	0.603	0.457	0.304	1.85	1.23	0.631	0.420
	32	1.86	1.23	0.582	0.387	0.996	0.663	0.475	0.316	2.10	1.40	0.691	0.460
	34	2.09	1.39	0.632	0.420	1.10	0.732	0.495	0.329	2.37	1.58	0.751	0.500
	36	2.35	1.56	0.681	0.453	1.22	0.815	0.516	0.343	2.66	1.77	0.812	0.540
	38	2.62	1.74	0.730	0.486	1.36	0.908	0.539	0.359	2.96	1.97	0.872	0.580
	40	2.90	1.93	0.779	0.519	1.51	1.01	0.570	0.379	3.28	2.18	0.932	0.620
	42	3.20	2.13	0.828	0.551	1.67	1.11	0.613	0.408	3.62	2.41	0.992	0.660
	44	3.51	2.33	0.877	0.584	1.83	1.22	0.657	0.437				
	46					2.00	1.33	0.700	0.466				
	48					2.18	1.45	0.744	0.495				
	50					2.36	1.57	0.788	0.524				

Other Constants and Properties

	256		247^c		232^c	
$b_y \times 10^3$, (kip-ft)$^{-1}$	2.60	1.73	1.88	1.25	2.92	1.94
$t_y \times 10^3$, (kips)$^{-1}$	0.444	0.295	0.461	0.307	0.491	0.327
$t_r \times 10^3$, (kips)$^{-1}$	0.545	0.363	0.566	0.377	0.603	0.402
r_x/r_y	5.62		4.06		5.65	
r_y, in.	2.65		3.74		2.62	

^{c} Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W36

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape	W36×											
	231^c				210^c				194^c			
Design	$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending												
0	0.510	0.339	0.370	0.246	0.554	0.368	0.428	0.285	0.616	0.410	0.464	0.309
11	0.553	0.368	0.370	0.246	0.653	0.435	0.445	0.296	0.727	0.484	0.485	0.322
12	0.562	0.374	0.370	0.246	0.678	0.451	0.454	0.302	0.750	0.499	0.496	0.330
13	0.571	0.380	0.370	0.246	0.705	0.469	0.465	0.309	0.776	0.516	0.507	0.337
14	0.581	0.387	0.375	0.249	0.736	0.489	0.475	0.316	0.805	0.536	0.519	0.345
15	0.593	0.394	0.381	0.253	0.770	0.512	0.486	0.323	0.841	0.560	0.532	0.354
16	0.605	0.403	0.387	0.257	0.809	0.538	0.498	0.331	0.884	0.588	0.545	0.363
17	0.619	0.412	0.393	0.261	0.852	0.567	0.510	0.339	0.932	0.620	0.559	0.372
18	0.633	0.421	0.399	0.266	0.901	0.599	0.523	0.348	0.986	0.656	0.574	0.382
19	0.649	0.432	0.406	0.270	0.955	0.635	0.536	0.357	1.05	0.696	0.589	0.392
20	0.667	0.443	0.412	0.274	1.02	0.676	0.550	0.366	1.11	0.741	0.606	0.403
22	0.709	0.472	0.426	0.284	1.16	0.772	0.580	0.386	1.28	0.848	0.641	0.427
24	0.761	0.506	0.442	0.294	1.34	0.893	0.614	0.409	1.48	0.984	0.681	0.453
26	0.821	0.546	0.458	0.305	1.57	1.05	0.653	0.434	1.73	1.15	0.726	0.483
28	0.892	0.594	0.476	0.316	1.82	1.21	0.696	0.463	2.01	1.34	0.786	0.523
30	0.975	0.649	0.494	0.329	2.09	1.39	0.765	0.509	2.31	1.54	0.873	0.581
32	1.07	0.713	0.515	0.343	2.38	1.58	0.841	0.559	2.63	1.75	0.961	0.639
34	1.19	0.789	0.537	0.357	2.69	1.79	0.917	0.610	2.96	1.97	1.05	0.699
36	1.32	0.880	0.562	0.374	3.01	2.00	0.993	0.661	3.32	2.21	1.14	0.758
38	1.47	0.981	0.588	0.391	3.36	2.23	1.07	0.712	3.70	2.46	1.23	0.818
40	1.63	1.09	0.631	0.420	3.72	2.48	1.15	0.763	4.10	2.73	1.32	0.878
42	1.80	1.20	0.680	0.452	4.10	2.73	1.22	0.814	4.52	3.01	1.41	0.938
44	1.98	1.31	0.729	0.485								
46	2.16	1.44	0.778	0.518								
48	2.35	1.56	0.828	0.551								
50	2.55	1.70	0.878	0.584								

Other Constants and Properties

	231^c ASD	231^c LRFD	210^c ASD	210^c LRFD	194^c ASD	194^c LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	2.02	1.35	3.33	2.22	3.65	2.43
$t_y \times 10^3$, (kips)$^{-1}$	0.490	0.326	0.540	0.359	0.586	0.390
$t_r \times 10^3$, (kips)$^{-1}$	0.602	0.401	0.663	0.442	0.720	0.480
r_x/r_y	4.07		5.66		5.70	
r_y, in.	3.71		2.58		2.56	

[c] Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W36

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

	Shape	W36×											
		182^c				170^c				160^c			
	Design	$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.667	0.444	0.496	0.330	0.729	0.485	0.533	0.355	0.788	0.524	0.571	0.380
	11	0.787	0.524	0.519	0.345	0.863	0.574	0.559	0.372	0.936	0.623	0.601	0.400
	12	0.813	0.541	0.531	0.353	0.891	0.593	0.573	0.381	0.967	0.643	0.616	0.410
	13	0.841	0.559	0.544	0.362	0.923	0.614	0.587	0.390	1.00	0.667	0.632	0.420
	14	0.873	0.581	0.557	0.371	0.958	0.637	0.602	0.400	1.04	0.693	0.648	0.431
	15	0.908	0.604	0.571	0.380	0.997	0.664	0.617	0.411	1.08	0.722	0.666	0.443
	16	0.947	0.630	0.586	0.390	1.04	0.693	0.634	0.422	1.13	0.754	0.684	0.455
	17	0.995	0.662	0.601	0.400	1.09	0.725	0.651	0.433	1.19	0.790	0.703	0.468
	18	1.05	0.701	0.618	0.411	1.14	0.762	0.670	0.445	1.25	0.831	0.724	0.482
	19	1.12	0.744	0.635	0.422	1.21	0.805	0.689	0.458	1.32	0.876	0.746	0.496
	20	1.19	0.792	0.653	0.435	1.29	0.858	0.710	0.472	1.39	0.928	0.769	0.511
	22	1.36	0.908	0.693	0.461	1.48	0.985	0.755	0.502	1.61	1.07	0.820	0.545
	24	1.58	1.05	0.738	0.491	1.72	1.15	0.806	0.536	1.88	1.25	0.878	0.584
	26	1.86	1.24	0.789	0.525	2.02	1.35	0.864	0.575	2.20	1.47	0.950	0.632
	28	2.16	1.43	0.868	0.577	2.35	1.56	0.966	0.643	2.56	1.70	1.07	0.714
	30	2.47	1.65	0.966	0.642	2.69	1.79	1.08	0.717	2.94	1.95	1.20	0.797
	32	2.81	1.87	1.07	0.709	3.07	2.04	1.19	0.792	3.34	2.22	1.33	0.883
	34	3.18	2.11	1.17	0.775	3.46	2.30	1.31	0.869	3.77	2.51	1.46	0.969
	36	3.56	2.37	1.27	0.843	3.88	2.58	1.42	0.946	4.23	2.81	1.59	1.06
	38	3.97	2.64	1.37	0.911	4.32	2.88	1.54	1.02	4.71	3.13	1.72	1.15
	40	4.40	2.93	1.47	0.979	4.79	3.19	1.66	1.10	5.22	3.47	1.86	1.23
	42	4.85	3.23	1.570	1.05	5.28	3.51	1.77	1.18				

Other Constants and Properties

	182^c ASD	182^c LRFD	170^c ASD	170^c LRFD	160^c ASD	160^c LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	3.93	2.61	4.25	2.83	4.61	3.07
$t_y \times 10^3$, (kips)$^{-1}$	0.623	0.415	0.668	0.444	0.711	0.473
$t_r \times 10^3$, (kips)$^{-1}$	0.765	0.510	0.821	0.547	0.873	0.582
r_x / r_y	5.69		5.73		5.76	
r_y, in.	2.55		2.53		2.50	

[c] Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c / r_y equal to or greater than 200.

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W36–W33

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape	W36×								W33×			
	150[c]				135[c,v]				387[h]			
Design	$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending												
0	0.848	0.564	0.613	0.408	0.963	0.641	0.700	0.466	0.293	0.195	0.228	0.152
11	1.01	0.673	0.648	0.431	1.16	0.772	0.748	0.498	0.320	0.213	0.228	0.152
12	1.05	0.696	0.665	0.442	1.20	0.800	0.769	0.512	0.326	0.217	0.228	0.152
13	1.08	0.721	0.682	0.454	1.25	0.832	0.791	0.526	0.332	0.221	0.228	0.152
14	1.13	0.750	0.701	0.466	1.30	0.867	0.814	0.541	0.339	0.225	0.230	0.153
15	1.18	0.782	0.721	0.479	1.36	0.907	0.838	0.558	0.346	0.230	0.232	0.155
16	1.23	0.818	0.741	0.493	1.43	0.951	0.864	0.575	0.354	0.236	0.235	0.156
17	1.29	0.858	0.763	0.508	1.50	1.00	0.892	0.593	0.363	0.241	0.237	0.158
18	1.36	0.903	0.786	0.523	1.59	1.06	0.921	0.613	0.372	0.248	0.239	0.159
19	1.43	0.953	0.811	0.540	1.68	1.12	0.952	0.634	0.383	0.255	0.242	0.161
20	1.52	1.01	0.837	0.557	1.78	1.19	0.986	0.656	0.394	0.262	0.244	0.163
22	1.74	1.16	0.895	0.596	2.06	1.37	1.06	0.706	0.419	0.279	0.250	0.166
24	2.04	1.36	0.962	0.640	2.44	1.62	1.15	0.763	0.449	0.299	0.255	0.170
26	2.40	1.59	1.06	0.706	2.87	1.91	1.31	0.871	0.483	0.322	0.261	0.174
28	2.78	1.85	1.2	0.799	3.32	2.21	1.49	0.989	0.524	0.348	0.267	0.178
30	3.19	2.12	1.34	0.894	3.82	2.54	1.67	1.11	0.571	0.380	0.273	0.182
32	3.63	2.42	1.49	0.991	4.34	2.89	1.85	1.23	0.626	0.416	0.280	0.186
34	4.10	2.73	1.64	1.09	4.90	3.26	2.05	1.36	0.690	0.459	0.287	0.191
36	4.59	3.06	1.79	1.19	5.49	3.66	2.24	1.49	0.766	0.510	0.294	0.196
38	5.12	3.41	1.94	1.29	6.12	4.07	2.44	1.62	0.854	0.568	0.302	0.201
40	5.67	3.77	2.10	1.40					0.946	0.629	0.310	0.206
42									1.04	0.694	0.318	0.212
44									1.14	0.762	0.327	0.218
46									1.25	0.832	0.337	0.224
48									1.36	0.906	0.347	0.231
50									1.48	0.984	0.358	0.238

Other Constants and Properties

	W36×150 ASD	W36×150 LRFD	W36×135 ASD	W36×135 LRFD	W33×387 ASD	W33×387 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	5.02	3.34	5.97	3.97	1.14	0.760
$t_y \times 10^3$, (kips)$^{-1}$	0.754	0.502	0.837	0.557	0.293	0.195
$t_r \times 10^3$, (kips)$^{-1}$	0.926	0.617	1.03	0.685	0.360	0.240
r_x/r_y	5.79		5.88		3.87	
r_y, in.	2.47		2.38		3.77	

[c] Shape is slender for compression for F_y = 50 ksi.

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(1) with F_y = 50 ksi; therefore, ϕ_v = 0.90 and Ω_v = 1.67.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

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W33

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W33×											
		354^h				318				291			
Design		$p\times10^3$ $(\text{kips})^{-1}$		$b_x\times10^3$ $(\text{kip-ft})^{-1}$		$p\times10^3$ $(\text{kips})^{-1}$		$b_x\times10^3$ $(\text{kip-ft})^{-1}$		$p\times10^3$ $(\text{kips})^{-1}$		$b_x\times10^3$ $(\text{kip-ft})^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.321	0.214	0.251	0.167	0.356	0.237	0.281	0.187	0.390	0.260	0.307	0.204
	11	0.352	0.234	0.251	0.167	0.391	0.260	0.281	0.187	0.429	0.285	0.307	0.204
	12	0.358	0.238	0.251	0.167	0.398	0.265	0.281	0.187	0.436	0.290	0.307	0.204
	13	0.365	0.243	0.251	0.167	0.406	0.270	0.281	0.187	0.445	0.296	0.307	0.204
	14	0.372	0.248	0.253	0.168	0.414	0.276	0.283	0.189	0.454	0.302	0.311	0.207
	15	0.380	0.253	0.256	0.170	0.423	0.282	0.287	0.191	0.465	0.309	0.315	0.210
	16	0.389	0.259	0.259	0.172	0.434	0.288	0.290	0.193	0.476	0.317	0.319	0.212
	17	0.399	0.266	0.261	0.174	0.445	0.296	0.294	0.195	0.488	0.325	0.323	0.215
	18	0.410	0.273	0.264	0.176	0.457	0.304	0.297	0.198	0.502	0.334	0.328	0.218
	19	0.421	0.280	0.267	0.178	0.470	0.313	0.301	0.200	0.517	0.344	0.332	0.221
	20	0.434	0.289	0.270	0.180	0.484	0.322	0.305	0.203	0.533	0.354	0.337	0.224
	22	0.462	0.308	0.277	0.184	0.516	0.343	0.313	0.208	0.568	0.378	0.346	0.230
	24	0.495	0.330	0.283	0.189	0.554	0.368	0.321	0.214	0.611	0.406	0.356	0.237
	26	0.534	0.355	0.290	0.193	0.598	0.398	0.330	0.220	0.660	0.439	0.367	0.244
	28	0.579	0.386	0.298	0.198	0.649	0.432	0.339	0.226	0.718	0.478	0.378	0.251
	30	0.632	0.421	0.305	0.203	0.710	0.472	0.349	0.232	0.786	0.523	0.390	0.259
	32	0.694	0.462	0.313	0.208	0.780	0.519	0.359	0.239	0.865	0.576	0.403	0.268
	34	0.767	0.510	0.322	0.214	0.863	0.574	0.370	0.246	0.959	0.638	0.416	0.277
	36	0.854	0.568	0.331	0.220	0.963	0.641	0.382	0.254	1.07	0.713	0.431	0.287
	38	0.951	0.633	0.340	0.227	1.07	0.714	0.395	0.263	1.19	0.794	0.447	0.297
	40	1.05	0.701	0.351	0.233	1.19	0.791	0.408	0.271	1.32	0.880	0.463	0.308
	42	1.16	0.773	0.361	0.240	1.31	0.872	0.422	0.281	1.46	0.970	0.482	0.320
	44	1.28	0.848	0.373	0.248	1.44	0.957	0.438	0.291	1.60	1.06	0.503	0.335
	46	1.39	0.927	0.385	0.256	1.57	1.05	0.454	0.302	1.75	1.16	0.533	0.354
	48	1.52	1.01	0.398	0.265	1.71	1.14	0.477	0.318	1.90	1.27	0.563	0.374
	50	1.65	1.10	0.412	0.274	1.86	1.24	0.502	0.334	2.07	1.37	0.592	0.394
Other Constants and Properties													
$b_y\times10^3$, $(\text{kip-ft})^{-1}$		1.26		0.841		1.43		0.948		1.58		1.05	
$t_y\times10^3$, $(\text{kips})^{-1}$		0.321		0.214		0.356		0.237		0.390		0.260	
$t_r\times10^3$, $(\text{kips})^{-1}$		0.394		0.263		0.438		0.292		0.479		0.320	
r_x/r_y		3.88				3.91				3.91			
r_y, in.		3.74				3.71				3.68			

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

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W33

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W33×											
		263				241^c				221^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.432	0.287	0.343	0.228	0.470	0.313	0.379	0.252	0.520	0.346	0.416	0.277
	11	0.475	0.316	0.343	0.228	0.518	0.344	0.379	0.252	0.568	0.378	0.416	0.277
	12	0.483	0.322	0.343	0.228	0.527	0.351	0.379	0.252	0.577	0.384	0.416	0.277
	13	0.493	0.328	0.343	0.228	0.538	0.358	0.380	0.253	0.587	0.391	0.418	0.278
	14	0.503	0.335	0.348	0.231	0.550	0.366	0.386	0.257	0.600	0.399	0.424	0.282
	15	0.515	0.343	0.352	0.234	0.563	0.374	0.391	0.260	0.615	0.409	0.431	0.286
	16	0.528	0.351	0.357	0.238	0.577	0.384	0.397	0.264	0.630	0.419	0.437	0.291
	17	0.542	0.360	0.362	0.241	0.593	0.394	0.403	0.268	0.648	0.431	0.444	0.296
	18	0.557	0.370	0.367	0.244	0.609	0.405	0.409	0.272	0.666	0.443	0.451	0.300
	19	0.573	0.381	0.373	0.248	0.628	0.418	0.416	0.276	0.687	0.457	0.459	0.305
	20	0.591	0.393	0.378	0.252	0.648	0.431	0.422	0.281	0.709	0.472	0.467	0.310
	22	0.631	0.420	0.390	0.259	0.693	0.461	0.436	0.290	0.760	0.505	0.483	0.321
	24	0.679	0.452	0.402	0.267	0.746	0.496	0.450	0.300	0.819	0.545	0.500	0.333
	26	0.734	0.488	0.415	0.276	0.809	0.538	0.466	0.310	0.889	0.591	0.519	0.345
	28	0.799	0.532	0.428	0.285	0.882	0.587	0.483	0.321	0.970	0.646	0.539	0.358
	30	0.875	0.582	0.443	0.295	0.968	0.644	0.501	0.333	1.07	0.710	0.560	0.373
	32	0.965	0.642	0.459	0.305	1.07	0.712	0.520	0.346	1.18	0.786	0.584	0.388
	34	1.07	0.712	0.476	0.317	1.19	0.791	0.541	0.360	1.32	0.876	0.609	0.405
	36	1.20	0.797	0.494	0.329	1.33	0.887	0.564	0.375	1.48	0.982	0.637	0.424
	38	1.33	0.888	0.514	0.342	1.48	0.988	0.589	0.392	1.64	1.09	0.667	0.444
	40	1.48	0.984	0.535	0.356	1.65	1.09	0.619	0.412	1.82	1.21	0.719	0.478
	42	1.63	1.08	0.562	0.374	1.81	1.21	0.663	0.441	2.01	1.34	0.772	0.514
	44	1.79	1.19	0.598	0.398	1.99	1.32	0.708	0.471	2.20	1.47	0.825	0.549
	46	1.96	1.30	0.635	0.422	2.18	1.45	0.753	0.501	2.41	1.60	0.879	0.585
	48	2.13	1.42	0.672	0.447	2.37	1.58	0.797	0.530	2.62	1.75	0.932	0.620
	50	2.31	1.54	0.708	0.471	2.57	1.71	0.842	0.560	2.85	1.89	0.986	0.656
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		1.76		1.17		1.96		1.30		2.17		1.45	
$t_y \times 10^3$, (kips)$^{-1}$		0.432		0.287		0.470		0.313		0.511		0.340	
$t_r \times 10^3$, (kips)$^{-1}$		0.530		0.353		0.577		0.385		0.628		0.419	
r_x/r_y		3.91				3.90				3.93			
r_y, in.		3.66				3.62				3.59			

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^{c} Shape is slender for compression for F_y = 50 ksi.

W33

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W33×											
		201^c				169^c				152^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.587	0.390	0.461	0.307	0.718	0.478	0.566	0.377	0.807	0.537	0.637	0.424
	11	0.641	0.426	0.461	0.307	0.855	0.569	0.595	0.396	0.964	0.641	0.673	0.447
	12	0.652	0.434	0.461	0.307	0.884	0.588	0.608	0.405	0.997	0.663	0.689	0.459
	13	0.664	0.441	0.464	0.309	0.917	0.610	0.623	0.415	1.03	0.688	0.707	0.470
	14	0.677	0.450	0.471	0.314	0.953	0.634	0.638	0.425	1.08	0.716	0.725	0.483
	15	0.691	0.460	0.479	0.319	0.994	0.661	0.654	0.435	1.12	0.747	0.745	0.496
	16	0.707	0.470	0.487	0.324	1.04	0.692	0.671	0.447	1.18	0.782	0.765	0.509
	17	0.724	0.482	0.495	0.329	1.10	0.731	0.689	0.458	1.23	0.821	0.787	0.524
	18	0.743	0.494	0.504	0.335	1.16	0.775	0.708	0.471	1.30	0.866	0.810	0.539
	19	0.763	0.508	0.512	0.341	1.24	0.825	0.728	0.484	1.39	0.923	0.834	0.555
	20	0.788	0.524	0.522	0.347	1.32	0.881	0.749	0.498	1.48	0.987	0.860	0.572
	22	0.845	0.562	0.541	0.360	1.52	1.01	0.794	0.528	1.71	1.14	0.917	0.610
	24	0.912	0.607	0.561	0.374	1.78	1.19	0.846	0.563	2.01	1.34	0.982	0.653
	26	0.991	0.659	0.584	0.388	2.09	1.39	0.905	0.602	2.36	1.57	1.07	0.709
	28	1.08	0.721	0.608	0.404	2.43	1.62	0.999	0.664	2.74	1.82	1.20	0.798
	30	1.19	0.794	0.634	0.422	2.79	1.85	1.11	0.737	3.15	2.09	1.33	0.888
	32	1.32	0.880	0.663	0.441	3.17	2.11	1.22	0.810	3.58	2.38	1.47	0.979
	34	1.48	0.984	0.694	0.462	3.58	2.38	1.33	0.883	4.04	2.69	1.61	1.07
	36	1.66	1.10	0.728	0.484	4.01	2.67	1.44	0.957	4.53	3.02	1.75	1.16
	38	1.85	1.23	0.782	0.520	4.47	2.98	1.55	1.03	5.05	3.36	1.89	1.26
	40	2.05	1.36	0.846	0.563	4.95	3.30	1.66	1.10	5.60	3.72	2.03	1.35
	42	2.26	1.50	0.910	0.606								
	44	2.48	1.65	0.975	0.649								
	46	2.71	1.80	1.04	0.692								
	48	2.95	1.96	1.11	0.736								
	50	3.20	2.13	1.17	0.780								
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		2.42		1.61		4.22		2.81		4.82		3.21	
$t_y \times 10^3$, (kips)$^{-1}$		0.565		0.376		0.675		0.449		0.744		0.495	
$t_r \times 10^3$, (kips)$^{-1}$		0.694		0.463		0.829		0.553		0.914		0.609	
r_x/r_y		3.93				5.48				5.47			
r_y, in.		3.56				2.50				2.47			

^{c} Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

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W33

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W33×											
		141^c				130^c				$118^{c,v}$			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.888	0.591	0.693	0.461	0.979	0.651	0.763	0.508	1.11	0.736	0.858	0.571
	11	1.07	0.710	0.735	0.489	1.18	0.786	0.814	0.542	1.35	0.897	0.926	0.616
	12	1.10	0.735	0.754	0.502	1.22	0.815	0.837	0.557	1.40	0.931	0.952	0.634
	13	1.15	0.763	0.774	0.515	1.27	0.847	0.860	0.572	1.46	0.970	0.98	0.652
	14	1.19	0.795	0.796	0.529	1.33	0.883	0.885	0.589	1.52	1.01	1.01	0.672
	15	1.25	0.830	0.818	0.544	1.39	0.924	0.911	0.606	1.60	1.06	1.04	0.693
	16	1.31	0.870	0.841	0.560	1.46	0.969	0.939	0.624	1.68	1.12	1.08	0.716
	17	1.37	0.915	0.866	0.576	1.53	1.02	0.968	0.644	1.77	1.18	1.11	0.740
	18	1.45	0.964	0.893	0.594	1.62	1.08	0.999	0.665	1.88	1.25	1.15	0.765
	19	1.53	1.02	0.921	0.613	1.71	1.14	1.03	0.687	1.99	1.33	1.19	0.793
	20	1.64	1.09	0.951	0.633	1.82	1.21	1.07	0.711	2.12	1.41	1.24	0.822
	22	1.91	1.27	1.02	0.677	2.13	1.42	1.15	0.764	2.48	1.65	1.34	0.888
	24	2.25	1.50	1.09	0.728	2.52	1.68	1.24	0.826	2.95	1.97	1.48	0.984
	26	2.64	1.76	1.21	0.808	2.96	1.97	1.41	0.939	3.47	2.31	1.70	1.13
	28	3.07	2.04	1.37	0.911	3.43	2.28	1.60	1.06	4.02	2.68	1.92	1.28
	30	3.52	2.34	1.53	1.02	3.94	2.62	1.78	1.19	4.62	3.07	2.16	1.44
	32	4.00	2.66	1.69	1.12	4.48	2.98	1.98	1.32	5.25	3.49	2.40	1.59
	34	4.52	3.01	1.85	1.23	5.06	3.37	2.17	1.45	5.93	3.95	2.64	1.76
	36	5.07	3.37	2.02	1.34	5.68	3.78	2.37	1.58	6.65	4.42	2.89	1.92
	38	5.65	3.76	2.18	1.45	6.32	4.21	2.57	1.71	7.41	4.93	3.14	2.09
	40	6.26	4.16	2.35	1.56								
	42												
	44												
	46												
	48												
	50												

Other Constants and Properties

	141 ASD	141 LRFD	130 ASD	130 LRFD	118 ASD	118 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	5.33	3.54	5.99	3.98	6.94	4.62
$t_y \times 10^3$, (kips)$^{-1}$	0.805	0.535	0.872	0.580	0.963	0.640
$t_r \times 10^3$, (kips)$^{-1}$	0.989	0.659	1.07	0.714	1.18	0.788
r_x/r_y	5.51		5.52		5.60	
r_y, in.	2.43		2.39		3.32	

[c] Shape is slender for compression for F_y = 50 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(1) with F_y = 50 ksi; therefore, ϕ_v = 0.90 and Ω_v = 1.67.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

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W30

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W30×											
		391^h				357^h				326^h			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.290	0.193	0.246	0.163	0.318	0.212	0.270	0.180	0.348	0.232	0.299	0.199
	11	0.319	0.212	0.246	0.163	0.350	0.233	0.270	0.180	0.384	0.256	0.299	0.199
	12	0.325	0.216	0.246	0.163	0.357	0.237	0.270	0.180	0.392	0.260	0.299	0.199
	13	0.331	0.221	0.246	0.164	0.364	0.242	0.270	0.180	0.400	0.266	0.300	0.200
	14	0.339	0.225	0.248	0.165	0.372	0.247	0.273	0.182	0.408	0.272	0.303	0.202
	15	0.346	0.230	0.250	0.166	0.380	0.253	0.276	0.183	0.418	0.278	0.307	0.204
	16	0.355	0.236	0.252	0.168	0.390	0.259	0.278	0.185	0.429	0.285	0.310	0.206
	17	0.364	0.242	0.255	0.169	0.400	0.266	0.281	0.187	0.440	0.293	0.313	0.208
	18	0.374	0.249	0.257	0.171	0.412	0.274	0.284	0.189	0.453	0.301	0.317	0.211
	19	0.385	0.256	0.259	0.172	0.424	0.282	0.287	0.191	0.467	0.311	0.320	0.213
	20	0.397	0.264	0.262	0.174	0.437	0.291	0.290	0.193	0.482	0.321	0.324	0.215
	22	0.424	0.282	0.267	0.177	0.467	0.311	0.296	0.197	0.516	0.343	0.331	0.220
	24	0.456	0.303	0.272	0.181	0.503	0.334	0.302	0.201	0.556	0.370	0.339	0.225
	26	0.493	0.328	0.277	0.184	0.544	0.362	0.308	0.205	0.603	0.401	0.347	0.231
	28	0.536	0.357	0.282	0.188	0.593	0.395	0.315	0.210	0.658	0.438	0.355	0.236
	30	0.587	0.391	0.288	0.192	0.650	0.433	0.322	0.215	0.724	0.481	0.364	0.242
	32	0.647	0.430	0.294	0.196	0.718	0.478	0.330	0.220	0.800	0.532	0.373	0.248
	34	0.717	0.477	0.300	0.200	0.797	0.530	0.338	0.225	0.891	0.593	0.383	0.255
	36	0.802	0.533	0.307	0.204	0.892	0.594	0.346	0.230	0.999	0.665	0.393	0.262
	38	0.893	0.594	0.314	0.209	0.994	0.662	0.355	0.236	1.11	0.741	0.404	0.269
	40	0.990	0.658	0.321	0.213	1.10	0.733	0.364	0.242	1.23	0.821	0.416	0.277
	42	1.09	0.726	0.328	0.218	1.21	0.808	0.373	0.248	1.36	0.905	0.428	0.285
	44	1.20	0.797	0.336	0.224	1.33	0.887	0.383	0.255	1.49	0.993	0.441	0.293
	46	1.31	0.871	0.344	0.229	1.46	0.969	0.394	0.262	1.63	1.09	0.454	0.302
	48	1.43	0.948	0.353	0.235	1.59	1.06	0.405	0.270	1.78	1.18	0.469	0.312
	50	1.55	1.03	0.362	0.241	1.72	1.15	0.417	0.278	1.93	1.28	0.485	0.322
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		1.15		0.765		1.28		0.850		1.41		0.941	
$t_y \times 10^3$, (kips)$^{-1}$		0.290		0.193		0.318		0.212		0.348		0.232	
$t_r \times 10^3$, (kips)$^{-1}$		0.357		0.238		0.391		0.260		0.428		0.285	
r_x/r_y		3.65				3.65				3.67			
r_y, in.		3.67				3.64				3.60			

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

W30

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W30×											
		292				261				235			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.388	0.258	0.336	0.224	0.434	0.289	0.378	0.251	0.482	0.321	0.421	0.280
	11	0.429	0.285	0.336	0.224	0.480	0.320	0.378	0.251	0.534	0.356	0.421	0.280
	12	0.437	0.291	0.336	0.224	0.490	0.326	0.378	0.251	0.545	0.363	0.421	0.280
	13	0.446	0.297	0.337	0.225	0.500	0.333	0.380	0.253	0.557	0.370	0.424	0.282
	14	0.456	0.304	0.341	0.227	0.512	0.341	0.385	0.256	0.570	0.379	0.430	0.286
	15	0.467	0.311	0.345	0.230	0.525	0.349	0.390	0.260	0.584	0.389	0.436	0.290
	16	0.479	0.319	0.349	0.232	0.539	0.358	0.395	0.263	0.600	0.399	0.442	0.294
	17	0.492	0.328	0.353	0.235	0.554	0.368	0.400	0.266	0.617	0.411	0.448	0.298
	18	0.507	0.337	0.358	0.238	0.570	0.379	0.406	0.270	0.636	0.423	0.455	0.302
	19	0.522	0.348	0.362	0.241	0.588	0.392	0.411	0.274	0.656	0.437	0.461	0.307
	20	0.539	0.359	0.366	0.244	0.608	0.405	0.417	0.277	0.678	0.451	0.468	0.311
	22	0.578	0.385	0.376	0.250	0.653	0.434	0.429	0.285	0.729	0.485	0.483	0.321
	24	0.623	0.415	0.385	0.256	0.706	0.470	0.441	0.294	0.788	0.525	0.498	0.331
	26	0.677	0.450	0.396	0.263	0.768	0.511	0.454	0.302	0.859	0.571	0.514	0.342
	28	0.740	0.492	0.406	0.270	0.841	0.560	0.468	0.312	0.942	0.627	0.531	0.354
	30	0.813	0.541	0.418	0.278	0.928	0.617	0.483	0.322	1.04	0.692	0.550	0.366
	32	0.901	0.599	0.430	0.286	1.03	0.686	0.499	0.332	1.16	0.769	0.570	0.379
	34	1.00	0.669	0.443	0.295	1.15	0.768	0.516	0.343	1.30	0.863	0.591	0.393
	36	1.13	0.749	0.456	0.304	1.29	0.861	0.534	0.356	1.45	0.968	0.614	0.409
	38	1.26	0.835	0.471	0.313	1.44	0.959	0.554	0.368	1.62	1.08	0.639	0.425
	40	1.39	0.925	0.486	0.323	1.60	1.06	0.575	0.382	1.80	1.19	0.666	0.443
	42	1.53	1.02	0.502	0.334	1.76	1.17	0.597	0.398	1.98	1.32	0.704	0.468
	44	1.68	1.12	0.520	0.346	1.93	1.29	0.626	0.416	2.17	1.45	0.748	0.498
	46	1.84	1.22	0.539	0.358	2.11	1.41	0.662	0.440	2.37	1.58	0.792	0.527
	48	2.00	1.33	0.564	0.375	2.30	1.53	0.698	0.464	2.59	1.72	0.837	0.557
	50	2.17	1.45	0.592	0.394	2.50	1.66	0.734	0.488	2.81	1.87	0.881	0.586
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		1.60		1.06		1.82		1.21		2.04		1.35	
$t_y \times 10^3$, (kips)$^{-1}$		0.388		0.258		0.434		0.289		0.482		0.321	
$t_r \times 10^3$, (kips)$^{-1}$		0.477		0.318		0.533		0.355		0.592		0.395	
r_x / r_y		3.69				3.71				3.70			
r_y, in.		3.58				3.53				3.51			

W30

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W30×											
		211				191^c				173^c			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.536	0.357	0.474	0.316	0.602	0.401	0.528	0.351	0.677	0.450	0.587	0.391
	11	0.595	0.396	0.474	0.316	0.662	0.441	0.528	0.351	0.745	0.495	0.587	0.391
	12	0.607	0.404	0.474	0.316	0.676	0.450	0.528	0.351	0.758	0.505	0.587	0.391
	13	0.620	0.413	0.479	0.319	0.691	0.460	0.534	0.355	0.774	0.515	0.596	0.396
	14	0.635	0.423	0.486	0.323	0.707	0.471	0.543	0.361	0.790	0.526	0.606	0.403
	15	0.651	0.433	0.493	0.328	0.726	0.483	0.551	0.367	0.809	0.538	0.616	0.410
	16	0.669	0.445	0.501	0.333	0.746	0.496	0.560	0.373	0.829	0.551	0.626	0.417
	17	0.688	0.458	0.509	0.338	0.768	0.511	0.570	0.379	0.851	0.566	0.637	0.424
	18	0.709	0.472	0.517	0.344	0.792	0.527	0.579	0.385	0.878	0.584	0.649	0.432
	19	0.732	0.487	0.525	0.349	0.818	0.544	0.589	0.392	0.908	0.604	0.660	0.439
	20	0.758	0.504	0.533	0.355	0.846	0.563	0.599	0.399	0.941	0.626	0.673	0.447
	22	0.815	0.542	0.551	0.367	0.911	0.606	0.621	0.413	1.01	0.675	0.698	0.465
	24	0.882	0.587	0.570	0.379	0.988	0.657	0.644	0.429	1.10	0.733	0.726	0.483
	26	0.962	0.640	0.591	0.393	1.08	0.718	0.669	0.445	1.21	0.802	0.756	0.503
	28	1.06	0.702	0.613	0.408	1.19	0.789	0.696	0.463	1.33	0.884	0.789	0.525
	30	1.17	0.777	0.636	0.423	1.31	0.874	0.726	0.483	1.48	0.982	0.825	0.549
	32	1.30	0.864	0.662	0.440	1.47	0.975	0.758	0.504	1.65	1.10	0.864	0.575
	34	1.46	0.971	0.690	0.459	1.65	1.10	0.793	0.527	1.86	1.24	0.906	0.603
	36	1.64	1.09	0.720	0.479	1.85	1.23	0.831	0.553	2.09	1.39	0.964	0.641
	38	1.82	1.21	0.753	0.501	2.06	1.37	0.889	0.591	2.32	1.55	1.05	0.696
	40	2.02	1.34	0.802	0.533	2.28	1.52	0.957	0.637	2.57	1.71	1.13	0.751
	42	2.23	1.48	0.858	0.571	2.52	1.67	1.03	0.683	2.84	1.89	1.21	0.807
	44	2.44	1.63	0.914	0.608	2.76	1.84	1.10	0.729	3.12	2.07	1.30	0.863
	46	2.67	1.78	0.970	0.645	3.02	2.01	1.16	0.775	3.41	2.27	1.38	0.919
	48	2.91	1.94	1.03	0.683	3.29	2.19	1.23	0.821	3.71	2.47	1.47	0.976
	50	3.16	2.10	1.08	0.720	3.57	2.37	1.30	0.867	4.02	2.68	1.55	1.03
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		2.30		1.53		2.58		1.72		2.90		1.93	
$t_y \times 10^3$, (kips)$^{-1}$		0.536		0.357		0.595		0.396		0.656		0.437	
$t_r \times 10^3$, (kips)$^{-1}$		0.659		0.439		0.731		0.488		0.806		0.537	
r_x/r_y		3.70				3.70				3.71			
r_y, in.		3.49				3.46				3.42			

[c] Shape is slender for compression for F_y = 50 ksi.

W30

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W30×											
		148^c				132^c				124^c			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.799	0.531	0.713	0.474	0.914	0.608	0.815	0.542	0.988	0.657	0.873	0.581
	11	0.986	0.656	0.765	0.509	1.13	0.753	0.882	0.587	1.23	0.817	0.949	0.631
	12	1.03	0.683	0.784	0.522	1.18	0.785	0.906	0.603	1.28	0.851	0.976	0.649
	13	1.08	0.718	0.804	0.535	1.23	0.820	0.931	0.620	1.34	0.890	1.00	0.668
	14	1.14	0.758	0.826	0.550	1.29	0.861	0.958	0.638	1.40	0.935	1.03	0.688
	15	1.21	0.804	0.849	0.565	1.37	0.915	0.987	0.657	1.48	0.985	1.07	0.710
	16	1.29	0.856	0.873	0.581	1.47	0.975	1.02	0.677	1.57	1.05	1.10	0.732
	17	1.38	0.915	0.898	0.598	1.57	1.04	1.05	0.699	1.69	1.12	1.14	0.757
	18	1.48	0.982	0.925	0.616	1.69	1.12	1.08	0.721	1.82	1.21	1.18	0.782
	19	1.59	1.06	0.954	0.635	1.82	1.21	1.12	0.746	1.97	1.31	1.22	0.810
	20	1.72	1.15	0.984	0.655	1.98	1.32	1.16	0.772	2.13	1.42	1.26	0.840
	22	2.05	1.36	1.05	0.700	2.36	1.57	1.25	0.831	2.55	1.70	1.36	0.907
	24	2.43	1.62	1.13	0.751	2.81	1.87	1.36	0.904	3.04	2.02	1.51	1.01
	26	2.86	1.90	1.25	0.828	3.30	2.19	1.54	1.02	3.57	2.37	1.72	1.14
	28	3.31	2.20	1.39	0.923	3.82	2.54	1.72	1.15	4.14	2.75	1.92	1.28
	30	3.80	2.53	1.53	1.02	4.39	2.92	1.91	1.27	4.75	3.16	2.13	1.42
	32	4.33	2.88	1.67	1.11	4.99	3.32	2.09	1.39	5.40	3.60	2.35	1.56
	34	4.89	3.25	1.82	1.21	5.64	3.75	2.28	1.52	6.10	4.06	2.56	1.70
	36	5.48	3.64	1.96	1.3	6.32	4.21	2.47	1.64	6.84	4.55	2.78	1.85

Other Constants and Properties

	148^c ASD	148^c LRFD	132^c ASD	132^c LRFD	124^c ASD	124^c LRFD
$b_y \times 10^3$, $(\text{kip-ft})^{-1}$	5.24	3.49	6.10	4.06	6.60	4.390
$t_y \times 10^3$, $(\text{kips})^{-1}$	0.766	0.510	0.861	0.573	0.915	0.609
$t_r \times 10^3$, $(\text{kips})^{-1}$	0.941	0.627	1.06	0.705	1.12	0.749
r_x / r_y	5.44		5.42		5.43	
r_y, in.	2.28		2.25		2.23	

[c] Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c / r_y equal to or greater than 200.

W30

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W30×											
		116^c				108^c				99^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	1.07	0.711	0.943	0.627	1.17	0.779	1.03	0.685	1.31	0.868	1.14	0.760
	11	1.34	0.889	1.03	0.686	1.47	0.981	1.14	0.755	1.66	1.10	1.27	0.846
	12	1.39	0.928	1.06	0.706	1.54	1.02	1.17	0.779	1.74	1.16	1.31	0.874
	13	1.46	0.972	1.09	0.728	1.62	1.08	1.21	0.804	1.82	1.21	1.36	0.903
	14	1.54	1.02	1.13	0.750	1.70	1.13	1.25	0.830	1.93	1.28	1.41	0.935
	15	1.62	1.08	1.16	0.775	1.80	1.20	1.29	0.859	2.04	1.36	1.46	0.969
	16	1.72	1.14	1.20	0.801	1.91	1.27	1.34	0.889	2.17	1.44	1.51	1.01
	17	1.84	1.23	1.24	0.828	2.04	1.35	1.39	0.922	2.31	1.54	1.57	1.04
	18	1.99	1.32	1.29	0.858	2.20	1.47	1.44	0.957	2.50	1.66	1.63	1.09
	19	2.16	1.44	1.34	0.890	2.40	1.60	1.50	0.995	2.73	1.81	1.7	1.13
	20	2.35	1.56	1.39	0.924	2.62	1.74	1.56	1.04	3.00	1.99	1.78	1.18
	22	2.83	1.88	1.51	1.00	3.16	2.11	1.7	1.13	3.63	2.41	2.00	1.33
	24	3.36	2.24	1.70	1.13	3.77	2.51	1.96	1.31	4.31	2.87	2.32	1.54
	26	3.95	2.63	1.94	1.29	4.42	2.94	2.24	1.49	5.06	3.37	2.65	1.76
	28	4.58	3.05	2.18	1.45	5.13	3.41	2.52	1.68	5.87	3.91	2.99	1.99
	30	5.26	3.50	2.42	1.61	5.88	3.91	2.81	1.87	6.74	4.49	3.34	2.22
	32	5.98	3.98	2.67	1.78	6.69	4.45	3.10	2.06	7.67	5.10	3.69	2.46
	34	6.75	4.49	2.92	1.94	7.56	5.03	3.40	2.26	8.66	5.76	4.06	2.70
	36	7.57	5.04	3.17	2.11								

Other Constants and Properties

	116^c ASD	116^c LRFD	108^c ASD	108^c LRFD	99^c ASD	99^c LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	7.24	4.82	8.12	5.40	9.23	6.140
$t_y \times 10^3$, (kips)$^{-1}$	0.977	0.650	1.05	0.701	1.15	0.766
$t_r \times 10^3$, (kips)$^{-1}$	1.20	0.800	1.290	0.863	1.41	0.943
r_x/r_y	5.48		5.53		5.57	
r_y, in.	2.19		2.15		2.10	

^{c} Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

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W30–W27

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W30×				W27×							
		$90^{c,v}$				539^{h}				368^{h}			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	1.49	0.990	1.26	0.838	0.210	0.140	0.189	0.125	0.306	0.204	0.287	0.191
	11	1.90	1.26	1.41	0.936	0.231	0.154	0.189	0.125	0.340	0.226	0.287	0.191
	12	1.99	1.32	1.45	0.968	0.235	0.157	0.189	0.125	0.347	0.231	0.287	0.191
	13	2.09	1.39	1.51	1.00	0.240	0.160	0.189	0.125	0.355	0.236	0.289	0.192
	14	2.21	1.47	1.56	1.04	0.245	0.163	0.190	0.126	0.363	0.242	0.291	0.194
	15	2.34	1.56	1.62	1.08	0.251	0.167	0.191	0.127	0.373	0.248	0.294	0.195
	16	2.49	1.65	1.68	1.12	0.257	0.171	0.192	0.128	0.383	0.255	0.296	0.197
	17	2.66	1.77	1.75	1.16	0.264	0.176	0.193	0.128	0.394	0.262	0.299	0.199
	18	2.85	1.90	1.82	1.21	0.271	0.181	0.194	0.129	0.406	0.270	0.301	0.200
	19	3.07	2.04	1.90	1.27	0.279	0.186	0.195	0.130	0.419	0.279	0.304	0.202
	20	3.34	2.22	1.99	1.32	0.288	0.192	0.196	0.131	0.434	0.289	0.306	0.204
	22	4.04	2.69	2.28	1.52	0.308	0.205	0.199	0.132	0.467	0.311	0.312	0.207
	24	4.80	3.20	2.65	1.76	0.331	0.220	0.201	0.134	0.506	0.336	0.317	0.211
	26	5.64	3.75	3.04	2.02	0.358	0.238	0.203	0.135	0.552	0.367	0.323	0.215
	28	6.54	4.35	3.44	2.29	0.390	0.260	0.206	0.137	0.606	0.403	0.329	0.219
	30	7.51	4.99	3.85	2.56	0.428	0.285	0.208	0.139	0.670	0.446	0.335	0.223
	32	8.54	5.68	4.27	2.84	0.472	0.314	0.211	0.140	0.746	0.497	0.342	0.227
	34	9.64	6.41	4.70	3.13	0.524	0.348	0.213	0.142	0.839	0.558	0.348	0.232
	36					0.586	0.390	0.216	0.144	0.941	0.626	0.355	0.236
	38					0.653	0.435	0.219	0.146	1.05	0.697	0.363	0.241
	40					0.724	0.481	0.222	0.148	1.16	0.773	0.370	0.246
	42					0.798	0.531	0.225	0.149	1.28	0.852	0.378	0.252
	44					0.876	0.583	0.228	0.151	1.41	0.935	0.386	0.257
	46					0.957	0.637	0.231	0.154	1.54	1.02	0.395	0.263
	48					1.04	0.693	0.234	0.156	1.67	1.11	0.404	0.269
	50					1.13	0.752	0.237	0.158	1.81	1.21	0.413	0.275
Other Constants and Properties													
$b_y \times 10^3$, $(\text{kip-ft})^{-1}$		10.3		6.83		0.815		0.542		1.28		0.850	
$t_y \times 10^3$, $(\text{kips})^{-1}$		1.27		0.845		0.210		0.140		0.306		0.204	
$t_r \times 10^3$, $(\text{kips})^{-1}$		1.56		1.04		0.258		0.172		0.376		0.251	
r_x/r_y		5.60				3.48				3.51			
r_y, in.		2.09				3.65				3.48			

[c] Shape is slender for compression for F_y = 50 ksi.

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(1) with F_y = 50 ksi; therefore, ϕ_v = 0.90 and Ω_v = 1.67.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W27

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W27×											
		336h				307h				281			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.337	0.224	0.315	0.210	0.370	0.246	0.346	0.230	0.402	0.267	0.381	0.253
	11	0.375	0.249	0.315	0.210	0.413	0.275	0.346	0.230	0.449	0.299	0.381	0.253
	12	0.382	0.254	0.315	0.210	0.422	0.281	0.346	0.230	0.459	0.305	0.381	0.253
	13	0.391	0.260	0.318	0.211	0.432	0.287	0.349	0.232	0.469	0.312	0.385	0.256
	14	0.400	0.266	0.320	0.213	0.442	0.294	0.353	0.235	0.481	0.320	0.389	0.259
	15	0.411	0.273	0.323	0.215	0.454	0.302	0.356	0.237	0.494	0.329	0.393	0.262
	16	0.422	0.281	0.326	0.217	0.467	0.311	0.360	0.239	0.508	0.338	0.397	0.264
	17	0.435	0.289	0.329	0.219	0.481	0.320	0.364	0.242	0.524	0.348	0.402	0.267
	18	0.448	0.298	0.332	0.221	0.497	0.330	0.367	0.244	0.541	0.360	0.406	0.270
	19	0.463	0.308	0.336	0.223	0.513	0.342	0.371	0.247	0.559	0.372	0.411	0.273
	20	0.480	0.319	0.339	0.225	0.532	0.354	0.375	0.250	0.580	0.386	0.416	0.277
	22	0.517	0.344	0.345	0.230	0.574	0.382	0.383	0.255	0.626	0.417	0.426	0.283
	24	0.560	0.373	0.352	0.234	0.624	0.415	0.392	0.261	0.681	0.453	0.436	0.290
	26	0.612	0.407	0.359	0.239	0.683	0.454	0.401	0.267	0.747	0.497	0.447	0.297
	28	0.674	0.448	0.367	0.244	0.753	0.501	0.410	0.273	0.824	0.548	0.458	0.305
	30	0.746	0.497	0.375	0.249	0.836	0.557	0.420	0.279	0.917	0.610	0.470	0.313
	32	0.833	0.554	0.383	0.255	0.936	0.623	0.430	0.286	1.03	0.683	0.482	0.321
	34	0.938	0.624	0.391	0.260	1.06	0.703	0.441	0.293	1.16	0.772	0.496	0.330
	36	1.05	0.700	0.400	0.266	1.18	0.788	0.452	0.301	1.30	0.865	0.510	0.339
	38	1.17	0.780	0.409	0.272	1.32	0.878	0.464	0.309	1.45	0.964	0.524	0.349
	40	1.30	0.864	0.419	0.279	1.46	0.972	0.476	0.317	1.61	1.07	0.540	0.359
	42	1.43	0.952	0.429	0.285	1.61	1.07	0.490	0.326	1.77	1.18	0.557	0.370
	44	1.57	1.05	0.439	0.292	1.77	1.18	0.504	0.335	1.94	1.29	0.574	0.382
	46	1.72	1.14	0.451	0.300	1.93	1.29	0.518	0.345	2.12	1.41	0.593	0.395
	48	1.87	1.24	0.462	0.308	2.10	1.40	0.534	0.355	2.31	1.54	0.614	0.408
	50	2.03	1.35	0.475	0.316	2.28	1.52	0.551	0.367	2.51	1.67	0.639	0.425

Other Constants and Properties

	336h ASD	336h LRFD	307h ASD	307h LRFD	281 ASD	281 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	1.41	0.941	1.57	1.04	1.73	1.15
$t_y \times 10^3$, (kips)$^{-1}$	0.337	0.224	0.370	0.246	0.402	0.267
$t_r \times 10^3$, (kips)$^{-1}$	0.414	0.276	0.455	0.303	0.494	0.329
r_x / r_y	3.51		3.52		3.54	
r_y, in.	3.45		3.41		3.39	

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

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W27

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W27×											
		258				235				217			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.439	0.292	0.418	0.278	0.481	0.320	0.461	0.307	0.523	0.348	0.501	0.333
	11	0.491	0.327	0.418	0.278	0.540	0.359	0.461	0.307	0.587	0.390	0.501	0.333
	12	0.502	0.334	0.419	0.279	0.552	0.367	0.463	0.308	0.600	0.399	0.503	0.335
	13	0.514	0.342	0.424	0.282	0.565	0.376	0.469	0.312	0.614	0.409	0.510	0.339
	14	0.527	0.351	0.429	0.285	0.580	0.386	0.475	0.316	0.630	0.419	0.517	0.344
	15	0.541	0.360	0.434	0.289	0.596	0.396	0.481	0.320	0.648	0.431	0.524	0.348
	16	0.557	0.371	0.439	0.292	0.614	0.408	0.487	0.324	0.667	0.444	0.531	0.353
	17	0.575	0.382	0.444	0.296	0.633	0.421	0.494	0.328	0.689	0.458	0.538	0.358
	18	0.594	0.395	0.450	0.299	0.655	0.436	0.500	0.333	0.712	0.474	0.546	0.363
	19	0.615	0.409	0.455	0.303	0.678	0.451	0.507	0.337	0.738	0.491	0.554	0.369
	20	0.637	0.424	0.461	0.307	0.704	0.468	0.514	0.342	0.766	0.510	0.562	0.374
	22	0.689	0.459	0.473	0.315	0.762	0.507	0.529	0.352	0.830	0.552	0.579	0.385
	24	0.751	0.500	0.485	0.323	0.832	0.553	0.544	0.362	0.906	0.603	0.597	0.398
	26	0.824	0.549	0.498	0.332	0.914	0.608	0.560	0.373	0.997	0.663	0.617	0.410
	28	0.912	0.607	0.512	0.341	1.01	0.674	0.578	0.384	1.11	0.735	0.637	0.424
	30	1.02	0.676	0.527	0.351	1.13	0.753	0.596	0.397	1.23	0.822	0.660	0.439
	32	1.14	0.760	0.543	0.361	1.27	0.848	0.616	0.410	1.39	0.927	0.683	0.455
	34	1.29	0.858	0.559	0.372	1.44	0.957	0.637	0.424	1.57	1.05	0.709	0.471
	36	1.45	0.962	0.577	0.384	1.61	1.07	0.660	0.439	1.76	1.17	0.736	0.490
	38	1.61	1.07	0.596	0.396	1.80	1.20	0.684	0.455	1.96	1.31	0.766	0.509
	40	1.78	1.19	0.616	0.410	1.99	1.33	0.710	0.472	2.18	1.45	0.798	0.531
	42	1.97	1.31	0.637	0.424	2.20	1.46	0.738	0.491	2.40	1.60	0.842	0.560
	44	2.16	1.44	0.660	0.439	2.41	1.60	0.776	0.516	2.63	1.75	0.892	0.593
	46	2.36	1.57	0.685	0.456	2.63	1.75	0.818	0.544	2.88	1.91	0.942	0.627
	48	2.57	1.71	0.721	0.479	2.87	1.91	0.861	0.573	3.13	2.09	0.992	0.660
	50	2.79	1.85	0.756	0.503	3.11	2.07	0.904	0.601	3.40	2.26	1.040	0.693

Other Constants and Properties

	258		235		217	
$b_y \times 10^3$, (kip-ft)$^{-1}$	1.91	1.27	2.12	1.41	2.31	1.540
$t_y \times 10^3$, (kips)$^{-1}$	0.439	0.292	0.481	0.320	0.523	0.348
$t_r \times 10^3$, (kips)$^{-1}$	0.539	0.359	0.591	0.394	0.642	0.428
r_x/r_y	3.54		3.54		3.55	
r_y, in.	3.36		3.33		3.32	

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W27

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape	W27×											
	194				178				161[c]			
Design	$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending												
0	0.585	0.389	0.565	0.376	0.636	0.423	0.625	0.416	0.703	0.467	0.692	0.460
11	0.658	0.438	0.565	0.376	0.718	0.478	0.625	0.416	0.793	0.527	0.692	0.460
12	0.673	0.448	0.568	0.378	0.734	0.489	0.630	0.419	0.811	0.540	0.698	0.465
13	0.689	0.459	0.576	0.383	0.753	0.501	0.640	0.426	0.832	0.554	0.710	0.472
14	0.708	0.471	0.584	0.389	0.773	0.515	0.650	0.432	0.855	0.569	0.722	0.480
15	0.728	0.484	0.593	0.395	0.796	0.530	0.661	0.439	0.881	0.586	0.735	0.489
16	0.750	0.499	0.602	0.401	0.821	0.546	0.671	0.447	0.909	0.604	0.747	0.497
17	0.775	0.516	0.612	0.407	0.849	0.565	0.683	0.454	0.939	0.625	0.761	0.506
18	0.802	0.533	0.621	0.413	0.879	0.585	0.694	0.462	0.973	0.647	0.775	0.515
19	0.831	0.553	0.631	0.420	0.912	0.607	0.706	0.470	1.01	0.672	0.789	0.525
20	0.863	0.574	0.641	0.427	0.948	0.631	0.718	0.478	1.05	0.699	0.804	0.535
22	0.937	0.623	0.663	0.441	1.03	0.686	0.745	0.495	1.14	0.761	0.835	0.556
24	1.02	0.682	0.686	0.456	1.13	0.752	0.773	0.514	1.25	0.835	0.869	0.578
26	1.13	0.751	0.711	0.473	1.25	0.830	0.803	0.534	1.39	0.924	0.906	0.603
28	1.25	0.834	0.737	0.490	1.39	0.925	0.836	0.556	1.55	1.03	0.946	0.630
30	1.40	0.934	0.766	0.509	1.56	1.04	0.871	0.580	1.74	1.16	0.990	0.659
32	1.59	1.06	0.797	0.530	1.77	1.18	0.910	0.606	1.98	1.31	1.04	0.691
34	1.79	1.19	0.830	0.552	2.00	1.33	0.952	0.634	2.23	1.48	1.09	0.726
36	2.01	1.34	0.867	0.577	2.24	1.49	0.999	0.665	2.50	1.66	1.17	0.781
38	2.24	1.49	0.906	0.603	2.49	1.66	1.07	0.713	2.79	1.85	1.27	0.844
40	2.48	1.65	0.968	0.644	2.76	1.84	1.15	0.765	3.09	2.05	1.36	0.907
42	2.73	1.82	1.03	0.687	3.05	2.03	1.23	0.817	3.40	2.26	1.46	0.970
44	3.00	2.00	1.10	0.729	3.34	2.23	1.31	0.869	3.73	2.48	1.55	1.03
46	3.28	2.18	1.16	0.771	3.66	2.43	1.38	0.920	4.08	2.72	1.65	1.10
48	3.57	2.38	1.22	0.813	3.98	2.65	1.46	0.972	4.44	2.96	1.74	1.16
50	3.88	2.58	1.29	0.855	4.32	2.87	1.54	1.02	4.82	3.21	1.84	1.22

Other Constants and Properties

	194 ASD	194 LRFD	178 ASD	178 LRFD	161[c] ASD	161[c] LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	2.62	1.74	2.92	1.94	3.27	2.17
$t_y \times 10^3$, (kips)$^{-1}$	0.585	0.389	0.636	0.423	0.702	0.467
$t_r \times 10^3$, (kips)$^{-1}$	0.718	0.479	0.781	0.521	0.862	0.575
r_x/r_y	3.56		3.57		3.56	
r_y, in.	3.29		3.25		3.23	

[c] Shape is slender for compression for F_y = 50 ksi.

W27

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W27×											
		146^c				129^c				114^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.790	0.526	0.768	0.511	0.908	0.604	0.902	0.600	1.04	0.694	1.04	0.691
	11	0.882	0.587	0.768	0.511	1.15	0.763	0.976	0.649	1.31	0.873	1.13	0.754
	12	0.901	0.599	0.777	0.517	1.21	0.802	1.00	0.666	1.37	0.912	1.17	0.775
	13	0.921	0.613	0.791	0.526	1.27	0.846	1.03	0.684	1.45	0.962	1.20	0.798
	14	0.946	0.629	0.805	0.535	1.35	0.897	1.06	0.703	1.53	1.02	1.24	0.822
	15	0.974	0.648	0.819	0.545	1.44	0.955	1.09	0.723	1.64	1.09	1.27	0.847
	16	1.01	0.669	0.835	0.555	1.53	1.02	1.12	0.744	1.75	1.17	1.31	0.874
	17	1.04	0.692	0.850	0.566	1.65	1.10	1.15	0.767	1.89	1.25	1.36	0.903
	18	1.08	0.718	0.867	0.577	1.78	1.18	1.19	0.791	2.04	1.36	1.40	0.934
	19	1.12	0.746	0.884	0.588	1.92	1.28	1.23	0.816	2.21	1.47	1.45	0.967
	20	1.17	0.776	0.901	0.600	2.09	1.39	1.27	0.843	2.41	1.60	1.51	1.00
	22	1.27	0.846	0.939	0.625	2.51	1.67	1.36	0.903	2.90	1.93	1.63	1.08
	24	1.40	0.930	0.980	0.652	2.99	1.99	1.46	0.973	3.46	2.30	1.80	1.20
	26	1.55	1.03	1.02	0.681	3.51	2.33	1.64	1.09	4.06	2.70	2.04	1.36
	28	1.73	1.15	1.07	0.714	4.07	2.71	1.82	1.21	4.70	3.13	2.27	1.51
	30	1.95	1.30	1.13	0.750	4.67	3.11	2.00	1.33	5.40	3.59	2.51	1.67
	32	2.22	1.48	1.19	0.789	5.31	3.54	2.18	1.45	6.14	4.09	2.75	1.83
	34	2.50	1.67	1.27	0.843	6.00	3.99	2.36	1.57	6.94	4.61	2.99	1.99
	36	2.81	1.87	1.38	0.919	6.73	4.47	2.54	1.69	7.78	5.17	3.23	2.15
	38	3.13	2.08	1.50	0.995								
	40	3.47	2.31	1.61	1.07								
	42	3.82	2.54	1.73	1.15								
	44	4.19	2.79	1.84	1.23								
	46	4.58	3.05	1.96	1.30								
	48	4.99	3.32	2.07	1.38								
	50	5.41	3.60	2.19	1.46								

Other Constants and Properties

	146^c ASD	146^c LRFD	129^c ASD	129^c LRFD	114^c ASD	114^c LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	3.65	2.43	6.19	4.12	7.23	4.81
$t_y \times 10^3$, (kips)$^{-1}$	0.773	0.514	0.884	0.588	0.994	0.661
$t_r \times 10^3$, (kips)$^{-1}$	0.950	0.633	1.09	0.724	1.22	0.814
r_x/r_y	3.59		5.07		5.05	
r_y, in.	3.20		2.21		2.18	

^{c} Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

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W27

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		**W27×**											
		102^c				**94^c**				**84^c**			
Design		**$p \times 10^3$ (kips)$^{-1}$**		**$b_x \times 10^3$ (kip-ft)$^{-1}$**		**$p \times 10^3$ (kips)$^{-1}$**		**$b_x \times 10^3$ (kip-ft)$^{-1}$**		**$p \times 10^3$ (kips)$^{-1}$**		**$b_x \times 10^3$ (kip-ft)$^{-1}$**	
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	**0**	1.20	0.802	1.17	0.777	1.33	0.887	1.28	0.853	1.53	1.02	1.46	0.971
	11	1.52	1.01	1.28	0.854	1.70	1.13	1.42	0.944	1.96	1.30	1.63	1.09
	12	1.59	1.06	1.32	0.880	1.78	1.18	1.46	0.974	2.06	1.37	1.69	1.12
	13	1.67	1.11	1.36	0.907	1.87	1.24	1.51	1.01	2.17	1.44	1.75	1.16
	14	1.76	1.17	1.41	0.935	1.97	1.31	1.56	1.04	2.29	1.52	1.81	1.20
	15	1.87	1.24	1.45	0.966	2.09	1.39	1.62	1.07	2.43	1.62	1.88	1.25
	16	1.99	1.33	1.50	0.999	2.22	1.48	1.67	1.11	2.59	1.73	1.95	1.30
	17	2.15	1.43	1.55	1.03	2.38	1.58	1.74	1.15	2.78	1.85	2.03	1.35
	18	2.33	1.55	1.61	1.07	2.59	1.72	1.80	1.20	3.00	1.99	2.11	1.41
	19	2.53	1.69	1.67	1.11	2.82	1.88	1.88	1.25	3.28	2.18	2.21	1.47
	20	2.77	1.84	1.74	1.16	3.09	2.06	1.95	1.30	3.62	2.41	2.31	1.53
	22	3.34	2.22	1.89	1.25	3.74	2.49	2.16	1.44	4.38	2.91	2.64	1.76
	24	3.98	2.65	2.15	1.43	4.45	2.96	2.50	1.66	5.21	3.47	3.06	2.04
	26	4.67	3.11	2.44	1.63	5.22	3.47	2.84	1.89	6.12	4.07	3.49	2.32
	28	5.42	3.60	2.74	1.82	6.06	4.03	3.19	2.12	7.10	4.72	3.93	2.62
	30	6.22	4.14	3.03	2.02	6.95	4.62	3.54	2.36	8.15	5.42	4.38	2.92
	32	7.07	4.71	3.33	2.22	7.91	5.26	3.90	2.59	9.27	6.17	4.84	3.22
	34	7.99	5.31	3.63	2.42	8.93	5.94	4.26	2.83	10.5	6.96	5.31	3.53
	36												
	38												
	40												
	42												
	44												
	46												
	48												
	50												

Other Constants and Properties

	102 ASD	102 LRFD	94 ASD	94 LRFD	84 ASD	84 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	8.21	5.46	9.18	6.11	10.7	7.14
$t_y \times 10^3$, (kips)$^{-1}$	1.11	0.741	1.21	0.805	1.35	0.900
$t_r \times 10^3$, (kips)$^{-1}$	1.37	0.912	1.49	0.991	1.66	1.11
r_x/r_y	5.12		5.14		5.17	
r_y, in.	2.15		2.12		2.07	

^{c} Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W24

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W24×											
		370^h				335^h				306^h			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.306	0.204	0.315	0.210	0.340	0.226	0.349	0.232	0.372	0.248	0.386	0.257
	11	0.345	0.230	0.315	0.210	0.384	0.255	0.349	0.232	0.422	0.281	0.386	0.257
	12	0.353	0.235	0.316	0.210	0.393	0.261	0.351	0.233	0.432	0.287	0.389	0.259
	13	0.362	0.241	0.319	0.212	0.403	0.268	0.354	0.235	0.443	0.295	0.392	0.261
	14	0.372	0.247	0.321	0.213	0.414	0.276	0.357	0.237	0.455	0.303	0.396	0.263
	15	0.382	0.254	0.323	0.215	0.426	0.284	0.359	0.239	0.469	0.312	0.399	0.266
	16	0.394	0.262	0.326	0.217	0.440	0.293	0.362	0.241	0.484	0.322	0.403	0.268
	17	0.407	0.271	0.328	0.218	0.455	0.303	0.365	0.243	0.501	0.333	0.406	0.270
	18	0.422	0.280	0.330	0.220	0.471	0.314	0.368	0.245	0.520	0.346	0.410	0.273
	19	0.437	0.291	0.333	0.221	0.489	0.325	0.371	0.247	0.540	0.359	0.414	0.275
	20	0.454	0.302	0.335	0.223	0.509	0.338	0.375	0.249	0.562	0.374	0.418	0.278
	22	0.494	0.328	0.340	0.226	0.554	0.368	0.381	0.254	0.612	0.407	0.426	0.283
	24	0.540	0.359	0.346	0.230	0.608	0.404	0.388	0.258	0.673	0.448	0.434	0.289
	26	0.596	0.397	0.351	0.234	0.672	0.447	0.395	0.263	0.746	0.496	0.442	0.294
	28	0.663	0.441	0.357	0.237	0.750	0.499	0.402	0.267	0.834	0.555	0.451	0.300
	30	0.743	0.495	0.363	0.241	0.843	0.561	0.409	0.272	0.939	0.625	0.461	0.306
	32	0.842	0.560	0.369	0.245	0.957	0.636	0.417	0.277	1.07	0.711	0.470	0.313
	34	0.950	0.632	0.375	0.249	1.08	0.718	0.425	0.283	1.21	0.802	0.480	0.320
	36	1.07	0.709	0.381	0.254	1.21	0.806	0.433	0.288	1.35	0.899	0.491	0.327
	38	1.19	0.790	0.388	0.258	1.35	0.897	0.442	0.294	1.51	1.00	0.502	0.334
	40	1.32	0.875	0.395	0.263	1.49	0.994	0.451	0.300	1.67	1.11	0.513	0.341
	42	1.45	0.965	0.402	0.267	1.65	1.10	0.460	0.306	1.84	1.22	0.525	0.349
	44	1.59	1.06	0.409	0.272	1.81	1.20	0.470	0.313	2.02	1.34	0.538	0.358
	46	1.74	1.16	0.417	0.277	1.98	1.32	0.480	0.319	2.21	1.47	0.551	0.367
	48	1.89	1.26	0.425	0.283	2.15	1.43	0.491	0.326	2.40	1.60	0.565	0.376
	50	2.05	1.37	0.433	0.288	2.34	1.55	0.502	0.334	2.61	1.73	0.579	0.386
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		1.33		0.888		1.50		0.996		1.66		1.11	
$t_y \times 10^3$, (kips)$^{-1}$		0.306		0.204		0.340		0.226		0.372		0.248	
$t_r \times 10^3$, (kips)$^{-1}$		0.376		0.251		0.417		0.278		0.457		0.305	
r_x/r_y		3.39				3.41				3.41			
r_y, in.		3.27				3.23				3.20			

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

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W24

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W24×											
		279[h]				250				229			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.408	0.271	0.427	0.284	0.454	0.302	0.479	0.319	0.497	0.331	0.528	0.351
	11	0.463	0.308	0.427	0.284	0.517	0.344	0.479	0.319	0.567	0.377	0.528	0.351
	12	0.474	0.316	0.430	0.286	0.530	0.353	0.483	0.322	0.581	0.387	0.534	0.355
	13	0.487	0.324	0.434	0.289	0.544	0.362	0.489	0.325	0.597	0.397	0.540	0.359
	14	0.501	0.333	0.438	0.292	0.560	0.373	0.494	0.329	0.615	0.409	0.547	0.364
	15	0.516	0.343	0.443	0.294	0.578	0.384	0.499	0.332	0.635	0.422	0.553	0.368
	16	0.533	0.355	0.447	0.297	0.597	0.397	0.505	0.336	0.657	0.437	0.560	0.372
	17	0.552	0.367	0.451	0.300	0.619	0.412	0.510	0.340	0.681	0.453	0.567	0.377
	18	0.573	0.381	0.456	0.303	0.642	0.427	0.516	0.343	0.707	0.471	0.574	0.382
	19	0.595	0.396	0.461	0.306	0.668	0.445	0.522	0.347	0.736	0.490	0.581	0.387
	20	0.620	0.413	0.465	0.310	0.697	0.463	0.528	0.351	0.768	0.511	0.588	0.391
	22	0.677	0.451	0.475	0.316	0.762	0.507	0.541	0.360	0.842	0.560	0.604	0.402
	24	0.746	0.496	0.485	0.323	0.841	0.559	0.554	0.368	0.930	0.619	0.620	0.412
	26	0.828	0.551	0.496	0.330	0.935	0.622	0.567	0.378	1.04	0.690	0.637	0.424
	28	0.927	0.617	0.507	0.337	1.05	0.698	0.582	0.387	1.17	0.776	0.655	0.436
	30	1.05	0.697	0.519	0.345	1.19	0.792	0.597	0.397	1.33	0.883	0.674	0.448
	32	1.19	0.793	0.531	0.353	1.35	0.901	0.613	0.408	1.51	1.00	0.694	0.462
	34	1.35	0.895	0.544	0.362	1.53	1.02	0.630	0.419	1.70	1.13	0.716	0.476
	36	1.51	1.00	0.557	0.371	1.71	1.14	0.648	0.431	1.91	1.27	0.739	0.491
	38	1.68	1.12	0.571	0.380	1.91	1.27	0.667	0.444	2.13	1.42	0.763	0.508
	40	1.86	1.24	0.586	0.390	2.12	1.41	0.687	0.457	2.36	1.57	0.789	0.525
	42	2.05	1.37	0.601	0.400	2.33	1.55	0.708	0.471	2.60	1.73	0.817	0.544
	44	2.25	1.50	0.618	0.411	2.56	1.70	0.731	0.486	2.85	1.90	0.847	0.563
	46	2.46	1.64	0.635	0.423	2.80	1.86	0.755	0.502	3.12	2.08	0.884	0.588
	48	2.68	1.78	0.653	0.435	3.05	2.03	0.781	0.519	3.40	2.26	0.928	0.617
	50	2.91	1.94	0.673	0.448	3.31	2.20	0.814	0.541	3.68	2.45	0.971	0.646
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		1.85		1.23		2.08		1.39		2.31		1.54	
$t_y \times 10^3$, (kips)$^{-1}$		0.408		0.271		0.454		0.302		0.497		0.331	
$t_r \times 10^3$, (kips)$^{-1}$		0.501		0.334		0.558		0.372		0.611		0.407	
r_x/r_y		3.41				3.41				3.44			
r_y, in.		3.17				3.14				3.11			

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

W24

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		**W24×**											
		207				**192**				**176**			
Design		**$p \times 10^3$ (kips)$^{-1}$**		**$b_x \times 10^3$ (kip-ft)$^{-1}$**		**$p \times 10^3$ (kips)$^{-1}$**		**$b_x \times 10^3$ (kip-ft)$^{-1}$**		**$p \times 10^3$ (kips)$^{-1}$**		**$b_x \times 10^3$ (kip-ft)$^{-1}$**	
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	**0**	0.550	0.366	0.588	0.391	0.591	0.393	0.637	0.424	0.646	0.430	0.697	0.464
	11	0.629	0.419	0.589	0.392	0.677	0.450	0.639	0.425	0.742	0.493	0.700	0.466
	12	0.646	0.430	0.596	0.397	0.694	0.462	0.647	0.431	0.761	0.506	0.710	0.472
	13	0.664	0.442	0.604	0.402	0.714	0.475	0.656	0.437	0.783	0.521	0.721	0.479
	14	0.684	0.455	0.612	0.407	0.736	0.490	0.665	0.443	0.808	0.537	0.731	0.487
	15	0.706	0.470	0.620	0.412	0.760	0.506	0.675	0.449	0.835	0.555	0.743	0.494
	16	0.731	0.486	0.628	0.418	0.787	0.524	0.684	0.455	0.865	0.575	0.754	0.502
	17	0.758	0.505	0.637	0.424	0.816	0.543	0.694	0.462	0.898	0.597	0.766	0.510
	18	0.788	0.525	0.646	0.429	0.849	0.565	0.705	0.469	0.934	0.622	0.778	0.518
	19	0.821	0.547	0.655	0.435	0.885	0.589	0.715	0.476	0.975	0.649	0.791	0.526
	20	0.858	0.571	0.664	0.442	0.924	0.615	0.726	0.483	1.02	0.678	0.804	0.535
	22	0.942	0.626	0.683	0.454	1.02	0.675	0.749	0.498	1.12	0.746	0.832	0.553
	24	1.04	0.694	0.704	0.468	1.13	0.749	0.773	0.514	1.25	0.829	0.861	0.573
	26	1.17	0.775	0.725	0.483	1.26	0.837	0.799	0.532	1.40	0.928	0.893	0.594
	28	1.31	0.874	0.749	0.498	1.42	0.944	0.827	0.550	1.58	1.05	0.927	0.617
	30	1.50	0.996	0.773	0.514	1.62	1.08	0.857	0.570	1.80	1.20	0.964	0.641
	32	1.70	1.13	0.800	0.532	1.84	1.23	0.888	0.591	2.05	1.37	1.00	0.668
	34	1.92	1.28	0.828	0.551	2.08	1.38	0.923	0.614	2.32	1.54	1.05	0.697
	36	2.16	1.43	0.858	0.571	2.33	1.55	0.960	0.639	2.60	1.73	1.09	0.728
	38	2.40	1.60	0.891	0.593	2.60	1.73	1.00	0.666	2.90	1.93	1.15	0.767
	40	2.66	1.77	0.926	0.616	2.88	1.92	1.05	0.697	3.21	2.13	1.23	0.818
	42	2.93	1.95	0.967	0.643	3.17	2.11	1.11	0.740	3.54	2.35	1.31	0.869
	44	3.22	2.14	1.02	0.679	3.48	2.32	1.17	0.782	3.88	2.58	1.38	0.920
	46	3.52	2.34	1.07	0.715	3.81	2.53	1.24	0.824	4.24	2.82	1.46	0.970
	48	3.83	2.55	1.13	0.751	4.15	2.76	1.30	0.866	4.62	3.07	1.53	1.02
	50	4.16	2.77	1.18	0.787	4.50	2.99	1.36	0.908	5.01	3.34	1.61	1.07
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		2.60		1.73		2.83		1.88		3.10		2.06	
$t_y \times 10^3$, (kips)$^{-1}$		0.550		0.366		0.591		0.393		0.646		0.430	
$t_r \times 10^3$, (kips)$^{-1}$		0.676		0.451		0.726		0.484		0.794		0.529	
r_x / r_y		3.44				3.42				3.45			
r_y, in.		3.08				3.07				3.04			

W24

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W24×											
		162				146				131			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	**0**	0.699	0.465	0.761	0.506	0.777	0.517	0.852	0.567	0.865	0.576	0.963	0.641
	11	0.801	0.533	0.764	0.508	0.894	0.595	0.857	0.571	1.00	0.665	0.972	0.646
	12	0.822	0.547	0.776	0.516	0.918	0.611	0.872	0.580	1.03	0.684	0.989	0.658
	13	0.846	0.563	0.788	0.524	0.945	0.629	0.887	0.590	1.06	0.704	1.01	0.670
	14	0.872	0.580	0.801	0.533	0.975	0.649	0.902	0.600	1.09	0.727	1.03	0.683
	15	0.901	0.600	0.814	0.541	1.01	0.671	0.918	0.611	1.13	0.753	1.05	0.696
	16	0.934	0.621	0.827	0.550	1.05	0.696	0.935	0.622	1.17	0.781	1.07	0.710
	17	0.969	0.645	0.841	0.560	1.09	0.723	0.952	0.633	1.22	0.813	1.09	0.724
	18	1.01	0.671	0.855	0.569	1.13	0.753	0.970	0.645	1.27	0.848	1.11	0.739
	19	1.05	0.700	0.870	0.579	1.18	0.786	0.988	0.657	1.33	0.886	1.13	0.754
	20	1.10	0.731	0.886	0.589	1.24	0.823	1.01	0.670	1.39	0.928	1.16	0.770
	22	1.21	0.804	0.918	0.611	1.36	0.907	1.05	0.697	1.54	1.03	1.21	0.804
	24	1.34	0.892	0.953	0.634	1.52	1.01	1.09	0.727	1.72	1.14	1.26	0.841
	26	1.50	0.999	0.991	0.660	1.70	1.13	1.14	0.759	1.94	1.29	1.33	0.882
	28	1.70	1.13	1.03	0.687	1.93	1.29	1.19	0.794	2.21	1.47	1.39	0.928
	30	1.94	1.29	1.08	0.716	2.21	1.47	1.25	0.832	2.53	1.68	1.47	0.977
	32	2.21	1.47	1.13	0.749	2.52	1.68	1.31	0.874	2.88	1.92	1.56	1.04
	34	2.49	1.66	1.18	0.784	2.84	1.89	1.39	0.926	3.25	2.16	1.70	1.13
	36	2.79	1.86	1.24	0.826	3.19	2.12	1.50	1.00	3.65	2.43	1.84	1.23
	38	3.11	2.07	1.33	0.886	3.55	2.36	1.62	1.08	4.06	2.70	1.99	1.32
	40	3.45	2.29	1.42	0.947	3.93	2.62	1.73	1.15	4.50	3.00	2.13	1.42
	42	3.80	2.53	1.51	1.01	4.34	2.89	1.85	1.23	4.96	3.30	2.28	1.52
	44	4.17	2.78	1.60	1.07	4.76	3.17	1.96	1.30	5.45	3.62	2.42	1.61
	46	4.56	3.03	1.69	1.13	5.20	3.46	2.07	1.38	5.95	3.96	2.57	1.71
	48	4.96	3.30	1.78	1.19	5.67	3.77	2.19	1.45	6.48	4.31	2.71	1.80
	50	5.39	3.58	1.87	1.25	6.15	4.09	2.30	1.53				
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		3.39		2.26		3.82		2.54		4.37		2.91	
$t_y \times 10^3$, (kips)$^{-1}$		0.699		0.465		0.777		0.517		0.865		0.576	
$t_r \times 10^3$, (kips)$^{-1}$		0.858		0.572		0.954		0.636		1.06		0.709	
r_x/r_y		3.41				3.42				3.43			
r_y, in.		3.05				3.01				2.97			

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W24

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape	W24×											
	117^c				104^c				103^c			
Design	$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	0.992	0.660	1.09	0.725	1.14	0.757	1.23	0.820	1.13	0.751	1.27	0.847
11	1.13	0.751	1.10	0.733	1.30	0.864	1.25	0.832	1.52	1.01	1.42	0.944
12	1.16	0.770	1.12	0.748	1.33	0.886	1.28	0.849	1.62	1.08	1.46	0.972
13	1.19	0.794	1.15	0.762	1.37	0.910	1.30	0.867	1.73	1.15	1.51	1.00
14	1.23	0.820	1.17	0.778	1.41	0.938	1.33	0.886	1.86	1.23	1.55	1.03
15	1.28	0.850	1.19	0.794	1.45	0.968	1.36	0.905	2.00	1.33	1.61	1.07
16	1.33	0.882	1.22	0.810	1.50	1.00	1.39	0.925	2.18	1.45	1.66	1.10
17	1.38	0.919	1.24	0.828	1.56	1.04	1.42	0.946	2.38	1.58	1.72	1.14
18	1.44	0.959	1.27	0.846	1.63	1.08	1.46	0.969	2.61	1.74	1.78	1.19
19	1.51	1.00	1.30	0.865	1.70	1.13	1.49	0.992	2.88	1.92	1.85	1.23
20	1.58	1.05	1.33	0.885	1.79	1.19	1.53	1.02	3.19	2.12	1.92	1.28
22	1.75	1.16	1.39	0.927	1.99	1.32	1.61	1.07	3.86	2.57	2.09	1.39
24	1.96	1.30	1.46	0.974	2.23	1.48	1.69	1.13	4.60	3.06	2.37	1.58
26	2.21	1.47	1.54	1.03	2.52	1.68	1.79	1.19	5.40	3.59	2.65	1.77
28	2.53	1.68	1.63	1.08	2.89	1.92	1.90	1.27	6.26	4.16	2.94	1.95
30	2.90	1.93	1.73	1.15	3.32	2.21	2.06	1.37	7.19	4.78	3.22	2.14
32	3.30	2.20	1.89	1.26	3.77	2.51	2.29	1.52	8.18	5.44	3.50	2.33
34	3.72	2.48	2.07	1.38	4.26	2.83	2.51	1.67				
36	4.18	2.78	2.25	1.50	4.78	3.18	2.74	1.82				
38	4.65	3.10	2.43	1.62	5.32	3.54	2.97	1.98				
40	5.16	3.43	2.62	1.74	5.90	3.92	3.20	2.13				
42	5.68	3.78	2.80	1.86	6.50	4.33	3.44	2.29				
44	6.24	4.15	2.98	1.99	7.13	4.75	3.67	2.44				
46	6.82	4.54	3.17	2.11	7.80	5.19	3.91	2.60				
48	7.42	4.94	3.35	2.23	8.49	5.65	4.14	2.76				

Other Constants and Properties

	117^c ASD	117^c LRFD	104^c ASD	104^c LRFD	103^c ASD	103^c LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	4.99	3.32	5.71	3.80	8.58	5.71
$t_y \times 10^3$, (kips)$^{-1}$	0.971	0.646	1.09	0.724	1.10	0.733
$t_r \times 10^3$, (kips)$^{-1}$	1.19	0.795	1.34	0.891	1.35	0.903
r_x / r_y	3.44		3.47		5.03	
r_y, in.	2.94		2.91		1.99	

^{c} Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c / r_y equal to or greater than 200.

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W24

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W24×											
		94^c				84^c				76^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	1.26	0.838	1.40	0.933	1.45	0.965	1.59	1.06	1.63	1.09	1.78	1.19
	6	1.37	0.910	1.40	0.933	1.58	1.05	1.59	1.06	1.78	1.19	1.78	1.19
	7	1.41	0.938	1.40	0.933	1.63	1.08	1.60	1.06	1.84	1.23	1.79	1.19
	8	1.46	0.971	1.44	0.960	1.69	1.12	1.64	1.09	1.91	1.27	1.85	1.23
	9	1.52	1.01	1.48	0.987	1.76	1.17	1.69	1.13	1.99	1.32	1.91	1.27
	10	1.59	1.06	1.53	1.02	1.84	1.22	1.75	1.16	2.09	1.39	1.97	1.31
	11	1.67	1.11	1.57	1.05	1.93	1.29	1.80	1.20	2.19	1.46	2.04	1.36
	12	1.78	1.18	1.62	1.08	2.04	1.36	1.87	1.24	2.32	1.54	2.11	1.41
	13	1.90	1.26	1.68	1.12	2.17	1.44	1.93	1.28	2.47	1.64	2.19	1.46
	14	2.04	1.36	1.73	1.15	2.33	1.55	2.00	1.33	2.63	1.75	2.28	1.52
	15	2.21	1.47	1.79	1.19	2.52	1.68	2.08	1.38	2.84	1.89	2.37	1.58
	16	2.40	1.60	1.86	1.24	2.75	1.83	2.16	1.44	3.10	2.06	2.47	1.64
	17	2.62	1.74	1.93	1.28	3.01	2.00	2.25	1.49	3.40	2.26	2.58	1.71
	18	2.88	1.92	2.01	1.33	3.32	2.21	2.34	1.56	3.76	2.50	2.69	1.79
	19	3.18	2.12	2.09	1.39	3.68	2.45	2.45	1.63	4.19	2.79	2.82	1.88
	20	3.53	2.35	2.17	1.45	4.08	2.71	2.56	1.70	4.64	3.09	3.02	2.01
	22	4.27	2.84	2.43	1.61	4.94	3.28	2.95	1.96	5.62	3.74	3.53	2.35
	24	5.08	3.38	2.76	1.84	5.88	3.91	3.37	2.24	6.68	4.45	4.05	2.69
	26	5.96	3.97	3.10	2.06	6.90	4.59	3.80	2.53	7.84	5.22	4.58	3.05
	28	6.92	4.60	3.44	2.29	8.00	5.32	4.24	2.82	9.10	6.05	5.12	3.41
	30	7.94	5.28	3.79	2.52	9.18	6.11	4.67	3.11	10.4	6.95	5.66	3.77
	32	9.03	6.01	4.13	2.75	10.4	6.95	5.11	3.40	11.9	7.90	6.21	4.13

Other Constants and Properties

	94^c ASD	94^c LRFD	84^c ASD	84^c LRFD	76^c ASD	76^c LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	9.50	6.32	10.9	7.27	12.5	8.29
$t_y \times 10^3$, (kips)$^{-1}$	1.21	0.802	1.35	0.900	1.49	0.992
$t_r \times 10^3$, (kips)$^{-1}$	1.48	0.987	1.66	1.11	1.83	1.22
r_x/r_y	4.98		5.02		5.05	
r_y, in.	1.98		1.95		1.92	

[c] Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

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W24

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W24×											
		68^c				62^c				55$^{c, v}$			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	1.86	1.24	2.01	1.34	2.07	1.38	2.33	1.55	2.41	1.60	2.66	1.77
	6	2.04	1.36	2.01	1.34	2.44	1.62	2.44	1.63	2.86	1.90	2.82	1.87
	7	2.11	1.41	2.04	1.36	2.58	1.72	2.56	1.70	3.04	2.03	2.95	1.96
	8	2.19	1.46	2.11	1.40	2.76	1.84	2.68	1.78	3.27	2.18	3.10	2.07
	9	2.29	1.52	2.18	1.45	2.99	1.99	2.82	1.87	3.55	2.36	3.27	2.18
	10	2.41	1.60	2.26	1.50	3.26	2.17	2.97	1.97	3.88	2.58	3.46	2.30
	11	2.54	1.69	2.34	1.56	3.58	2.38	3.13	2.08	4.29	2.86	3.67	2.44
	12	2.69	1.79	2.43	1.62	4.07	2.71	3.32	2.21	4.80	3.19	3.91	2.60
	13	2.87	1.91	2.53	1.68	4.67	3.11	3.53	2.35	5.57	3.70	4.18	2.78
	14	3.07	2.04	2.63	1.75	5.42	3.60	3.77	2.51	6.46	4.29	4.51	3.00
	15	3.30	2.20	2.75	1.83	6.22	4.14	4.15	2.76	7.41	4.93	5.08	3.38
	16	3.59	2.39	2.87	1.91	7.08	4.71	4.62	3.08	8.43	5.61	5.68	3.78
	17	3.97	2.64	3.01	2.00	7.99	5.31	5.11	3.40	9.52	6.33	6.29	4.18
	18	4.42	2.94	3.16	2.10	8.96	5.96	5.60	3.72	10.7	7.10	6.91	4.60
	19	4.92	3.27	3.35	2.23	9.98	6.64	6.10	4.06	11.9	7.91	7.55	5.02
	20	5.45	3.63	3.66	2.43	11.1	7.36	6.61	4.40	13.2	8.77	8.20	5.46
	22	6.60	4.39	4.29	2.85	13.4	8.90	7.64	5.08	15.9	10.6	9.52	6.34
	24	7.85	5.22	4.94	3.29								
	26	9.21	6.13	5.61	3.74								
	28	10.7	7.11	6.30	4.19								
	30	12.3	8.16	6.99	4.65								

Other Constants and Properties

	68 ASD	68 LRFD	62 ASD	62 LRFD	55 ASD	55 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	14.5	9.67	22.7	15.1	26.8	17.8
$t_y \times 10^3$, (kips)$^{-1}$	1.66	1.11	1.84	1.22	2.06	1.37
$t_r \times 10^3$, (kips)$^{-1}$	2.04	1.36	2.25	1.50	2.53	1.69
r_x / r_y	5.11		6.69		6.80	
r_y, in.	1.87		1.38		1.34	

[c] Shape is slender for compression for F_y = 50 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(1) with F_y = 50 ksi; therefore, ϕ_v = 0.90 and Ω_v = 1.67.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

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W21

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W21×											
		275[h]				248				223			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.408	0.272	0.476	0.316	0.453	0.301	0.531	0.353	0.502	0.334	0.593	0.394
	6	0.425	0.283	0.476	0.316	0.471	0.313	0.531	0.353	0.523	0.348	0.593	0.394
	7	0.431	0.287	0.476	0.316	0.478	0.318	0.531	0.353	0.531	0.353	0.593	0.394
	8	0.438	0.291	0.476	0.316	0.486	0.323	0.531	0.353	0.540	0.359	0.593	0.394
	9	0.446	0.297	0.476	0.316	0.495	0.329	0.531	0.353	0.551	0.366	0.593	0.394
	10	0.456	0.303	0.476	0.316	0.506	0.336	0.531	0.353	0.563	0.374	0.593	0.394
	11	0.466	0.310	0.476	0.317	0.518	0.344	0.532	0.354	0.576	0.384	0.594	0.395
	12	0.478	0.318	0.480	0.319	0.531	0.353	0.536	0.357	0.592	0.394	0.600	0.399
	13	0.491	0.327	0.483	0.322	0.546	0.363	0.541	0.360	0.609	0.405	0.606	0.403
	14	0.506	0.337	0.487	0.324	0.563	0.374	0.546	0.363	0.628	0.418	0.612	0.407
	15	0.522	0.348	0.491	0.327	0.581	0.387	0.551	0.366	0.649	0.432	0.618	0.411
	16	0.541	0.360	0.495	0.329	0.601	0.400	0.556	0.370	0.672	0.447	0.625	0.416
	17	0.560	0.373	0.499	0.332	0.624	0.415	0.561	0.373	0.698	0.464	0.631	0.420
	18	0.582	0.387	0.503	0.335	0.648	0.431	0.566	0.376	0.727	0.483	0.638	0.424
	19	0.606	0.403	0.508	0.338	0.676	0.450	0.571	0.380	0.758	0.504	0.644	0.429
	20	0.633	0.421	0.512	0.341	0.706	0.469	0.576	0.383	0.792	0.527	0.651	0.433
	22	0.694	0.462	0.521	0.346	0.774	0.515	0.587	0.391	0.872	0.580	0.665	0.443
	24	0.767	0.511	0.530	0.352	0.858	0.571	0.599	0.398	0.968	0.644	0.680	0.453
	26	0.856	0.570	0.539	0.359	0.958	0.638	0.611	0.406	1.08	0.722	0.696	0.463
	28	0.964	0.641	0.549	0.365	1.08	0.719	0.623	0.415	1.23	0.816	0.712	0.474
	30	1.10	0.730	0.559	0.372	1.23	0.819	0.636	0.423	1.40	0.933	0.729	0.485
	32	1.25	0.830	0.569	0.379	1.40	0.932	0.649	0.432	1.60	1.06	0.747	0.497
	34	1.41	0.937	0.580	0.386	1.58	1.05	0.663	0.441	1.80	1.20	0.765	0.509
	36	1.58	1.05	0.592	0.394	1.77	1.18	0.678	0.451	2.02	1.34	0.785	0.522
	38	1.76	1.17	0.603	0.401	1.98	1.31	0.693	0.461	2.25	1.50	0.806	0.536
	40	1.95	1.30	0.616	0.410	2.19	1.46	0.709	0.472	2.49	1.66	0.827	0.550

Other Constants and Properties

	275[h] ASD	275[h] LRFD	248 ASD	248 LRFD	223 ASD	223 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	1.87	1.24	2.10	1.39	2.38	1.58
$t_y \times 10^3$, (kips)$^{-1}$	0.408	0.272	0.453	0.301	0.502	0.334
$t_r \times 10^3$, (kips)$^{-1}$	0.502	0.334	0.556	0.371	0.617	0.411
r_x/r_y	3.13		3.12		3.14	
r_y, in.	3.10		3.08		3.04	

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

W21

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W21×											
		201				182				166			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.563	0.375	0.672	0.447	0.623	0.415	0.748	0.498	0.684	0.455	0.825	0.549
	6	0.587	0.391	0.672	0.447	0.650	0.432	0.748	0.498	0.714	0.475	0.825	0.549
	7	0.596	0.397	0.672	0.447	0.660	0.439	0.748	0.498	0.725	0.482	0.825	0.549
	8	0.606	0.403	0.672	0.447	0.672	0.447	0.748	0.498	0.738	0.491	0.825	0.549
	9	0.618	0.411	0.672	0.447	0.685	0.456	0.748	0.498	0.753	0.501	0.825	0.549
	10	0.632	0.421	0.672	0.447	0.700	0.466	0.748	0.498	0.770	0.512	0.825	0.549
	11	0.648	0.431	0.675	0.449	0.718	0.478	0.752	0.500	0.789	0.525	0.829	0.552
	12	0.665	0.443	0.682	0.454	0.737	0.491	0.761	0.507	0.811	0.540	0.841	0.559
	13	0.685	0.455	0.690	0.459	0.759	0.505	0.771	0.513	0.835	0.556	0.852	0.567
	14	0.706	0.470	0.698	0.464	0.784	0.521	0.780	0.519	0.862	0.574	0.864	0.575
	15	0.730	0.486	0.706	0.470	0.811	0.539	0.790	0.526	0.892	0.594	0.876	0.583
	16	0.757	0.504	0.714	0.475	0.841	0.559	0.801	0.533	0.925	0.616	0.888	0.591
	17	0.786	0.523	0.723	0.481	0.874	0.581	0.811	0.540	0.962	0.640	0.901	0.599
	18	0.819	0.545	0.731	0.487	0.910	0.606	0.822	0.547	1.00	0.667	0.914	0.608
	19	0.854	0.568	0.740	0.492	0.951	0.632	0.833	0.554	1.05	0.697	0.927	0.617
	20	0.894	0.595	0.749	0.498	0.995	0.662	0.844	0.562	1.10	0.729	0.941	0.626
	22	0.985	0.655	0.768	0.511	1.10	0.730	0.868	0.577	1.21	0.805	0.970	0.645
	24	1.10	0.729	0.788	0.524	1.22	0.813	0.893	0.594	1.35	0.897	1.00	0.666
	26	1.23	0.818	0.809	0.538	1.37	0.914	0.919	0.612	1.52	1.01	1.03	0.688
	28	1.39	0.926	0.831	0.553	1.56	1.04	0.947	0.630	1.72	1.15	1.07	0.711
	30	1.59	1.06	0.854	0.568	1.79	1.19	0.977	0.650	1.98	1.31	1.11	0.736
	32	1.81	1.21	0.878	0.584	2.03	1.35	1.01	0.671	2.25	1.50	1.15	0.763
	34	2.05	1.36	0.904	0.602	2.30	1.53	1.04	0.694	2.54	1.69	1.19	0.792
	36	2.30	1.53	0.932	0.620	2.57	1.71	1.08	0.718	2.85	1.89	1.24	0.823
	38	2.56	1.70	0.961	0.640	2.87	1.91	1.12	0.744	3.17	2.11	1.29	0.857
	40	2.83	1.89	0.993	0.660	3.18	2.11	1.16	0.772	3.51	2.34	1.34	0.895

Other Constants and Properties

	201 ASD	201 LRFD	182 ASD	182 LRFD	166 ASD	166 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	2.68	1.78	2.99	1.99	3.30	2.19
$t_y \times 10^3$, (kips)$^{-1}$	0.563	0.375	0.623	0.415	0.684	0.455
$t_r \times 10^3$, (kips)$^{-1}$	0.692	0.461	0.765	0.510	0.841	0.560
r_x/r_y	3.14		3.13		3.13	
r_y, in.	3.02		3.00		2.99	

W21

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W21×											
		147				132				122			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.773	0.514	0.955	0.635	0.861	0.573	1.07	0.712	0.930	0.619	1.16	0.772
	6	0.808	0.537	0.955	0.635	0.900	0.599	1.07	0.712	0.973	0.647	1.16	0.772
	7	0.820	0.546	0.955	0.635	0.914	0.608	1.07	0.712	0.988	0.658	1.16	0.772
	8	0.835	0.556	0.955	0.635	0.931	0.620	1.07	0.712	1.01	0.670	1.16	0.772
	9	0.853	0.567	0.955	0.635	0.951	0.633	1.07	0.712	1.03	0.684	1.16	0.772
	10	0.873	0.581	0.955	0.635	0.973	0.647	1.07	0.712	1.05	0.700	1.16	0.772
	11	0.895	0.596	0.963	0.641	0.999	0.664	1.08	0.719	1.08	0.719	1.17	0.781
	12	0.920	0.612	0.978	0.651	1.03	0.683	1.10	0.731	1.11	0.739	1.19	0.795
	13	0.949	0.631	0.993	0.661	1.06	0.705	1.12	0.743	1.15	0.763	1.22	0.809
	14	0.980	0.652	1.01	0.671	1.09	0.728	1.14	0.756	1.19	0.789	1.24	0.823
	15	1.02	0.675	1.02	0.682	1.13	0.755	1.16	0.769	1.23	0.817	1.26	0.838
	16	1.05	0.701	1.04	0.693	1.18	0.784	1.18	0.782	1.28	0.849	1.28	0.854
	17	1.10	0.730	1.06	0.704	1.23	0.816	1.20	0.796	1.33	0.884	1.31	0.870
	18	1.14	0.761	1.08	0.716	1.28	0.852	1.22	0.811	1.39	0.924	1.33	0.887
	19	1.20	0.796	1.09	0.728	1.34	0.892	1.24	0.826	1.45	0.967	1.36	0.905
	20	1.25	0.835	1.11	0.740	1.41	0.935	1.26	0.841	1.52	1.01	1.39	0.923
	22	1.39	0.924	1.15	0.767	1.56	1.04	1.31	0.874	1.69	1.13	1.45	0.961
	24	1.55	1.03	1.19	0.795	1.74	1.16	1.37	0.910	1.89	1.26	1.51	1.00
	26	1.75	1.17	1.24	0.825	1.97	1.31	1.43	0.948	2.14	1.43	1.58	1.05
	28	2.00	1.33	1.29	0.858	2.25	1.50	1.49	0.990	2.45	1.63	1.65	1.10
	30	2.29	1.53	1.34	0.894	2.59	1.72	1.56	1.04	2.82	1.87	1.74	1.16
	32	2.61	1.74	1.40	0.933	2.95	1.96	1.63	1.09	3.20	2.13	1.83	1.22
	34	2.95	1.96	1.47	0.975	3.32	2.21	1.72	1.14	3.62	2.41	1.97	1.31
	36	3.30	2.20	1.54	1.02	3.73	2.48	1.85	1.23	4.06	2.70	2.12	1.41
	38	3.68	2.45	1.64	1.09	4.15	2.76	1.98	1.32	4.52	3.01	2.28	1.52
	40	4.08	2.71	1.75	1.16	4.60	3.06	2.12	1.41	5.01	3.33	2.44	1.62

Other Constants and Properties

	147		132		122	
$b_y \times 10^3$, (kip-ft)$^{-1}$	3.85	2.56	4.33	2.88	4.71	3.14
$t_y \times 10^3$, (kips)$^{-1}$	0.773	0.514	0.861	0.573	0.930	0.619
$t_r \times 10^3$, (kips)$^{-1}$	0.950	0.633	1.06	0.705	1.14	0.762
r_x/r_y	3.11		3.11		3.11	
r_y, in.	2.95		2.93		2.92	

W21

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W21×											
		111				101^c				93			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	1.02	0.682	1.28	0.850	1.13	0.753	1.41	0.937	1.22	0.814	1.61	1.07
	6	1.07	0.713	1.28	0.850	1.18	0.784	1.41	0.937	1.37	0.910	1.61	1.07
	7	1.09	0.725	1.28	0.850	1.20	0.796	1.41	0.937	1.42	0.948	1.63	1.09
	8	1.11	0.739	1.28	0.850	1.22	0.809	1.41	0.937	1.49	0.993	1.68	1.12
	9	1.13	0.754	1.28	0.850	1.24	0.826	1.41	0.937	1.57	1.05	1.73	1.15
	10	1.16	0.773	1.28	0.850	1.27	0.846	1.41	0.937	1.67	1.11	1.78	1.18
	11	1.19	0.793	1.29	0.861	1.31	0.869	1.43	0.951	1.78	1.19	1.83	1.22
	12	1.23	0.816	1.32	0.877	1.34	0.894	1.46	0.969	1.91	1.27	1.89	1.25
	13	1.27	0.842	1.34	0.894	1.39	0.923	1.49	0.989	2.07	1.38	1.95	1.29
	14	1.31	0.871	1.37	0.911	1.43	0.955	1.52	1.01	2.25	1.50	2.01	1.34
	15	1.36	0.903	1.40	0.929	1.49	0.990	1.55	1.03	2.46	1.64	2.08	1.38
	16	1.41	0.939	1.42	0.947	1.55	1.03	1.58	1.05	2.71	1.80	2.15	1.43
	17	1.47	0.979	1.45	0.966	1.61	1.07	1.61	1.07	3.01	2.00	2.23	1.48
	18	1.54	1.02	1.48	0.986	1.69	1.12	1.65	1.10	3.36	2.23	2.32	1.54
	19	1.61	1.07	1.51	1.01	1.77	1.18	1.69	1.12	3.74	2.49	2.41	1.60
	20	1.69	1.12	1.55	1.03	1.86	1.23	1.72	1.15	4.15	2.76	2.51	1.67
	22	1.88	1.25	1.62	1.08	2.06	1.37	1.81	1.20	5.02	3.34	2.77	1.84
	24	2.11	1.40	1.69	1.13	2.32	1.54	1.90	1.26	5.97	3.97	3.12	2.07
	26	2.39	1.59	1.78	1.18	2.63	1.75	2.00	1.33	7.01	4.66	3.46	2.30
	28	2.74	1.82	1.87	1.24	3.02	2.01	2.11	1.41	8.13	5.41	3.81	2.54
	30	3.14	2.09	1.97	1.31	3.46	2.30	2.24	1.49	9.33	6.21	4.16	2.77
	32	3.58	2.38	2.12	1.41	3.94	2.62	2.46	1.64				
	34	4.04	2.69	2.31	1.53	4.45	2.96	2.69	1.79				
	36	4.53	3.01	2.50	1.66	4.99	3.32	2.92	1.94				
	38	5.05	3.36	2.69	1.79	5.56	3.70	3.14	2.09				
	40	5.59	3.72	2.88	1.91	6.16	4.10	3.37	2.24				
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		5.22		3.48		5.77		3.84		10.3		6.83	
$t_y \times 10^3$, (kips)$^{-1}$		1.02		0.682		1.12		0.746		1.22		0.814	
$t_r \times 10^3$, (kips)$^{-1}$		1.26		0.839		1.38		0.918		1.50		1.00	
r_x/r_y		3.12				3.12				4.73			
r_y, in.		2.90				2.89				1.84			

^{c} Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W21

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W21×											
		83^c				73^c				68^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	1.37	0.914	1.82	1.21	1.61	1.07	2.07	1.38	1.76	1.17	2.23	1.48
	6	1.53	1.02	1.82	1.21	1.78	1.19	2.07	1.38	1.95	1.30	2.23	1.48
	7	1.60	1.06	1.85	1.23	1.85	1.23	2.11	1.40	2.02	1.35	2.27	1.51
	8	1.67	1.11	1.90	1.26	1.93	1.28	2.18	1.45	2.11	1.40	2.35	1.56
	9	1.77	1.17	1.96	1.30	2.02	1.34	2.25	1.49	2.21	1.47	2.43	1.62
	10	1.87	1.25	2.02	1.34	2.14	1.43	2.32	1.55	2.33	1.55	2.51	1.67
	11	2.00	1.33	2.09	1.39	2.29	1.52	2.40	1.60	2.47	1.65	2.61	1.73
	12	2.15	1.43	2.16	1.43	2.47	1.64	2.49	1.66	2.67	1.77	2.70	1.80
	13	2.33	1.55	2.23	1.48	2.67	1.78	2.58	1.72	2.89	1.92	2.81	1.87
	14	2.53	1.69	2.31	1.54	2.92	1.94	2.68	1.79	3.16	2.10	2.93	1.95
	15	2.78	1.85	2.40	1.60	3.20	2.13	2.79	1.86	3.47	2.31	3.05	2.03
	16	3.06	2.04	2.49	1.66	3.54	2.35	2.91	1.94	3.84	2.55	3.19	2.12
	17	3.40	2.26	2.59	1.72	3.93	2.62	3.04	2.02	4.27	2.84	3.34	2.22
	18	3.80	2.53	2.70	1.80	4.41	2.93	3.18	2.11	4.79	3.19	3.50	2.33
	19	4.23	2.82	2.82	1.88	4.91	3.27	3.33	2.22	5.34	3.55	3.72	2.48
	20	4.69	3.12	2.95	1.96	5.44	3.62	3.58	2.38	5.91	3.93	4.03	2.68
	22	5.67	3.78	3.37	2.24	6.58	4.38	4.13	2.75	7.16	4.76	4.66	3.10
	24	6.75	4.49	3.81	2.53	7.83	5.21	4.68	3.12	8.52	5.67	5.31	3.53
	26	7.93	5.27	4.25	2.83	9.19	6.12	5.24	3.49	9.99	6.65	5.95	3.96
	28	9.19	6.12	4.69	3.12	10.7	7.09	5.81	3.86	11.6	7.71	6.60	4.39
	30	10.6	7.02	5.13	3.41	12.2	8.14	6.37	4.24	13.3	8.85	7.26	4.83

Other Constants and Properties

	83^c ASD	83^c LRFD	73^c ASD	73^c LRFD	68^c ASD	68^c LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	11.7	7.77	13.4	8.91	14.6	9.71
$t_y \times 10^3$, (kips)$^{-1}$	1.37	0.911	1.55	1.03	1.67	1.11
$t_r \times 10^3$, (kips)$^{-1}$	1.68	1.12	1.91	1.27	2.05	1.37
r_x/r_y	4.74		4.77		4.78	
r_y, in.	1.83		1.81		1.80	

^{c} Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W21

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W21×											
		62^c				57^c				55^c			
Design		$p\times10^3$ (kips)$^{-1}$		$b_x\times10^3$ (kip-ft)$^{-1}$		$p\times10^3$ (kips)$^{-1}$		$b_x\times10^3$ (kip-ft)$^{-1}$		$p\times10^3$ (kips)$^{-1}$		$b_x\times10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	1.97	1.31	2.47	1.65	2.17	1.44	2.76	1.84	2.28	1.52	2.83	1.88
	6	2.19	1.45	2.47	1.65	2.58	1.72	2.91	1.94	2.54	1.69	2.83	1.88
	7	2.27	1.51	2.54	1.69	2.75	1.83	3.04	2.03	2.64	1.76	2.92	1.94
	8	2.37	1.58	2.62	1.74	2.96	1.97	3.19	2.12	2.76	1.84	3.02	2.01
	9	2.49	1.66	2.71	1.81	3.21	2.14	3.35	2.23	2.90	1.93	3.14	2.09
	10	2.63	1.75	2.81	1.87	3.56	2.37	3.53	2.35	3.07	2.04	3.27	2.17
	11	2.79	1.86	2.92	1.94	4.02	2.68	3.73	2.48	3.27	2.18	3.40	2.26
	12	2.98	1.98	3.04	2.02	4.60	3.06	3.95	2.63	3.50	2.33	3.55	2.36
	13	3.22	2.14	3.16	2.10	5.32	3.54	4.20	2.79	3.78	2.51	3.71	2.47
	14	3.53	2.35	3.30	2.19	6.17	4.10	4.48	2.98	4.11	2.73	3.89	2.58
	15	3.89	2.59	3.44	2.29	7.08	4.71	4.94	3.29	4.55	3.03	4.08	2.71
	16	4.31	2.87	3.61	2.40	8.06	5.36	5.47	3.64	5.07	3.38	4.29	2.86
	17	4.83	3.21	3.78	2.52	9.10	6.05	6.01	4.00	5.71	3.80	4.53	3.01
	18	5.41	3.60	3.98	2.65	10.2	6.79	6.55	4.36	6.40	4.26	4.92	3.27
	19	6.03	4.01	4.33	2.88	11.4	7.56	7.10	4.72	7.13	4.75	5.38	3.58
	20	6.68	4.45	4.70	3.13	12.6	8.38	7.65	5.09	7.90	5.26	5.86	3.90
	21	7.37	4.90	5.08	3.38	13.9	9.24	8.20	5.46	8.71	5.80	6.34	4.22
	22	8.09	5.38	5.46	3.63	15.2	10.1	8.76	5.83	9.56	6.36	6.84	4.55
	23	8.84	5.88	5.85	3.89					10.5	6.95	7.34	4.88
	24	9.63	6.40	6.24	4.15					11.4	7.57	7.84	5.22
	25	10.4	6.95	6.63	4.41					12.3	8.22	8.35	5.56
	26	11.3	7.52	7.02	4.67					13.4	8.89	8.86	5.90
	27	12.2	8.10	7.42	4.94					14.4	9.58	9.38	6.24
	28	13.1	8.72	7.81	5.20					15.5	10.3	9.90	6.59
	29	14.1	9.35	8.21	5.46								

Other Constants and Properties

	62^c ASD	62^c LRFD	57^c ASD	57^c LRFD	55^c ASD	55^c LRFD
$b_y\times10^3$, (kip-ft)$^{-1}$	16.4	10.9	24.1	16.0	19.4	12.9
$t_y\times10^3$, (kips)$^{-1}$	1.83	1.21	2.00	1.33	2.06	1.37
$t_r\times10^3$, (kips)$^{-1}$	2.24	1.49	2.46	1.64	2.53	1.69
r_x/r_y	4.82		6.19		4.86	
r_y, in.	1.77		1.35		1.73	

[c] Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

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Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

W21

F_y = 50 ksi

Shape		W21×											
		50^c				$48^{c,f}$				44^c			
Design		$p \times 10^3$ $(kips)^{-1}$		$b_x \times 10^3$ $(kip\text{-}ft)^{-1}$		$p \times 10^3$ $(kips)^{-1}$		$b_x \times 10^3$ $(kip\text{-}ft)^{-1}$		$p \times 10^3$ $(kips)^{-1}$		$b_x \times 10^3$ $(kip\text{-}ft)^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	2.53	1.69	3.24	2.15	2.70	1.80	3.36	2.23	2.96	1.97	3.73	2.48
	6	3.05	2.03	3.45	2.30	3.03	2.02	3.36	2.23	3.61	2.40	4.03	2.68
	7	3.26	2.17	3.63	2.41	3.16	2.10	3.47	2.31	3.87	2.58	4.24	2.82
	8	3.53	2.35	3.81	2.54	3.31	2.20	3.61	2.40	4.20	2.79	4.48	2.98
	9	3.85	2.56	4.02	2.68	3.50	2.33	3.76	2.50	4.61	3.07	4.75	3.16
	10	4.25	2.83	4.26	2.83	3.72	2.47	3.92	2.61	5.11	3.40	5.05	3.36
	11	4.83	3.21	4.52	3.01	3.98	2.65	4.10	2.73	5.73	3.81	5.39	3.59
	12	5.57	3.71	4.82	3.21	4.28	2.85	4.30	2.86	6.68	4.45	5.79	3.85
	13	6.52	4.34	5.16	3.43	4.63	3.08	4.51	3.00	7.84	5.22	6.25	4.16
	14	7.56	5.03	5.67	3.77	5.05	3.36	4.74	3.16	9.10	6.05	7.11	4.73
	15	8.68	5.77	6.36	4.23	5.60	3.72	5.01	3.33	10.4	6.95	7.99	5.32
	16	9.87	6.57	7.06	4.70	6.31	4.20	5.30	3.52	11.9	7.91	8.90	5.92
	17	11.1	7.42	7.78	5.17	7.13	4.74	5.75	3.82	13.4	8.93	9.83	6.54
	18	12.5	8.31	8.51	5.66	7.99	5.32	6.35	4.22	15.0	10.0	10.8	7.18
	19	13.9	9.26	9.24	6.15	8.90	5.92	6.97	4.63	16.8	11.1	11.8	7.82
	20	15.4	10.3	9.99	6.65	9.86	6.56	7.60	5.06	18.6	12.4	12.7	8.47
	21	17.0	11.3	10.7	7.15	10.9	7.23	8.25	5.49	20.5	13.6	13.7	9.13
	22					11.9	7.94	8.91	5.93				
	23					13.0	8.68	9.58	6.37				
	24					14.2	9.45	10.3	6.82				
	25					15.4	10.3	10.9	7.28				
	26					16.7	11.1	11.6	7.75				
	27					18.0	12.0	12.3	8.22				

Other Constants and Properties

	W21×50 ASD	W21×50 LRFD	W21×48 ASD	W21×48 LRFD	W21×44 ASD	W21×44 LRFD
$b_y \times 10^3$, $(kip\text{-}ft)^{-1}$	29.2	19.4	24.2	16.1	35.0	23.3
$t_y \times 10^3$, $(kips)^{-1}$	2.27	1.51	2.37	1.58	2.57	1.71
$t_r \times 10^3$, $(kips)^{-1}$	2.79	1.86	2.91	1.94	3.16	2.10
r_x/r_y	6.29		4.96		6.40	
r_y, in.	1.30		1.66		1.26	

[c] Shape is slender for compression for F_y = 50 ksi.

[f] Shape does not meet compact limit for flexure for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W18

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W18×											
		311^h				283^h				258^h			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.365	0.243	0.473	0.314	0.401	0.267	0.527	0.351	0.439	0.292	0.583	0.388
	6	0.381	0.253	0.473	0.314	0.419	0.279	0.527	0.351	0.460	0.306	0.583	0.388
	7	0.387	0.257	0.473	0.314	0.426	0.284	0.527	0.351	0.468	0.311	0.583	0.388
	8	0.394	0.262	0.473	0.314	0.434	0.289	0.527	0.351	0.477	0.317	0.583	0.388
	9	0.402	0.268	0.473	0.314	0.443	0.295	0.527	0.351	0.487	0.324	0.583	0.388
	10	0.412	0.274	0.473	0.314	0.454	0.302	0.527	0.351	0.499	0.332	0.583	0.388
	11	0.422	0.281	0.474	0.315	0.466	0.310	0.530	0.352	0.512	0.341	0.587	0.390
	12	0.434	0.289	0.477	0.317	0.480	0.319	0.533	0.355	0.528	0.351	0.591	0.393
	13	0.447	0.298	0.480	0.319	0.495	0.329	0.537	0.357	0.545	0.362	0.595	0.396
	14	0.462	0.308	0.483	0.321	0.512	0.340	0.540	0.359	0.564	0.375	0.600	0.399
	15	0.479	0.319	0.486	0.323	0.530	0.353	0.544	0.362	0.585	0.389	0.604	0.402
	16	0.497	0.331	0.489	0.325	0.551	0.367	0.548	0.364	0.608	0.405	0.609	0.405
	17	0.517	0.344	0.492	0.327	0.574	0.382	0.551	0.367	0.634	0.422	0.613	0.408
	18	0.540	0.359	0.495	0.329	0.600	0.399	0.555	0.369	0.663	0.441	0.618	0.411
	19	0.564	0.375	0.498	0.331	0.628	0.418	0.559	0.372	0.695	0.462	0.623	0.414
	20	0.592	0.394	0.501	0.333	0.659	0.439	0.563	0.374	0.730	0.486	0.627	0.417
	22	0.655	0.436	0.507	0.338	0.732	0.487	0.571	0.380	0.812	0.541	0.637	0.424
	24	0.732	0.487	0.514	0.342	0.821	0.546	0.579	0.385	0.913	0.607	0.648	0.431
	26	0.826	0.550	0.521	0.347	0.929	0.618	0.588	0.391	1.04	0.690	0.658	0.438
	28	0.942	0.627	0.528	0.351	1.06	0.708	0.596	0.397	1.19	0.793	0.669	0.445
	30	1.08	0.720	0.535	0.356	1.22	0.813	0.605	0.403	1.37	0.910	0.680	0.453
	32	1.23	0.819	0.542	0.361	1.39	0.925	0.614	0.409	1.56	1.04	0.692	0.460
	34	1.39	0.924	0.550	0.366	1.57	1.04	0.624	0.415	1.76	1.17	0.704	0.468
	36	1.56	1.04	0.557	0.371	1.76	1.17	0.634	0.422	1.97	1.31	0.716	0.477
	38	1.74	1.15	0.565	0.376	1.96	1.30	0.644	0.428	2.19	1.46	0.729	0.485
	40	1.92	1.28	0.573	0.382	2.17	1.45	0.654	0.435	2.43	1.62	0.743	0.494
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		1.72		1.15		1.93		1.28		2.15		1.43	
$t_y \times 10^3$, (kips)$^{-1}$		0.365		0.243		0.401		0.267		0.439		0.292	
$t_r \times 10^3$, (kips)$^{-1}$		0.448		0.299		0.493		0.328		0.540		0.360	
r_x/r_y		2.96				2.96				2.96			
r_y, in.		2.95				2.91				2.88			

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

W18

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W18×											
		234[h]				211				192			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.487	0.324	0.649	0.432	0.536	0.357	0.727	0.484	0.594	0.395	0.806	0.536
	6	0.510	0.339	0.649	0.432	0.562	0.374	0.727	0.484	0.624	0.415	0.806	0.536
	7	0.519	0.345	0.649	0.432	0.572	0.381	0.727	0.484	0.635	0.423	0.806	0.536
	8	0.529	0.352	0.649	0.432	0.584	0.388	0.727	0.484	0.648	0.431	0.806	0.536
	9	0.541	0.360	0.649	0.432	0.597	0.397	0.727	0.484	0.663	0.441	0.806	0.536
	10	0.554	0.369	0.649	0.432	0.612	0.407	0.727	0.484	0.680	0.453	0.807	0.537
	11	0.570	0.379	0.654	0.435	0.629	0.419	0.734	0.488	0.700	0.466	0.815	0.542
	12	0.587	0.390	0.659	0.438	0.649	0.432	0.740	0.493	0.722	0.480	0.823	0.548
	13	0.606	0.403	0.664	0.442	0.671	0.446	0.747	0.497	0.747	0.497	0.831	0.553
	14	0.628	0.418	0.670	0.446	0.695	0.462	0.754	0.502	0.775	0.515	0.840	0.559
	15	0.652	0.434	0.675	0.449	0.722	0.480	0.761	0.506	0.806	0.536	0.848	0.564
	16	0.678	0.451	0.681	0.453	0.752	0.501	0.768	0.511	0.840	0.559	0.857	0.570
	17	0.708	0.471	0.687	0.457	0.786	0.523	0.775	0.516	0.879	0.585	0.866	0.576
	18	0.741	0.493	0.692	0.461	0.823	0.548	0.782	0.520	0.921	0.613	0.875	0.582
	19	0.777	0.517	0.698	0.465	0.865	0.575	0.790	0.525	0.968	0.644	0.884	0.588
	20	0.818	0.544	0.704	0.469	0.910	0.606	0.797	0.531	1.02	0.679	0.894	0.595
	22	0.912	0.607	0.717	0.477	1.02	0.677	0.813	0.541	1.14	0.761	0.913	0.608
	24	1.03	0.683	0.729	0.485	1.15	0.765	0.829	0.552	1.30	0.862	0.934	0.621
	26	1.17	0.778	0.742	0.494	1.31	0.873	0.846	0.563	1.48	0.987	0.955	0.636
	28	1.35	0.897	0.756	0.503	1.52	1.01	0.864	0.575	1.72	1.14	0.978	0.651
	30	1.55	1.03	0.770	0.513	1.74	1.16	0.882	0.587	1.97	1.31	1.00	0.666
	32	1.76	1.17	0.785	0.522	1.98	1.32	0.902	0.600	2.24	1.49	1.03	0.683
	34	1.99	1.32	0.800	0.533	2.24	1.49	0.922	0.613	2.53	1.68	1.05	0.700
	36	2.23	1.48	0.816	0.543	2.51	1.67	0.943	0.627	2.84	1.89	1.08	0.718
	38	2.48	1.65	0.833	0.554	2.79	1.86	0.965	0.642	3.16	2.10	1.11	0.737
	40	2.75	1.83	0.850	0.566	3.09	2.06	0.988	0.657	3.50	2.33	1.14	0.757
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		2.39		1.59		2.70		1.80		2.99		1.99	
$t_y \times 10^3$, (kips)$^{-1}$		0.487		0.324		0.536		0.357		0.594		0.395	
$t_r \times 10^3$, (kips)$^{-1}$		0.598		0.399		0.659		0.439		0.730		0.487	
r_x / r_y		2.96				2.96				2.97			
r_y, in.		2.85				2.82				2.79			

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

W18

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W18×											
		175				158				143			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.650	0.432	0.895	0.596	0.721	0.480	1.00	0.666	0.795	0.529	1.11	0.736
	6	0.683	0.454	0.895	0.596	0.759	0.505	1.00	0.666	0.837	0.557	1.11	0.736
	7	0.695	0.463	0.895	0.596	0.773	0.514	1.00	0.666	0.853	0.567	1.11	0.736
	8	0.710	0.472	0.895	0.596	0.789	0.525	1.00	0.666	0.871	0.580	1.11	0.736
	9	0.727	0.484	0.895	0.596	0.808	0.538	1.00	0.666	0.892	0.594	1.11	0.736
	10	0.746	0.496	0.898	0.597	0.830	0.552	1.00	0.668	0.917	0.610	1.11	0.740
	11	0.768	0.511	0.907	0.604	0.855	0.569	1.02	0.676	0.945	0.629	1.13	0.750
	12	0.793	0.528	0.917	0.610	0.883	0.587	1.03	0.685	0.976	0.649	1.14	0.760
	13	0.821	0.546	0.927	0.617	0.914	0.608	1.04	0.693	1.01	0.673	1.16	0.770
	14	0.852	0.567	0.938	0.624	0.950	0.632	1.05	0.702	1.05	0.699	1.17	0.780
	15	0.887	0.590	0.948	0.631	0.989	0.658	1.07	0.710	1.10	0.729	1.19	0.791
	16	0.926	0.616	0.959	0.638	1.03	0.687	1.08	0.719	1.14	0.762	1.21	0.802
	17	0.969	0.645	0.970	0.645	1.08	0.720	1.10	0.729	1.20	0.798	1.22	0.814
	18	1.02	0.677	0.981	0.653	1.14	0.756	1.11	0.738	1.26	0.839	1.24	0.825
	19	1.07	0.712	0.993	0.661	1.20	0.796	1.12	0.748	1.33	0.884	1.26	0.838
	20	1.13	0.752	1.00	0.669	1.26	0.841	1.14	0.758	1.41	0.935	1.28	0.850
	22	1.27	0.844	1.03	0.685	1.42	0.946	1.17	0.779	1.58	1.05	1.32	0.876
	24	1.44	0.958	1.06	0.702	1.62	1.08	1.20	0.801	1.81	1.20	1.36	0.904
	26	1.65	1.10	1.08	0.720	1.86	1.24	1.24	0.824	2.08	1.39	1.40	0.933
	28	1.92	1.28	1.11	0.739	2.16	1.44	1.28	0.849	2.42	1.61	1.45	0.965
	30	2.20	1.47	1.14	0.759	2.48	1.65	1.32	0.875	2.77	1.85	1.50	0.999
	32	2.51	1.67	1.17	0.780	2.82	1.88	1.36	0.903	3.16	2.10	1.56	1.03
	34	2.83	1.88	1.21	0.803	3.19	2.12	1.40	0.933	3.56	2.37	1.61	1.07
	36	3.17	2.11	1.24	0.827	3.57	2.38	1.45	0.965	4.00	2.66	1.68	1.12
	38	3.53	2.35	1.28	0.852	3.98	2.65	1.50	0.999	4.45	2.96	1.74	1.16
	40	3.91	2.60	1.32	0.878	4.41	2.93	1.56	1.04	4.93	3.28	1.82	1.21
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		3.36		2.24		3.76		2.50		4.17		2.78	
$t_y \times 10^3$, (kips)$^{-1}$		0.650		0.432		0.721		0.480		0.795		0.529	
$t_r \times 10^3$, (kips)$^{-1}$		0.798		0.532		0.886		0.591		0.977		0.651	
r_x/r_y		2.97				2.96				2.97			
r_y, in.		2.76				2.74				2.72			

W18

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W18×											
		130				119				106			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.872	0.580	1.23	0.817	0.952	0.633	1.36	0.905	1.07	0.715	1.55	1.03
	6	0.919	0.611	1.23	0.817	1.00	0.667	1.36	0.905	1.13	0.754	1.55	1.03
	7	0.936	0.623	1.23	0.817	1.02	0.680	1.36	0.905	1.16	0.769	1.55	1.03
	8	0.957	0.636	1.23	0.817	1.04	0.695	1.36	0.905	1.18	0.786	1.55	1.03
	9	0.980	0.652	1.23	0.817	1.07	0.712	1.36	0.905	1.21	0.806	1.55	1.03
	10	1.01	0.670	1.24	0.823	1.10	0.732	1.37	0.912	1.25	0.829	1.56	1.04
	11	1.04	0.691	1.25	0.835	1.13	0.755	1.39	0.926	1.29	0.856	1.59	1.06
	12	1.07	0.714	1.27	0.847	1.17	0.781	1.41	0.941	1.33	0.885	1.62	1.08
	13	1.11	0.741	1.29	0.859	1.22	0.810	1.44	0.956	1.38	0.919	1.65	1.10
	14	1.16	0.770	1.31	0.872	1.27	0.842	1.46	0.972	1.44	0.957	1.68	1.12
	15	1.21	0.803	1.33	0.886	1.32	0.878	1.49	0.989	1.50	0.999	1.71	1.14
	16	1.26	0.840	1.35	0.899	1.38	0.919	1.51	1.01	1.57	1.05	1.74	1.16
	17	1.32	0.881	1.37	0.913	1.45	0.964	1.54	1.02	1.65	1.10	1.78	1.18
	18	1.39	0.926	1.39	0.928	1.52	1.01	1.57	1.04	1.74	1.16	1.81	1.21
	19	1.47	0.977	1.42	0.943	1.61	1.07	1.59	1.06	1.84	1.22	1.85	1.23
	20	1.55	1.03	1.44	0.959	1.70	1.13	1.62	1.08	1.95	1.30	1.89	1.26
	22	1.75	1.17	1.49	0.992	1.92	1.28	1.69	1.12	2.21	1.47	1.97	1.31
	24	2.00	1.33	1.54	1.03	2.20	1.46	1.75	1.17	2.53	1.68	2.06	1.37
	26	2.32	1.54	1.60	1.06	2.55	1.70	1.83	1.21	2.94	1.96	2.15	1.43
	28	2.69	1.79	1.66	1.10	2.96	1.97	1.90	1.27	3.41	2.27	2.26	1.50
	30	3.09	2.05	1.73	1.15	3.39	2.26	1.99	1.32	3.92	2.61	2.38	1.58
	32	3.51	2.34	1.80	1.20	3.86	2.57	2.08	1.39	4.46	2.97	2.51	1.67
	34	3.97	2.64	1.87	1.25	4.36	2.90	2.19	1.46	5.03	3.35	2.72	1.81
	36	4.45	2.96	1.96	1.30	4.89	3.25	2.34	1.56	5.64	3.75	2.92	1.94
	38	4.95	3.30	2.08	1.38	5.45	3.62	2.50	1.66	6.29	4.18	3.12	2.08
	40	5.49	3.65	2.20	1.47	6.04	4.02	2.65	1.77	6.97	4.63	3.32	2.21
Other Constants and Properties													
$b_y \times 10^3$, $(\text{kip-ft})^{-1}$		4.64		3.09		5.16		3.43		5.89		3.92	
$t_y \times 10^3$, $(\text{kips})^{-1}$		0.872		0.580		0.952		0.633		1.07		0.715	
$t_r \times 10^3$, $(\text{kips})^{-1}$		1.07		0.714		1.17		0.779		1.32		0.879	
r_x / r_y		2.97				2.94				2.95			
r_y, in.		2.70				2.69				2.66			

W18

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W18×											
		97				86				76^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	1.17	0.780	1.69	1.12	1.32	0.878	1.92	1.27	1.51	1.01	2.19	1.45
	6	1.24	0.823	1.69	1.12	1.39	0.928	1.92	1.27	1.59	1.06	2.19	1.45
	7	1.26	0.839	1.69	1.12	1.42	0.946	1.92	1.27	1.62	1.08	2.19	1.45
	8	1.29	0.858	1.69	1.12	1.46	0.968	1.92	1.27	1.65	1.10	2.19	1.45
	9	1.32	0.880	1.69	1.12	1.49	0.994	1.92	1.27	1.70	1.13	2.19	1.45
	10	1.36	0.906	1.71	1.14	1.54	1.02	1.94	1.29	1.75	1.16	2.22	1.48
	11	1.41	0.935	1.74	1.16	1.59	1.06	1.98	1.32	1.81	1.20	2.27	1.51
	12	1.45	0.968	1.77	1.18	1.64	1.09	2.02	1.35	1.87	1.24	2.32	1.54
	13	1.51	1.00	1.81	1.20	1.71	1.14	2.06	1.37	1.94	1.29	2.37	1.58
	14	1.57	1.05	1.84	1.23	1.78	1.18	2.11	1.40	2.03	1.35	2.43	1.62
	15	1.64	1.09	1.88	1.25	1.86	1.24	2.15	1.43	2.12	1.41	2.49	1.65
	16	1.72	1.14	1.92	1.28	1.95	1.30	2.20	1.47	2.22	1.48	2.55	1.69
	17	1.81	1.20	1.96	1.30	2.05	1.36	2.25	1.50	2.34	1.56	2.61	1.74
	18	1.90	1.27	2.00	1.33	2.16	1.44	2.31	1.53	2.47	1.64	2.68	1.78
	19	2.01	1.34	2.04	1.36	2.29	1.52	2.36	1.57	2.62	1.74	2.75	1.83
	20	2.13	1.42	2.09	1.39	2.43	1.61	2.42	1.61	2.78	1.85	2.82	1.88
	22	2.42	1.61	2.18	1.45	2.76	1.83	2.54	1.69	3.16	2.11	2.98	1.98
	24	2.78	1.85	2.29	1.52	3.17	2.11	2.68	1.79	3.65	2.43	3.16	2.10
	26	3.24	2.15	2.41	1.60	3.70	2.46	2.84	1.89	4.26	2.84	3.36	2.24
	28	3.75	2.50	2.54	1.69	4.29	2.86	3.01	2.00	4.94	3.29	3.67	2.44
	30	4.31	2.87	2.68	1.78	4.93	3.28	3.29	2.19	5.68	3.78	4.06	2.70
	32	4.90	3.26	2.91	1.93	5.61	3.73	3.59	2.39	6.46	4.30	4.45	2.96
	34	5.53	3.68	3.15	2.09	6.33	4.21	3.90	2.60	7.29	4.85	4.85	3.22
	36	6.20	4.13	3.38	2.25	7.10	4.72	4.21	2.80	8.17	5.44	5.24	3.49
	38	6.91	4.60	3.62	2.41	7.91	5.26	4.51	3.00	9.11	6.06	5.64	3.75
	40	7.66	5.10	3.86	2.57	8.76	5.83	4.82	3.21	10.1	6.71	6.04	4.02
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		6.44		4.29		7.36		4.90		8.44		5.62	
$t_y \times 10^3$, (kips)$^{-1}$		1.17		0.780		1.32		0.878		1.50		0.997	
$t_r \times 10^3$, (kips)$^{-1}$		1.44		0.960		1.62		1.08		1.84		1.23	
r_x/r_y		2.95				2.95				2.96			
r_y, in.		2.65				2.63				2.61			

[c] Shape is slender for compression for F_y = 50 ksi.

W18

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W18×											
		71				65				60^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	1.60	1.06	2.44	1.62	1.75	1.16	2.68	1.78	1.94	1.29	2.90	1.93
	6	1.82	1.21	2.44	1.62	2.00	1.33	2.68	1.78	2.18	1.45	2.90	1.93
	7	1.91	1.27	2.51	1.67	2.09	1.39	2.76	1.84	2.28	1.52	3.00	1.99
	8	2.02	1.34	2.59	1.72	2.21	1.47	2.85	1.90	2.41	1.60	3.10	2.06
	9	2.15	1.43	2.67	1.78	2.36	1.57	2.95	1.96	2.57	1.71	3.20	2.13
	10	2.30	1.53	2.76	1.83	2.53	1.68	3.05	2.03	2.76	1.83	3.32	2.21
	11	2.48	1.65	2.85	1.90	2.73	1.82	3.15	2.10	2.98	1.98	3.44	2.29
	12	2.70	1.80	2.95	1.96	2.97	1.98	3.27	2.18	3.25	2.16	3.58	2.38
	13	2.96	1.97	3.05	2.03	3.26	2.17	3.39	2.26	3.56	2.37	3.72	2.48
	14	3.26	2.17	3.17	2.11	3.60	2.40	3.53	2.35	3.94	2.62	3.88	2.58
	15	3.63	2.41	3.29	2.19	4.01	2.67	3.67	2.44	4.39	2.92	4.05	2.69
	16	4.06	2.70	3.42	2.28	4.50	2.99	3.83	2.55	4.94	3.28	4.23	2.82
	17	4.58	3.05	3.57	2.37	5.08	3.38	4.00	2.66	5.57	3.71	4.44	2.95
	18	5.14	3.42	3.72	2.48	5.69	3.79	4.19	2.79	6.25	4.16	4.66	3.10
	19	5.73	3.81	3.89	2.59	6.34	4.22	4.43	2.95	6.96	4.63	5.02	3.34
	20	6.34	4.22	4.12	2.74	7.02	4.67	4.76	3.17	7.71	5.13	5.41	3.60
	22	7.68	5.11	4.69	3.12	8.50	5.66	5.44	3.62	9.33	6.21	6.19	4.12
	24	9.14	6.08	5.25	3.50	10.1	6.73	6.11	4.07	11.1	7.39	6.98	4.64
	26	10.7	7.13	5.82	3.87	11.9	7.90	6.79	4.51	13.0	8.67	7.76	5.16
	28	12.4	8.27	6.38	4.25	13.8	9.16	7.46	4.96	15.1	10.1	8.55	5.69

Other Constants and Properties

	71 ASD	71 LRFD	65 ASD	65 LRFD	60 ASD	60 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	14.4	9.60	15.8	10.5	17.3	11.5
$t_y \times 10^3$, (kips)$^{-1}$	1.60	1.06	1.75	1.16	1.90	1.26
$t_r \times 10^3$, (kips)$^{-1}$	1.96	1.31	2.15	1.43	2.33	1.55
r_x / r_y	4.41		4.43		4.45	
r_y, in.	1.70		1.69		1.68	

[c] Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c / r_y equal to or greater than 200.

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W18

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape	W18×											
	55^c				50^c				46^c			
Design	$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending												
0	2.14	1.42	3.18	2.12	2.42	1.61	3.53	2.35	2.64	1.76	3.93	2.61
6	2.41	1.60	3.19	2.12	2.73	1.81	3.55	2.36	3.20	2.13	4.19	2.79
7	2.51	1.67	3.30	2.20	2.85	1.90	3.68	2.45	3.44	2.29	4.39	2.92
8	2.64	1.75	3.42	2.27	3.00	1.99	3.81	2.54	3.73	2.48	4.61	3.07
9	2.80	1.86	3.54	2.36	3.17	2.11	3.96	2.64	4.13	2.75	4.85	3.23
10	3.01	2.00	3.68	2.45	3.38	2.25	4.12	2.74	4.66	3.10	5.12	3.41
11	3.26	2.17	3.83	2.55	3.63	2.41	4.29	2.86	5.32	3.54	5.43	3.61
12	3.55	2.36	3.99	2.65	3.97	2.64	4.48	2.98	6.15	4.09	5.77	3.84
13	3.90	2.60	4.16	2.77	4.37	2.91	4.69	3.12	7.21	4.80	6.16	4.10
14	4.32	2.87	4.35	2.89	4.85	3.23	4.91	3.27	8.36	5.56	6.69	4.45
15	4.82	3.21	4.55	3.03	5.42	3.61	5.16	3.43	9.60	6.38	7.45	4.95
16	5.43	3.61	4.78	3.18	6.13	4.08	5.44	3.62	10.9	7.26	8.21	5.46
17	6.13	4.08	5.03	3.35	6.92	4.60	5.76	3.83	12.3	8.20	8.98	5.97
18	6.87	4.57	5.39	3.59	7.76	5.16	6.31	4.20	13.8	9.19	9.75	6.49
19	7.65	5.09	5.85	3.89	8.64	5.75	6.86	4.57	15.4	10.2	10.5	7.01
20	8.48	5.64	6.32	4.20	9.58	6.37	7.43	4.94	17.1	11.3	11.3	7.53
21	9.35	6.22	6.78	4.51	10.6	7.02	7.99	5.32	18.8	12.5	12.1	8.05
22	10.3	6.83	7.26	4.83	11.6	7.71	8.56	5.70				
23	11.2	7.46	7.73	5.14	12.7	8.43	9.14	6.08				
24	12.2	8.13	8.20	5.46	13.8	9.17	9.72	6.47				
25	13.3	8.82	8.68	5.78	15.0	9.95	10.3	6.85				
26	14.3	9.54	9.16	6.09	16.2	10.8	10.9	7.24				
27	15.5	10.3	9.63	6.41	17.5	11.6	11.5	7.63				

Other Constants and Properties

	W18×55 ASD	W18×55 LRFD	W18×50 ASD	W18×50 LRFD	W18×46 ASD	W18×46 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	19.3	12.8	21.5	14.3	30.5	20.3
$t_y \times 10^3$, (kips)$^{-1}$	2.06	1.37	2.27	1.51	2.47	1.65
$t_r \times 10^3$, (kips)$^{-1}$	2.53	1.69	2.79	1.86	3.04	2.03
r_x/r_y	4.44		4.47		5.62	
r_y, in.	1.67		1.65		1.29	

^{c} Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W18

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W18×							
		40^c				35^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	3.14	2.09	4.54	3.02	3.69	2.46	5.36	3.56
	6	3.84	2.55	4.88	3.25	4.57	3.04	5.84	3.89
	7	4.12	2.74	5.13	3.41	4.94	3.29	6.17	4.11
	8	4.48	2.98	5.40	3.59	5.40	3.59	6.54	4.35
	9	4.93	3.28	5.71	3.80	5.97	3.97	6.96	4.63
	10	5.47	3.64	6.05	4.03	6.68	4.44	7.43	4.94
	11	6.24	4.15	6.44	4.28	7.63	5.08	7.97	5.30
	12	7.25	4.82	6.88	4.58	9.00	5.99	8.60	5.72
	13	8.51	5.66	7.38	4.91	10.6	7.03	9.67	6.43
	14	9.87	6.56	8.30	5.52	12.2	8.15	11.0	7.29
	15	11.3	7.54	9.27	6.17	14.1	9.36	12.3	8.17
	16	12.9	8.57	10.3	6.83	16.0	10.6	13.6	9.07
	17	14.5	9.68	11.3	7.50	18.1	12.0	15.0	10.0
	18	16.3	10.9	12.3	8.17	20.2	13.5	16.4	10.9
	19	18.2	12.1	13.3	8.85	22.6	15.0	17.9	11.9
	20	20.1	13.4	14.3	9.54	25.0	16.6	19.3	12.8
	21	22.2	14.8	15.4	10.2				
	22								
	23								
	24								
	25								
	26								
	27								
	28								
	29								
	30								

Other Constants and Properties

	W18×40		W18×35	
$b_y \times 10^3$, (kip-ft)$^{-1}$	35.6	23.7	44.2	29.4
$t_y \times 10^3$, (kips)$^{-1}$	2.83	1.88	3.24	2.16
$t_r \times 10^3$, (kips)$^{-1}$	3.48	2.32	3.98	2.66
r_x / r_y	5.68		5.77	
r_y, in.	1.27		1.22	

^{c} Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c / r_y equal to or greater than 200.

W16

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W16×											
		100				89				77			
Design		$p\times10^3$ (kips)$^{-1}$		$b_x\times10^3$ (kip-ft)$^{-1}$		$p\times10^3$ (kips)$^{-1}$		$b_x\times10^3$ (kip-ft)$^{-1}$		$p\times10^3$ (kips)$^{-1}$		$b_x\times10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	**0**	1.14	0.756	1.80	1.20	1.27	0.848	2.04	1.35	1.48	0.983	2.38	1.58
	6	1.21	0.803	1.80	1.20	1.36	0.902	2.04	1.35	1.57	1.05	2.38	1.58
	7	1.23	0.820	1.80	1.20	1.39	0.922	2.04	1.35	1.61	1.07	2.38	1.58
	8	1.26	0.841	1.80	1.20	1.42	0.946	2.04	1.35	1.65	1.10	2.38	1.58
	9	1.30	0.865	1.80	1.20	1.46	0.973	2.04	1.36	1.70	1.13	2.39	1.59
	10	1.34	0.893	1.83	1.22	1.51	1.01	2.08	1.38	1.76	1.17	2.44	1.62
	11	1.39	0.925	1.86	1.24	1.57	1.04	2.12	1.41	1.82	1.21	2.49	1.65
	12	1.45	0.962	1.89	1.26	1.63	1.08	2.16	1.44	1.89	1.26	2.54	1.69
	13	1.51	1.00	1.93	1.28	1.70	1.13	2.20	1.46	1.98	1.32	2.59	1.72
	14	1.58	1.05	1.96	1.30	1.78	1.18	2.24	1.49	2.07	1.38	2.65	1.76
	15	1.65	1.10	1.99	1.33	1.87	1.24	2.29	1.52	2.18	1.45	2.71	1.80
	16	1.74	1.16	2.03	1.35	1.97	1.31	2.34	1.55	2.30	1.53	2.77	1.84
	17	1.84	1.23	2.07	1.38	2.08	1.39	2.38	1.59	2.43	1.62	2.83	1.89
	18	1.95	1.30	2.11	1.40	2.21	1.47	2.43	1.62	2.59	1.72	2.90	1.93
	19	2.08	1.38	2.15	1.43	2.35	1.57	2.49	1.65	2.76	1.83	2.97	1.98
	20	2.22	1.47	2.19	1.46	2.51	1.67	2.54	1.69	2.95	1.96	3.05	2.03
	22	2.55	1.70	2.28	1.51	2.90	1.93	2.66	1.77	3.41	2.27	3.21	2.14
	24	2.98	1.98	2.37	1.58	3.40	2.26	2.79	1.86	4.00	2.66	3.39	2.26
	26	3.50	2.33	2.48	1.65	3.99	2.65	2.93	1.95	4.70	3.13	3.59	2.39
	28	4.06	2.70	2.59	1.72	4.62	3.08	3.09	2.06	5.45	3.62	3.83	2.55
	30	4.66	3.10	2.72	1.81	5.31	3.53	3.27	2.17	6.25	4.16	4.20	2.80
	32	5.30	3.52	2.85	1.90	6.04	4.02	3.54	2.36	7.12	4.73	4.57	3.04
	34	5.98	3.98	3.04	2.03	6.82	4.54	3.82	2.54	8.03	5.34	4.94	3.29
	36	6.70	4.46	3.26	2.17	7.64	5.09	4.09	2.72	9.01	5.99	5.31	3.53
	38	7.47	4.97	3.47	2.31	8.52	5.67	4.36	2.90	10.0	6.68	5.68	3.78
	40	8.28	5.51	3.68	2.45	9.44	6.28	4.63	3.08	11.1	7.40	6.04	4.02

Other Constants and Properties

	100 ASD	100 LRFD	89 ASD	89 LRFD	77 ASD	77 LRFD
$b_y\times10^3$, (kip-ft)$^{-1}$	6.49	4.32	7.41	4.93	8.67	5.77
$t_y\times10^3$, (kips)$^{-1}$	1.14	0.756	1.27	0.848	1.48	0.983
$t_r\times10^3$, (kips)$^{-1}$	1.40	0.930	1.57	1.04	1.82	1.21
r_x/r_y	2.83		2.83		2.83	
r_y, in.	2.51		2.49		2.47	

W16

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape	W16×											
	67^c				57				50^c			
Design	$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	1.70	1.13	2.74	1.82	1.99	1.32	3.39	2.26	2.30	1.53	3.87	2.58
6	1.81	1.21	2.74	1.82	2.31	1.53	3.43	2.28	2.64	1.76	3.92	2.61
7	1.86	1.23	2.74	1.82	2.43	1.62	3.54	2.35	2.79	1.85	4.06	2.70
8	1.90	1.27	2.74	1.82	2.59	1.72	3.65	2.43	2.97	1.97	4.21	2.80
9	1.96	1.31	2.76	1.84	2.77	1.85	3.78	2.51	3.18	2.12	4.36	2.90
10	2.03	1.35	2.82	1.88	3.00	2.00	3.91	2.60	3.45	2.29	4.53	3.02
11	2.10	1.40	2.88	1.92	3.27	2.18	4.05	2.70	3.76	2.50	4.72	3.14
12	2.19	1.46	2.95	1.96	3.59	2.39	4.20	2.80	4.14	2.75	4.91	3.27
13	2.29	1.52	3.02	2.01	3.98	2.65	4.37	2.91	4.59	3.06	5.13	3.41
14	2.40	1.59	3.09	2.06	4.45	2.96	4.55	3.03	5.14	3.42	5.37	3.57
15	2.52	1.68	3.17	2.11	5.02	3.34	4.74	3.15	5.80	3.86	5.63	3.74
16	2.66	1.77	3.25	2.16	5.70	3.79	4.95	3.29	6.60	4.39	5.91	3.93
17	2.82	1.87	3.33	2.22	6.44	4.28	5.18	3.45	7.45	4.96	6.23	4.14
18	2.99	1.99	3.42	2.27	7.22	4.80	5.43	3.61	8.35	5.56	6.74	4.48
19	3.19	2.12	3.51	2.34	8.04	5.35	5.81	3.86	9.31	6.19	7.28	4.85
20	3.42	2.27	3.61	2.40	8.91	5.93	6.23	4.14	10.3	6.86	7.83	5.21
22	3.96	2.63	3.83	2.55	10.8	7.17	7.07	4.70	12.5	8.30	8.93	5.94
24	4.65	3.10	4.07	2.71	12.8	8.54	7.90	5.26	14.8	9.88	10.0	6.67
26	5.46	3.63	4.34	2.89	15.1	10.0	8.74	5.82	17.4	11.6	11.1	7.40
28	6.33	4.21	4.82	3.21								
30	7.27	4.84	5.31	3.53								
32	8.27	5.50	5.80	3.86								
34	9.34	6.21	6.29	4.18								
36	10.5	6.96	6.77	4.51								
38	11.7	7.76	7.26	4.83								
40	12.9	8.60	7.75	5.15								

Other Constants and Properties

	67^c		57		50^c	
$b_y \times 10^3$, (kip-ft)$^{-1}$	10.0	6.68	18.9	12.5	21.9	14.5
$t_y \times 10^3$, (kips)$^{-1}$	1.70	1.13	1.99	1.32	2.27	1.51
$t_r \times 10^3$, (kips)$^{-1}$	2.09	1.40	2.44	1.63	2.79	1.86
r_x/r_y	2.83		4.20		4.20	
r_y, in.	2.46		1.60		1.59	

[c] Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W16

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W16×											
		45^c				40^c				36^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	2.60	1.73	4.33	2.88	3.02	2.01	4.88	3.25	3.41	2.27	5.57	3.70
	6	2.97	1.98	4.40	2.93	3.46	2.30	4.96	3.30	3.93	2.62	5.71	3.80
	7	3.12	2.08	4.56	3.03	3.63	2.42	5.16	3.43	4.14	2.76	5.94	3.95
	8	3.30	2.20	4.74	3.15	3.84	2.55	5.36	3.57	4.39	2.92	6.20	4.12
	9	3.55	2.36	4.92	3.28	4.09	2.72	5.59	3.72	4.70	3.12	6.48	4.31
	10	3.85	2.56	5.13	3.41	4.39	2.92	5.83	3.88	5.06	3.37	6.78	4.51
	11	4.21	2.80	5.35	3.56	4.75	3.16	6.10	4.06	5.50	3.66	7.12	4.74
	12	4.65	3.09	5.59	3.72	5.24	3.48	6.39	4.25	6.07	4.04	7.49	4.98
	13	5.17	3.44	5.86	3.90	5.83	3.88	6.72	4.47	6.81	4.53	7.90	5.26
	14	5.80	3.86	6.15	4.09	6.54	4.35	7.07	4.71	7.70	5.12	8.36	5.56
	15	6.58	4.37	6.47	4.30	7.41	4.93	7.47	4.97	8.80	5.86	8.88	5.91
	16	7.48	4.98	6.82	4.54	8.43	5.61	7.96	5.30	10.0	6.66	9.79	6.51
	17	8.45	5.62	7.36	4.90	9.52	6.33	8.76	5.83	11.3	7.52	10.8	7.19
	18	9.47	6.30	8.03	5.34	10.7	7.10	9.58	6.38	12.7	8.43	11.9	7.89
	19	10.5	7.02	8.70	5.79	11.9	7.91	10.4	6.93	14.1	9.40	12.9	8.59
	20	11.7	7.78	9.37	6.23	13.2	8.77	11.2	7.48	15.6	10.4	14.0	9.31
	21	12.9	8.57	10.0	6.68	14.5	9.66	12.1	8.04	17.3	11.5	15.1	10.0
	22	14.1	9.41	10.7	7.14	15.9	10.6	12.9	8.61	18.9	12.6	16.2	10.8
	23	15.5	10.3	11.4	7.59	17.4	11.6	13.8	9.17	20.7	13.8	17.3	11.5
	24	16.8	11.2	12.1	8.04	19.0	12.6	14.6	9.74	22.5	15.0	18.4	12.2
	25	18.3	12.2	12.8	8.50	20.6	13.7	15.5	10.3	24.4	16.3	19.5	13.0
	26	19.8	13.1	13.5	8.95	22.3	14.8	16.4	10.9				

Other Constants and Properties

	45^c ASD	45^c LRFD	40^c ASD	40^c LRFD	36^c ASD	36^c LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	24.6	16.3	28.1	18.7	33.0	21.9
$t_y \times 10^3$, (kips)$^{-1}$	2.51	1.67	2.83	1.88	3.15	2.10
$t_r \times 10^3$, (kips)$^{-1}$	3.08	2.06	3.48	2.32	3.87	2.58
r_x/r_y	4.24		4.22		4.28	
r_y, in.	1.57		1.57		1.52	

^{c} Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

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W16–W14

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape	W16×							
	31[c]				26[c, v]			
Design	$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	4.08	2.71	6.60	4.39	5.04	3.35	8.06	5.36
6	5.16	3.43	7.28	4.85	6.50	4.32	9.07	6.03
7	5.62	3.74	7.71	5.13	7.12	4.74	9.66	6.43
8	6.20	4.12	8.19	5.45	7.92	5.27	10.3	6.87
9	6.93	4.61	8.74	5.82	8.92	5.93	11.1	7.39
10	7.89	5.25	9.37	6.23	10.2	6.78	12.0	7.99
11	9.28	6.17	10.1	6.71	12.0	8.01	13.1	8.69
12	11.0	7.34	11.1	7.35	14.3	9.53	15.0	10.0
13	13.0	8.62	12.6	8.39	16.8	11.2	17.2	11.5
14	15.0	10.0	14.2	9.45	19.5	13.0	19.5	13.0
15	17.2	11.5	15.8	10.5	22.4	14.9	21.9	14.6
16	19.6	13.1	17.5	11.6	25.5	16.9	24.3	16.2
17	22.2	14.7	19.2	12.8	28.7	19.1	26.7	17.8
18	24.8	16.5	20.9	13.9	32.2	21.4	29.2	19.4
19	27.7	18.4	22.6	15.0				

Other Constants and Properties

	31 ASD	31 LRFD	26 ASD	26 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	50.7	33.7	65.0	43.3
$t_y \times 10^3$, (kips)$^{-1}$	3.66	2.43	4.35	2.89
$t_r \times 10^3$, (kips)$^{-1}$	4.49	3.00	5.34	3.56
r_x/r_y	5.48		5.59	
r_y, in.	1.17		1.12	

[c] Shape is slender for compression for F_y = 50 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(1) with F_y = 50 ksi; therefore, ϕ_v = 0.90 and Ω_v = 1.67.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W14

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W14×											
		873[h]				808[h]				730[h]			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.130	0.0865	0.176	0.117	0.140	0.0934	0.195	0.130	0.155	0.103	0.215	0.143
	11	0.137	0.0912	0.176	0.117	0.148	0.0986	0.195	0.130	0.165	0.110	0.215	0.143
	12	0.138	0.0921	0.176	0.117	0.150	0.0996	0.195	0.130	0.166	0.111	0.215	0.143
	13	0.140	0.0931	0.176	0.117	0.151	0.101	0.195	0.130	0.168	0.112	0.215	0.143
	14	0.142	0.0942	0.176	0.117	0.153	0.102	0.195	0.130	0.171	0.114	0.215	0.143
	15	0.143	0.0954	0.176	0.117	0.155	0.103	0.195	0.130	0.173	0.115	0.215	0.143
	16	0.145	0.0967	0.176	0.117	0.158	0.105	0.195	0.130	0.176	0.117	0.215	0.143
	17	0.148	0.0982	0.176	0.117	0.160	0.106	0.195	0.130	0.178	0.119	0.215	0.143
	18	0.150	0.0997	0.176	0.117	0.162	0.108	0.195	0.130	0.181	0.121	0.215	0.143
	19	0.152	0.101	0.176	0.117	0.165	0.110	0.195	0.130	0.185	0.123	0.216	0.143
	20	0.155	0.103	0.176	0.117	0.168	0.112	0.196	0.130	0.188	0.125	0.216	0.144
	22	0.161	0.107	0.177	0.118	0.175	0.116	0.196	0.131	0.196	0.130	0.217	0.144
	24	0.167	0.111	0.177	0.118	0.182	0.121	0.197	0.131	0.205	0.136	0.217	0.145
	26	0.175	0.116	0.178	0.118	0.190	0.127	0.198	0.131	0.215	0.143	0.218	0.145
	28	0.183	0.122	0.178	0.119	0.200	0.133	0.198	0.132	0.226	0.150	0.219	0.146
	30	0.193	0.128	0.179	0.119	0.211	0.140	0.199	0.132	0.239	0.159	0.220	0.146
	32	0.204	0.135	0.179	0.119	0.223	0.148	0.199	0.133	0.254	0.169	0.221	0.147
	34	0.216	0.144	0.180	0.120	0.236	0.157	0.200	0.133	0.270	0.180	0.221	0.147
	36	0.229	0.153	0.181	0.120	0.252	0.168	0.201	0.134	0.289	0.192	0.222	0.148
	38	0.245	0.163	0.181	0.121	0.269	0.179	0.201	0.134	0.310	0.206	0.223	0.148
	40	0.262	0.174	0.182	0.121	0.289	0.192	0.202	0.134	0.334	0.222	0.224	0.149
	42	0.282	0.187	0.182	0.121	0.311	0.207	0.203	0.135	0.361	0.240	0.225	0.150
	44	0.304	0.202	0.183	0.122	0.336	0.224	0.203	0.135	0.392	0.261	0.226	0.150
	46	0.329	0.219	0.183	0.122	0.365	0.243	0.204	0.136	0.429	0.285	0.226	0.151
	48	0.358	0.238	0.184	0.122	0.398	0.265	0.205	0.136	0.467	0.311	0.227	0.151
	50	0.388	0.258	0.185	0.123	0.431	0.287	0.206	0.137	0.506	0.337	0.228	0.152

Other Constants and Properties

	873[h] ASD	873[h] LRFD	808[h] ASD	808[h] LRFD	730[h] ASD	730[h] LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	0.349	0.232	0.383	0.255	0.437	0.290
$t_y \times 10^3$, (kips)$^{-1}$	0.130	0.0865	0.140	0.0934	0.155	0.103
$t_r \times 10^3$, (kips)$^{-1}$	0.160	0.106	0.172	0.115	0.191	0.127
r_x/r_y	1.71		1.69		1.74	
r_y, in.	4.90		4.83		4.69	

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

W14

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W14×											
		665^h				605^h				550^h			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.170	0.113	0.241	0.160	0.188	0.125	0.270	0.180	0.206	0.137	0.302	0.201
	11	0.181	0.120	0.241	0.160	0.200	0.133	0.270	0.180	0.220	0.146	0.302	0.201
	12	0.183	0.122	0.241	0.160	0.202	0.134	0.270	0.180	0.222	0.148	0.302	0.201
	13	0.185	0.123	0.241	0.160	0.204	0.136	0.270	0.180	0.225	0.150	0.302	0.201
	14	0.188	0.125	0.241	0.160	0.207	0.138	0.270	0.180	0.228	0.152	0.302	0.201
	15	0.190	0.127	0.241	0.160	0.210	0.140	0.270	0.180	0.232	0.154	0.302	0.201
	16	0.193	0.129	0.241	0.160	0.214	0.142	0.270	0.180	0.236	0.157	0.302	0.201
	17	0.197	0.131	0.241	0.160	0.217	0.145	0.270	0.180	0.240	0.160	0.303	0.201
	18	0.200	0.133	0.242	0.161	0.221	0.147	0.271	0.180	0.244	0.162	0.303	0.202
	19	0.204	0.135	0.242	0.161	0.225	0.150	0.272	0.181	0.249	0.166	0.304	0.202
	20	0.208	0.138	0.242	0.161	0.230	0.153	0.272	0.181	0.254	0.169	0.305	0.203
	22	0.216	0.144	0.243	0.162	0.240	0.160	0.273	0.182	0.265	0.177	0.306	0.204
	24	0.226	0.151	0.244	0.163	0.252	0.167	0.274	0.183	0.279	0.185	0.308	0.205
	26	0.238	0.158	0.245	0.163	0.265	0.176	0.276	0.183	0.293	0.195	0.309	0.206
	28	0.251	0.167	0.246	0.164	0.280	0.186	0.277	0.184	0.310	0.207	0.310	0.207
	30	0.266	0.177	0.247	0.164	0.297	0.197	0.278	0.185	0.330	0.219	0.312	0.208
	32	0.282	0.188	0.248	0.165	0.316	0.210	0.279	0.186	0.352	0.234	0.313	0.209
	34	0.301	0.201	0.249	0.166	0.338	0.225	0.280	0.187	0.377	0.251	0.315	0.209
	36	0.323	0.215	0.250	0.166	0.363	0.241	0.282	0.187	0.406	0.270	0.316	0.210
	38	0.347	0.231	0.251	0.167	0.391	0.260	0.283	0.188	0.438	0.292	0.318	0.211
	40	0.375	0.250	0.252	0.168	0.423	0.282	0.284	0.189	0.475	0.316	0.319	0.213
	42	0.407	0.271	0.253	0.168	0.460	0.306	0.285	0.190	0.518	0.345	0.321	0.214
	44	0.443	0.295	0.254	0.169	0.503	0.335	0.287	0.191	0.568	0.378	0.322	0.215
	46	0.485	0.322	0.255	0.170	0.550	0.366	0.288	0.191	0.621	0.413	0.324	0.216
	48	0.528	0.351	0.256	0.171	0.599	0.399	0.289	0.192	0.676	0.450	0.326	0.217
	50	0.573	0.381	0.257	0.171	0.650	0.432	0.290	0.193	0.733	0.488	0.327	0.218
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		0.488		0.325		0.546		0.364		0.611		0.407	
$t_y \times 10^3$, (kips)$^{-1}$		0.170		0.113		0.188		0.125		0.206		0.137	
$t_r \times 10^3$, (kips)$^{-1}$		0.209		0.140		0.230		0.154		0.253		0.169	
r_x/r_y		1.73				1.71				1.70			
r_y, in.		4.62				4.55				4.49			

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

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W14

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W14×											
		500^h				455^h				426^h			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.227	0.151	0.339	0.226	0.249	0.166	0.381	0.253	0.267	0.178	0.410	0.273
	11	0.242	0.161	0.339	0.226	0.266	0.177	0.381	0.253	0.286	0.190	0.410	0.273
	12	0.245	0.163	0.339	0.226	0.270	0.179	0.381	0.253	0.290	0.193	0.410	0.273
	13	0.249	0.166	0.339	0.226	0.273	0.182	0.381	0.253	0.294	0.195	0.410	0.273
	14	0.252	0.168	0.339	0.226	0.278	0.185	0.381	0.253	0.298	0.198	0.410	0.273
	15	0.256	0.171	0.339	0.226	0.282	0.188	0.381	0.253	0.303	0.202	0.410	0.273
	16	0.261	0.173	0.340	0.226	0.287	0.191	0.381	0.254	0.308	0.205	0.411	0.273
	17	0.265	0.177	0.340	0.227	0.292	0.194	0.382	0.254	0.314	0.209	0.412	0.274
	18	0.270	0.180	0.341	0.227	0.298	0.198	0.383	0.255	0.320	0.213	0.413	0.275
	19	0.276	0.183	0.342	0.228	0.304	0.202	0.384	0.256	0.327	0.218	0.414	0.276
	20	0.282	0.187	0.343	0.228	0.310	0.207	0.385	0.256	0.334	0.222	0.415	0.276
	22	0.295	0.196	0.345	0.229	0.325	0.216	0.387	0.258	0.350	0.233	0.418	0.278
	24	0.309	0.206	0.346	0.230	0.342	0.227	0.389	0.259	0.369	0.245	0.420	0.280
	26	0.327	0.217	0.348	0.232	0.361	0.240	0.392	0.261	0.390	0.259	0.423	0.281
	28	0.346	0.230	0.350	0.233	0.383	0.255	0.394	0.262	0.414	0.276	0.425	0.283
	30	0.368	0.245	0.352	0.234	0.408	0.272	0.396	0.263	0.442	0.294	0.428	0.285
	32	0.394	0.262	0.353	0.235	0.437	0.291	0.398	0.265	0.474	0.315	0.430	0.286
	34	0.422	0.281	0.355	0.236	0.470	0.313	0.400	0.266	0.510	0.339	0.433	0.288
	36	0.455	0.303	0.357	0.238	0.508	0.338	0.403	0.268	0.551	0.367	0.435	0.290
	38	0.493	0.328	0.359	0.239	0.551	0.366	0.405	0.269	0.599	0.399	0.438	0.291
	40	0.536	0.357	0.361	0.240	0.600	0.399	0.407	0.271	0.654	0.435	0.441	0.293
	42	0.586	0.390	0.363	0.241	0.657	0.437	0.409	0.272	0.718	0.478	0.443	0.295
	44	0.643	0.428	0.365	0.243	0.721	0.480	0.412	0.274	0.788	0.524	0.446	0.297
	46	0.703	0.468	0.367	0.244	0.789	0.525	0.414	0.276	0.861	0.573	0.449	0.299
	48	0.765	0.509	0.369	0.245	0.859	0.571	0.417	0.277	0.938	0.624	0.452	0.300
	50	0.830	0.552	0.371	0.247	0.932	0.620	0.419	0.279	1.02	0.677	0.454	0.302

Other Constants and Properties

	500^h ASD	500^h LRFD	455^h ASD	455^h LRFD	426^h ASD	426^h LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	0.683	0.454	0.761	0.506	0.821	0.546
$t_y \times 10^3$, (kips)$^{-1}$	0.227	0.151	0.249	0.166	0.267	0.178
$t_r \times 10^3$, (kips)$^{-1}$	0.279	0.186	0.306	0.204	0.328	0.219
r_x/r_y	1.69		1.67		1.67	
r_y, in.	4.43		4.38		4.34	

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

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W14

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W14×											
		398^h				370^h				342^h			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.285	0.190	0.445	0.296	0.306	0.204	0.484	0.322	0.331	0.220	0.530	0.353
	11	0.306	0.203	0.445	0.296	0.329	0.219	0.484	0.322	0.355	0.236	0.530	0.353
	12	0.310	0.206	0.445	0.296	0.333	0.222	0.484	0.322	0.360	0.239	0.530	0.353
	13	0.314	0.209	0.445	0.296	0.338	0.225	0.484	0.322	0.365	0.243	0.530	0.353
	14	0.319	0.212	0.445	0.296	0.343	0.228	0.484	0.322	0.371	0.247	0.530	0.353
	15	0.324	0.216	0.445	0.296	0.349	0.232	0.484	0.322	0.377	0.251	0.530	0.353
	16	0.330	0.220	0.446	0.297	0.355	0.236	0.485	0.323	0.384	0.256	0.532	0.354
	17	0.336	0.224	0.447	0.298	0.362	0.241	0.487	0.324	0.392	0.261	0.534	0.355
	18	0.343	0.228	0.449	0.298	0.369	0.246	0.489	0.325	0.400	0.266	0.536	0.356
	19	0.350	0.233	0.450	0.299	0.377	0.251	0.490	0.326	0.409	0.272	0.538	0.358
	20	0.358	0.238	0.451	0.300	0.386	0.257	0.492	0.327	0.418	0.278	0.539	0.359
	22	0.376	0.250	0.454	0.302	0.405	0.270	0.495	0.329	0.439	0.292	0.543	0.361
	24	0.396	0.263	0.457	0.304	0.427	0.284	0.498	0.331	0.463	0.308	0.547	0.364
	26	0.419	0.279	0.460	0.306	0.453	0.301	0.501	0.334	0.491	0.327	0.551	0.367
	28	0.445	0.296	0.462	0.308	0.482	0.321	0.505	0.336	0.523	0.348	0.555	0.369
	30	0.475	0.316	0.465	0.310	0.515	0.343	0.508	0.338	0.560	0.373	0.559	0.372
	32	0.510	0.339	0.468	0.312	0.554	0.368	0.512	0.340	0.602	0.401	0.563	0.374
	34	0.550	0.366	0.471	0.314	0.597	0.397	0.515	0.343	0.651	0.433	0.567	0.377
	36	0.595	0.396	0.474	0.316	0.648	0.431	0.519	0.345	0.706	0.470	0.571	0.380
	38	0.647	0.431	0.477	0.318	0.705	0.469	0.522	0.347	0.770	0.513	0.575	0.383
	40	0.707	0.470	0.480	0.320	0.772	0.514	0.526	0.350	0.844	0.562	0.580	0.386
	42	0.778	0.517	0.484	0.322	0.850	0.566	0.529	0.352	0.931	0.619	0.584	0.389
	44	0.853	0.568	0.487	0.324	0.933	0.621	0.533	0.355	1.02	0.680	0.588	0.391
	46	0.933	0.621	0.490	0.326	1.02	0.679	0.537	0.357	1.12	0.743	0.593	0.394
	48	1.02	0.676	0.493	0.328	1.11	0.739	0.541	0.360	1.22	0.809	0.597	0.397
	50	1.10	0.733	0.496	0.330	1.21	0.802	0.545	0.362	1.32	0.878	0.602	0.401

Other Constants and Properties

	398^h ASD	398^h LRFD	370^h ASD	370^h LRFD	342^h ASD	342^h LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	0.886	0.590	0.963	0.641	1.05	0.701
$t_y \times 10^3$, (kips)$^{-1}$	0.285	0.190	0.306	0.204	0.331	0.220
$t_r \times 10^3$, (kips)$^{-1}$	0.351	0.234	0.376	0.251	0.406	0.271
r_x / r_y	1.66		1.66		1.65	
r_y, in.	4.31		4.27		4.24	

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

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W14

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape	W14×											
	311[h]				283[h]				257			
Design	$p\times10^3$ (kips)$^{-1}$		$b_x\times10^3$ (kip-ft)$^{-1}$		$p\times10^3$ (kips)$^{-1}$		$b_x\times10^3$ (kip-ft)$^{-1}$		$p\times10^3$ (kips)$^{-1}$		$b_x\times10^3$ (kip-ft)$^{-1}$	
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	0.365	0.243	0.591	0.393	0.401	0.267	0.657	0.437	0.442	0.294	0.732	0.487
11	0.393	0.261	0.591	0.393	0.431	0.287	0.657	0.437	0.476	0.317	0.732	0.487
12	0.398	0.265	0.591	0.393	0.437	0.291	0.657	0.437	0.483	0.321	0.732	0.487
13	0.404	0.269	0.591	0.393	0.444	0.296	0.657	0.437	0.490	0.326	0.732	0.487
14	0.411	0.273	0.591	0.393	0.451	0.300	0.657	0.437	0.499	0.332	0.732	0.487
15	0.418	0.278	0.591	0.393	0.459	0.306	0.658	0.438	0.508	0.338	0.733	0.488
16	0.426	0.283	0.593	0.395	0.468	0.312	0.661	0.440	0.517	0.344	0.736	0.490
17	0.434	0.289	0.596	0.396	0.478	0.318	0.663	0.441	0.528	0.351	0.740	0.492
18	0.443	0.295	0.598	0.398	0.488	0.325	0.666	0.443	0.540	0.359	0.743	0.494
19	0.453	0.302	0.600	0.399	0.499	0.332	0.669	0.445	0.552	0.367	0.746	0.497
20	0.464	0.309	0.602	0.401	0.511	0.340	0.672	0.447	0.566	0.376	0.750	0.499
22	0.488	0.325	0.607	0.404	0.537	0.358	0.677	0.451	0.596	0.396	0.757	0.503
24	0.515	0.343	0.612	0.407	0.568	0.378	0.683	0.455	0.630	0.419	0.764	0.508
26	0.547	0.364	0.617	0.410	0.604	0.402	0.689	0.458	0.671	0.446	0.771	0.513
28	0.583	0.388	0.621	0.413	0.645	0.429	0.695	0.462	0.717	0.477	0.778	0.518
30	0.625	0.416	0.626	0.417	0.691	0.460	0.701	0.466	0.770	0.512	0.786	0.523
32	0.673	0.448	0.631	0.420	0.745	0.496	0.707	0.471	0.831	0.553	0.794	0.528
34	0.729	0.485	0.636	0.423	0.807	0.537	0.713	0.475	0.902	0.600	0.801	0.533
36	0.792	0.527	0.641	0.427	0.879	0.585	0.720	0.479	0.983	0.654	0.809	0.539
38	0.865	0.576	0.647	0.430	0.961	0.640	0.726	0.483	1.08	0.717	0.818	0.544
40	0.951	0.633	0.652	0.434	1.06	0.704	0.733	0.488	1.19	0.791	0.826	0.549
42	1.05	0.697	0.657	0.437	1.17	0.776	0.740	0.492	1.31	0.872	0.834	0.555
44	1.15	0.765	0.663	0.441	1.28	0.852	0.747	0.497	1.44	0.957	0.843	0.561
46	1.26	0.837	0.669	0.445	1.40	0.931	0.754	0.501	1.57	1.05	0.852	0.567
48	1.37	0.911	0.674	0.449	1.52	1.01	0.761	0.506	1.71	1.14	0.861	0.573
50	1.49	0.988	0.680	0.452	1.65	1.10	0.768	0.511	1.86	1.24	0.870	0.579

Other Constants and Properties

	311[h] ASD	311[h] LRFD	283[h] ASD	283[h] LRFD	257 ASD	257 LRFD
$b_y\times10^3$, (kip-ft)$^{-1}$	1.17	0.780	1.30	0.865	1.45	0.964
$t_y\times10^3$, (kips)$^{-1}$	0.365	0.243	0.401	0.267	0.442	0.294
$t_r\times10^3$, (kips)$^{-1}$	0.449	0.299	0.493	0.328	0.543	0.362

	311[h]	283[h]	257
r_x/r_y	1.64	1.63	1.62
r_y, in.	4.20	4.17	4.13

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

W14

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W14×											
		233				211				193			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.488	0.324	0.817	0.544	0.539	0.358	0.914	0.608	0.588	0.391	1.00	0.668
	11	0.526	0.350	0.817	0.544	0.582	0.387	0.914	0.608	0.636	0.423	1.00	0.668
	12	0.534	0.355	0.817	0.544	0.590	0.393	0.914	0.608	0.645	0.429	1.00	0.668
	13	0.542	0.361	0.817	0.544	0.600	0.399	0.914	0.608	0.655	0.436	1.00	0.668
	14	0.551	0.367	0.817	0.544	0.610	0.406	0.914	0.608	0.667	0.444	1.00	0.668
	15	0.561	0.374	0.819	0.545	0.622	0.414	0.917	0.610	0.679	0.452	1.01	0.670
	16	0.572	0.381	0.823	0.548	0.634	0.422	0.922	0.613	0.693	0.461	1.01	0.675
	17	0.584	0.389	0.827	0.551	0.647	0.431	0.927	0.617	0.708	0.471	1.02	0.679
	18	0.597	0.397	0.832	0.553	0.662	0.440	0.932	0.620	0.724	0.482	1.03	0.683
	19	0.611	0.407	0.836	0.556	0.678	0.451	0.937	0.623	0.741	0.493	1.03	0.687
	20	0.626	0.417	0.840	0.559	0.695	0.462	0.942	0.627	0.760	0.506	1.04	0.691
	22	0.660	0.439	0.849	0.565	0.733	0.488	0.953	0.634	0.802	0.534	1.05	0.700
	24	0.699	0.465	0.857	0.571	0.777	0.517	0.964	0.641	0.851	0.566	1.07	0.709
	26	0.745	0.495	0.866	0.576	0.828	0.551	0.975	0.649	0.908	0.604	1.08	0.718
	28	0.797	0.530	0.876	0.583	0.887	0.590	0.987	0.656	0.973	0.647	1.09	0.727
	30	0.857	0.570	0.885	0.589	0.955	0.635	1.00	0.664	1.05	0.697	1.11	0.737
	32	0.926	0.616	0.895	0.595	1.03	0.687	1.01	0.672	1.13	0.755	1.12	0.747
	34	1.01	0.669	0.904	0.602	1.12	0.747	1.02	0.680	1.23	0.822	1.14	0.757
	36	1.10	0.731	0.914	0.608	1.23	0.817	1.04	0.689	1.35	0.899	1.15	0.767
	38	1.20	0.801	0.925	0.615	1.35	0.897	1.05	0.697	1.49	0.989	1.17	0.778
	40	1.33	0.886	0.935	0.622	1.49	0.993	1.06	0.706	1.65	1.09	1.19	0.789
	42	1.47	0.976	0.946	0.629	1.65	1.09	1.08	0.715	1.81	1.21	1.20	0.800
	44	1.61	1.07	0.957	0.637	1.81	1.20	1.09	0.725	1.99	1.32	1.22	0.812
	46	1.76	1.17	0.968	0.644	1.97	1.31	1.10	0.734	2.18	1.45	1.24	0.824
	48	1.92	1.28	0.979	0.652	2.15	1.43	1.12	0.744	2.37	1.58	1.26	0.836
	50	2.08	1.38	0.991	0.659	2.33	1.55	1.13	0.754	2.57	1.71	1.28	0.848
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		1.61		1.07		1.80		1.20		1.98		1.32	
$t_y \times 10^3$, (kips)$^{-1}$		0.488		0.324		0.539		0.358		0.588		0.391	
$t_r \times 10^3$, (kips)$^{-1}$		0.599		0.399		0.662		0.441		0.722		0.482	
r_x / r_y		1.62				1.61				1.60			
r_y, in.		4.10				4.07				4.05			

W14

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W14×											
		176				159				145			
Design		$p\times10^3$ (kips)$^{-1}$		$b_x\times10^3$ (kip-ft)$^{-1}$		$p\times10^3$ (kips)$^{-1}$		$b_x\times10^3$ (kip-ft)$^{-1}$		$p\times10^3$ (kips)$^{-1}$		$b_x\times10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.645	0.429	1.11	0.741	0.715	0.476	1.24	0.826	0.782	0.520	1.37	0.912
	11	0.698	0.464	1.11	0.741	0.774	0.515	1.24	0.826	0.848	0.564	1.37	0.912
	12	0.708	0.471	1.11	0.741	0.786	0.523	1.24	0.826	0.861	0.573	1.37	0.912
	13	0.720	0.479	1.11	0.741	0.799	0.532	1.24	0.826	0.875	0.582	1.37	0.912
	14	0.733	0.487	1.11	0.741	0.814	0.541	1.24	0.826	0.891	0.593	1.37	0.912
	15	0.747	0.497	1.12	0.745	0.829	0.552	1.25	0.831	0.908	0.604	1.38	0.919
	16	0.762	0.507	1.13	0.750	0.846	0.563	1.26	0.837	0.927	0.617	1.39	0.926
	17	0.778	0.518	1.13	0.755	0.865	0.576	1.27	0.843	0.948	0.631	1.40	0.933
	18	0.796	0.530	1.14	0.760	0.885	0.589	1.28	0.850	0.970	0.645	1.41	0.941
	19	0.816	0.543	1.15	0.765	0.907	0.603	1.29	0.856	0.994	0.662	1.43	0.949
	20	0.837	0.557	1.16	0.770	0.931	0.619	1.30	0.863	1.02	0.679	1.44	0.956
	22	0.884	0.588	1.17	0.781	0.983	0.654	1.32	0.876	1.08	0.718	1.46	0.973
	24	0.938	0.624	1.19	0.791	1.04	0.695	1.34	0.889	1.15	0.763	1.49	0.989
	26	1.00	0.666	1.21	0.803	1.12	0.742	1.36	0.904	1.23	0.816	1.51	1.01
	28	1.07	0.715	1.22	0.814	1.20	0.797	1.38	0.918	1.32	0.876	1.54	1.02
	30	1.16	0.771	1.24	0.826	1.29	0.860	1.40	0.933	1.42	0.947	1.57	1.04
	32	1.26	0.836	1.26	0.838	1.40	0.934	1.43	0.949	1.54	1.03	1.60	1.06
	34	1.37	0.911	1.28	0.851	1.53	1.02	1.45	0.965	1.69	1.12	1.63	1.08
	36	1.50	0.998	1.30	0.864	1.68	1.12	1.47	0.981	1.85	1.23	1.66	1.10
	38	1.65	1.10	1.32	0.877	1.85	1.23	1.50	0.998	2.05	1.36	1.69	1.12
	40	1.83	1.22	1.34	0.891	2.05	1.36	1.53	1.02	2.27	1.51	1.72	1.15
	42	2.02	1.34	1.36	0.905	2.26	1.50	1.56	1.03	2.50	1.66	1.76	1.17
	44	2.22	1.47	1.38	0.920	2.48	1.65	1.58	1.05	2.74	1.82	1.79	1.19
	46	2.42	1.61	1.41	0.935	2.71	1.81	1.61	1.07	3.00	1.99	1.83	1.22
	48	2.64	1.75	1.43	0.951	2.95	1.97	1.64	1.09	3.26	2.17	1.87	1.24
	50	2.86	1.90	1.45	0.967	3.21	2.13	1.68	1.12	3.54	2.36	1.91	1.27
Other Constants and Properties													
$b_y\times10^3$, (kip-ft)$^{-1}$		2.19		1.45		2.44		1.62		2.68		1.78	
$t_y\times10^3$, (kips)$^{-1}$		0.645		0.429		0.715		0.476		0.782		0.520	
$t_r\times10^3$, (kips)$^{-1}$		0.792		0.528		0.878		0.586		0.961		0.641	
r_x/r_y		1.60				1.60				1.59			
r_y, in.		4.02				4.00				3.98			

W14

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W14×											
		132				120				109			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.861	0.573	1.52	1.01	0.946	0.630	1.68	1.12	1.04	0.694	1.86	1.23
	11	0.942	0.627	1.52	1.01	1.04	0.690	1.68	1.12	1.14	0.761	1.86	1.23
	12	0.958	0.638	1.52	1.01	1.05	0.702	1.68	1.12	1.16	0.774	1.86	1.23
	13	0.976	0.650	1.52	1.01	1.07	0.715	1.68	1.12	1.19	0.789	1.86	1.23
	14	0.996	0.663	1.53	1.02	1.10	0.730	1.69	1.13	1.21	0.805	1.87	1.25
	15	1.02	0.677	1.55	1.03	1.12	0.746	1.71	1.14	1.24	0.823	1.89	1.26
	16	1.04	0.693	1.56	1.04	1.15	0.763	1.73	1.15	1.27	0.843	1.91	1.27
	17	1.07	0.710	1.57	1.05	1.18	0.783	1.74	1.16	1.30	0.864	1.93	1.29
	18	1.10	0.729	1.59	1.06	1.21	0.803	1.76	1.17	1.33	0.887	1.95	1.30
	19	1.13	0.749	1.60	1.07	1.24	0.826	1.78	1.18	1.37	0.913	1.98	1.31
	20	1.16	0.771	1.62	1.08	1.28	0.851	1.80	1.20	1.41	0.940	2.00	1.33
	22	1.23	0.821	1.65	1.10	1.36	0.906	1.84	1.22	1.51	1.00	2.04	1.36
	24	1.32	0.880	1.68	1.12	1.46	0.971	1.88	1.25	1.61	1.07	2.09	1.39
	26	1.42	0.948	1.71	1.14	1.57	1.05	1.92	1.28	1.74	1.16	2.14	1.43
	28	1.54	1.03	1.75	1.16	1.71	1.14	1.96	1.30	1.89	1.26	2.20	1.46
	30	1.68	1.12	1.79	1.19	1.86	1.24	2.00	1.33	2.06	1.37	2.25	1.50
	32	1.85	1.23	1.82	1.21	2.05	1.36	2.05	1.37	2.27	1.51	2.31	1.54
	34	2.04	1.35	1.86	1.24	2.26	1.50	2.10	1.40	2.50	1.67	2.37	1.58
	36	2.26	1.51	1.90	1.27	2.51	1.67	2.15	1.43	2.79	1.86	2.44	1.62
	38	2.52	1.68	1.95	1.29	2.80	1.86	2.21	1.47	3.11	2.07	2.51	1.67
	40	2.79	1.86	1.99	1.32	3.10	2.07	2.27	1.51	3.44	2.29	2.58	1.72
	42	3.08	2.05	2.04	1.36	3.42	2.28	2.33	1.55	3.80	2.53	2.66	1.77
	44	3.38	2.25	2.09	1.39	3.76	2.50	2.39	1.59	4.17	2.77	2.74	1.82
	46	3.70	2.46	2.14	1.42	4.11	2.73	2.46	1.63	4.55	3.03	2.82	1.88
	48	4.02	2.68	2.19	1.46	4.47	2.97	2.53	1.68	4.96	3.30	2.92	1.94
	50	4.37	2.91	2.25	1.50	4.85	3.23	2.60	1.73	5.38	3.58	3.05	2.03
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		3.15		2.10		3.49		2.32		3.84		2.56	
$t_y \times 10^3$, (kips)$^{-1}$		0.861		0.573		0.946		0.630		1.04		0.694	
$t_r \times 10^3$, (kips)$^{-1}$		1.06		0.705		1.16		0.775		1.28		0.855	
r_x / r_y		1.67				1.67				1.67			
r_y, in.		3.76				3.74				3.73			

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W14

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W14×			
		99[f]			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	1.15	0.764	2.07	1.38
	11	1.26	0.838	2.07	1.38
	12	1.28	0.853	2.07	1.38
	13	1.31	0.869	2.07	1.38
	14	1.33	0.887	2.08	1.38
	15	1.36	0.907	2.10	1.40
	16	1.40	0.929	2.13	1.42
	17	1.43	0.953	2.15	1.43
	18	1.47	0.978	2.18	1.45
	19	1.51	1.01	2.21	1.47
	20	1.56	1.04	2.23	1.49
	22	1.66	1.11	2.29	1.52
	24	1.78	1.19	2.35	1.56
	26	1.92	1.28	2.41	1.60
	28	2.09	1.39	2.48	1.65
	30	2.28	1.52	2.55	1.69
	32	2.51	1.67	2.62	1.74
	34	2.78	1.85	2.70	1.80
	36	3.10	2.06	2.78	1.85
	38	3.45	2.30	2.87	1.91
	40	3.83	2.55	2.96	1.97
	42	4.22	2.81	3.06	2.04
	44	4.63	3.08	3.17	2.11
	46	5.06	3.37	3.31	2.20
	48	5.51	3.67	3.48	2.32
	50	5.98	3.98	3.66	2.43

Other Constants and Properties

	ASD	LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	4.29	2.85
$t_y \times 10^3$, (kips)$^{-1}$	1.15	0.764
$t_r \times 10^3$, (kips)$^{-1}$	1.41	0.940
r_x/r_y	1.66	
r_y, in.	3.71	

[f] Shape does not meet compact limit for flexure for F_y = 50 ksi.

W14

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W14×											
		90[f]				82				74			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	1.26	0.839	2.33	1.55	1.39	0.926	2.56	1.71	1.53	1.02	2.83	1.88
	6	1.30	0.862	2.33	1.55	1.48	0.985	2.56	1.71	1.63	1.08	2.83	1.88
	7	1.31	0.871	2.33	1.55	1.51	1.01	2.56	1.71	1.67	1.11	2.83	1.88
	8	1.32	0.881	2.33	1.55	1.55	1.03	2.56	1.71	1.71	1.14	2.83	1.88
	9	1.34	0.892	2.33	1.55	1.60	1.06	2.57	1.71	1.76	1.17	2.84	1.89
	10	1.36	0.906	2.33	1.55	1.65	1.10	2.61	1.74	1.82	1.21	2.89	1.92
	11	1.38	0.920	2.33	1.55	1.71	1.14	2.66	1.77	1.88	1.25	2.94	1.96
	12	1.41	0.937	2.33	1.55	1.78	1.18	2.70	1.80	1.96	1.30	2.99	1.99
	13	1.44	0.955	2.33	1.55	1.86	1.24	2.74	1.83	2.05	1.36	3.05	2.03
	14	1.47	0.975	2.33	1.55	1.95	1.30	2.79	1.86	2.14	1.43	3.10	2.06
	15	1.50	0.997	2.33	1.55	2.05	1.36	2.84	1.89	2.25	1.50	3.16	2.10
	16	1.53	1.02	2.35	1.57	2.16	1.44	2.89	1.92	2.37	1.58	3.22	2.14
	17	1.57	1.05	2.38	1.59	2.28	1.52	2.94	1.96	2.51	1.67	3.29	2.19
	18	1.62	1.08	2.42	1.61	2.42	1.61	2.99	1.99	2.67	1.78	3.35	2.23
	19	1.66	1.11	2.45	1.63	2.58	1.72	3.05	2.03	2.84	1.89	3.42	2.28
	20	1.71	1.14	2.48	1.65	2.76	1.84	3.11	2.07	3.04	2.02	3.49	2.32
	22	1.83	1.22	2.55	1.70	3.19	2.12	3.23	2.15	3.51	2.33	3.65	2.43
	24	1.96	1.31	2.62	1.74	3.74	2.49	3.36	2.24	4.12	2.74	3.81	2.54
	26	2.12	1.41	2.70	1.80	4.39	2.92	3.51	2.33	4.83	3.21	3.99	2.66
	28	2.30	1.53	2.78	1.85	5.09	3.39	3.66	2.44	5.60	3.73	4.20	2.79
	30	2.52	1.68	2.87	1.91	5.84	3.89	3.83	2.55	6.43	4.28	4.42	2.94
	32	2.77	1.84	2.96	1.97	6.65	4.42	4.02	2.67	7.32	4.87	4.72	3.14
	34	3.07	2.04	3.06	2.03	7.50	4.99	4.26	2.84	8.26	5.50	5.07	3.38
	36	3.42	2.28	3.16	2.10	8.41	5.60	4.56	3.03	9.26	6.16	5.43	3.61
	38	3.81	2.54	3.27	2.18	9.37	6.24	4.85	3.22	10.3	6.86	5.78	3.85
	40	4.23	2.81	3.39	2.26	10.4	6.91	5.14	3.42	11.4	7.61	6.14	4.08

Other Constants and Properties

	90[f] ASD	90[f] LRFD	82 ASD	82 LRFD	74 ASD	74 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	4.90	3.26	7.95	5.29	8.80	5.85
$t_y \times 10^3$, (kips)$^{-1}$	1.26	0.839	1.39	0.926	1.53	1.02
$t_r \times 10^3$, (kips)$^{-1}$	1.55	1.03	1.71	1.14	1.88	1.25
r_x / r_y	1.66		2.44		2.44	
r_y, in.	3.70		2.48		2.48	

[f] Shape does not meet compact limit for flexure for F_y = 50 ksi.

W14

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W14×											
		68				61				53			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	1.67	1.11	3.10	2.06	1.87	1.24	3.49	2.32	2.14	1.42	4.09	2.72
	6	1.78	1.18	3.10	2.06	1.99	1.32	3.49	2.32	2.37	1.58	4.09	2.72
	7	1.82	1.21	3.10	2.06	2.03	1.35	3.49	2.32	2.46	1.64	4.11	2.74
	8	1.87	1.24	3.10	2.06	2.09	1.39	3.49	2.32	2.57	1.71	4.21	2.80
	9	1.92	1.28	3.12	2.07	2.15	1.43	3.52	2.34	2.70	1.80	4.32	2.88
	10	1.99	1.32	3.17	2.11	2.22	1.48	3.59	2.39	2.85	1.90	4.44	2.95
	11	2.06	1.37	3.23	2.15	2.31	1.54	3.66	2.44	3.02	2.01	4.56	3.03
	12	2.15	1.43	3.30	2.19	2.40	1.60	3.74	2.49	3.23	2.15	4.68	3.11
	13	2.24	1.49	3.36	2.24	2.51	1.67	3.82	2.54	3.47	2.31	4.81	3.20
	14	2.35	1.56	3.43	2.28	2.63	1.75	3.90	2.59	3.75	2.49	4.96	3.30
	15	2.47	1.64	3.50	2.33	2.77	1.84	3.99	2.65	4.07	2.71	5.11	3.40
	16	2.61	1.73	3.57	2.38	2.92	1.95	4.08	2.71	4.45	2.96	5.26	3.50
	17	2.76	1.84	3.65	2.43	3.10	2.06	4.17	2.78	4.89	3.25	5.43	3.62
	18	2.93	1.95	3.73	2.48	3.29	2.19	4.27	2.84	5.40	3.59	5.61	3.74
	19	3.13	2.08	3.81	2.53	3.51	2.34	4.38	2.91	6.01	4.00	5.81	3.86
	20	3.35	2.23	3.90	2.59	3.76	2.50	4.49	2.98	6.66	4.43	6.01	4.00
	22	3.88	2.58	4.08	2.72	4.36	2.90	4.72	3.14	8.06	5.36	6.47	4.31
	24	4.56	3.03	4.29	2.85	5.14	3.42	4.99	3.32	9.60	6.38	7.22	4.80
	26	5.35	3.56	4.51	3.00	6.03	4.01	5.28	3.51	11.3	7.49	7.99	5.32
	28	6.21	4.13	4.77	3.17	6.99	4.65	5.66	3.77	13.1	8.69	8.76	5.83
	30	7.12	4.74	5.10	3.39	8.02	5.34	6.20	4.13	15.0	9.98	9.53	6.34
	32	8.11	5.39	5.53	3.68	9.13	6.07	6.74	4.48	17.1	11.3	10.3	6.85
	34	9.15	6.09	5.96	3.96	10.3	6.86	7.27	4.84				
	36	10.3	6.83	6.38	4.25	11.6	7.69	7.81	5.20				
	38	11.4	7.60	6.81	4.53	12.9	8.57	8.34	5.55				
	40	12.7	8.43	7.23	4.81	14.3	9.49	8.87	5.90				

Other Constants and Properties

	68 ASD	68 LRFD	61 ASD	61 LRFD	53 ASD	53 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	9.65	6.42	10.9	7.23	16.2	10.8
$t_y \times 10^3$, (kips)$^{-1}$	1.67	1.11	1.87	1.24	2.14	1.42
$t_r \times 10^3$, (kips)$^{-1}$	2.05	1.37	2.29	1.53	2.63	1.75
r_x/r_y	2.44		2.44		3.07	
r_y, in.	2.46		2.45		1.92	

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W14

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W14×											
		48				43^c				38^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	2.37	1.58	4.54	3.02	2.67	1.78	5.12	3.41	3.05	2.03	5.79	3.85
	6	2.63	1.75	4.54	3.02	2.95	1.96	5.12	3.41	3.51	2.33	5.90	3.93
	7	2.73	1.82	4.57	3.04	3.06	2.04	5.17	3.44	3.70	2.46	6.12	4.07
	8	2.85	1.90	4.70	3.13	3.20	2.13	5.31	3.54	3.95	2.63	6.36	4.23
	9	2.99	1.99	4.83	3.21	3.37	2.24	5.47	3.64	4.25	2.83	6.61	4.40
	10	3.16	2.10	4.96	3.30	3.56	2.37	5.64	3.75	4.62	3.08	6.89	4.58
	11	3.36	2.23	5.11	3.40	3.79	2.52	5.82	3.87	5.07	3.37	7.19	4.78
	12	3.59	2.39	5.26	3.50	4.05	2.70	6.01	4.00	5.61	3.73	7.52	5.00
	13	3.86	2.57	5.43	3.61	4.36	2.90	6.21	4.13	6.25	4.16	7.88	5.24
	14	4.17	2.77	5.60	3.73	4.72	3.14	6.42	4.27	7.04	4.68	8.27	5.50
	15	4.53	3.02	5.79	3.85	5.15	3.42	6.66	4.43	8.01	5.33	8.71	5.80
	16	4.96	3.30	5.98	3.98	5.64	3.75	6.90	4.59	9.11	6.06	9.20	6.12
	17	5.45	3.63	6.20	4.12	6.21	4.13	7.17	4.77	10.3	6.85	9.99	6.65
	18	6.03	4.01	6.42	4.27	6.90	4.59	7.46	4.97	11.5	7.68	10.9	7.23
	19	6.72	4.47	6.67	4.44	7.68	5.11	7.78	5.17	12.9	8.55	11.8	7.82
	20	7.45	4.96	6.94	4.61	8.51	5.66	8.12	5.40	14.2	9.48	12.6	8.41
	21	8.21	5.46	7.22	4.80	9.39	6.25	8.71	5.80	15.7	10.4	13.5	9.00
	22	9.01	6.00	7.69	5.12	10.3	6.85	9.31	6.19	17.2	11.5	14.4	9.60
	23	9.85	6.56	8.16	5.43	11.3	7.49	9.90	6.59	18.8	12.5	15.3	10.2
	24	10.7	7.14	8.64	5.75	12.3	8.16	10.5	6.99	20.5	13.6	16.2	10.8
	25	11.6	7.74	9.12	6.07	13.3	8.85	11.1	7.39	22.3	14.8	17.1	11.4
	26	12.6	8.38	9.59	6.38	14.4	9.57	11.7	7.78				
	27	13.6	9.03	10.1	6.70	15.5	10.3	12.3	8.18				
	28	14.6	9.72	10.5	7.01	16.7	11.1	12.9	8.58				
	29	15.7	10.4	11.0	7.33	17.9	11.9	13.5	8.98				
	30	16.8	11.2	11.5	7.65	19.2	12.7	14.1	9.37				

Other Constants and Properties

	48		43^c		38^c	
$b_y \times 10^3$, (kip-ft)$^{-1}$	18.2	12.1	20.6	13.7	29.4	19.6
$t_y \times 10^3$, (kips)$^{-1}$	2.37	1.58	2.65	1.76	2.98	1.98
$t_r \times 10^3$, (kips)$^{-1}$	2.91	1.94	3.26	2.17	3.66	2.44
r_x/r_y	3.06		3.08		3.79	
r_y, in.	1.91		1.89		1.55	

^{c} Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W14

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W14×											
		34^c				30^c				26^c			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	3.49	2.32	6.53	4.34	4.01	2.67	7.53	5.01	4.72	3.14	8.86	5.90
	6	4.03	2.68	6.67	4.44	4.65	3.10	7.76	5.16	6.24	4.15	10.0	6.67
	7	4.24	2.82	6.94	4.61	4.91	3.27	8.09	5.38	6.90	4.59	10.7	7.10
	8	4.50	2.99	7.22	4.80	5.22	3.48	8.44	5.62	7.75	5.16	11.4	7.59
	9	4.81	3.20	7.53	5.01	5.60	3.73	8.83	5.88	9.02	6.00	12.3	8.15
	10	5.24	3.48	7.87	5.23	6.06	4.03	9.26	6.16	10.7	7.13	13.2	8.80
	11	5.76	3.83	8.24	5.48	6.70	4.46	9.74	6.48	12.9	8.60	14.4	9.56
	12	6.38	4.25	8.64	5.75	7.47	4.97	10.3	6.83	15.4	10.2	16.5	11.0
	13	7.14	4.75	9.09	6.05	8.41	5.60	10.8	7.21	18.1	12.0	18.7	12.4
	14	8.07	5.37	9.58	6.37	9.56	6.36	11.5	7.65	20.9	13.9	20.9	13.9
	15	9.21	6.13	10.1	6.74	11.0	7.30	12.3	8.20	24.0	16.0	23.2	15.4
	16	10.5	6.97	11.0	7.29	12.5	8.31	13.7	9.12	27.3	18.2	25.5	17.0
	17	11.8	7.87	12.0	8.01	14.1	9.38	15.1	10.0	30.9	20.5	27.8	18.5
	18	13.3	8.82	13.1	8.73	15.8	10.5	16.5	11.0	34.6	23.0	30.1	20.0
	19	14.8	9.83	14.2	9.47	17.6	11.7	18.0	12.0				
	20	16.4	10.9	15.3	10.2	19.5	13.0	19.4	12.9				
	21	18.0	12.0	16.5	11.0	21.5	14.3	20.9	13.9				
	22	19.8	13.2	17.6	11.7	23.6	15.7	22.4	14.9				
	23	21.6	14.4	18.7	12.4	25.8	17.2	23.9	15.9				
	24	23.6	15.7	19.8	13.2	28.1	18.7	25.4	16.9				
	25	25.6	17.0	21.0	13.9								

Other Constants and Properties

	34^c ASD	34^c LRFD	30^c ASD	30^c LRFD	26^c ASD	26^c LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	33.6	22.4	39.6	26.4	64.3	42.8
$t_y \times 10^3$, (kips)$^{-1}$	3.34	2.22	3.77	2.51	4.34	2.89
$t_r \times 10^3$, (kips)$^{-1}$	4.10	2.74	4.64	3.09	5.33	3.56
r_x / r_y	3.81		3.85		5.23	
r_y, in.	1.53		1.49		1.08	

[c] Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c / r_y equal to or greater than 200.

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W14

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W14×			
		22^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	5.80	3.86	10.7	7.14
	6	7.81	5.20	12.4	8.24
	7	8.70	5.79	13.3	8.83
	8	9.84	6.55	14.3	9.51
	9	11.3	7.54	15.5	10.3
	10	13.6	9.08	16.9	11.2
	11	16.5	11.0	19.2	12.8
	12	19.7	13.1	22.3	14.8
	13	23.1	15.3	25.4	16.9
	14	26.8	17.8	28.5	19.0
	15	30.7	20.4	31.8	21.2
	16	34.9	23.2	35.1	23.3
	17	39.4	26.2	38.4	25.6

Other Constants and Properties

	ASD	LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	81.2	54.0
$t_y \times 10^3$, (kips)$^{-1}$	5.15	3.42
$t_r \times 10^3$, (kips)$^{-1}$	6.32	4.21
r_x/r_y	5.33	
r_y, in.	1.04	

[c] Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W12

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W12×											
		336[h]				305[h]				279[h]			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.338	0.225	0.591	0.393	0.373	0.248	0.663	0.441	0.408	0.271	0.741	0.493
	6	0.349	0.232	0.591	0.393	0.385	0.256	0.663	0.441	0.422	0.280	0.741	0.493
	7	0.352	0.235	0.591	0.393	0.390	0.259	0.663	0.441	0.427	0.284	0.741	0.493
	8	0.357	0.238	0.591	0.393	0.395	0.263	0.663	0.441	0.433	0.288	0.741	0.493
	9	0.363	0.241	0.591	0.393	0.401	0.267	0.663	0.441	0.439	0.292	0.741	0.493
	10	0.369	0.245	0.591	0.393	0.408	0.272	0.663	0.441	0.447	0.298	0.741	0.493
	11	0.375	0.250	0.591	0.393	0.416	0.277	0.663	0.441	0.456	0.303	0.741	0.493
	12	0.383	0.255	0.591	0.393	0.425	0.283	0.663	0.441	0.466	0.310	0.741	0.493
	13	0.391	0.260	0.592	0.394	0.435	0.289	0.666	0.443	0.477	0.317	0.744	0.495
	14	0.401	0.267	0.594	0.395	0.445	0.296	0.668	0.444	0.489	0.325	0.746	0.497
	15	0.411	0.274	0.596	0.397	0.457	0.304	0.670	0.446	0.502	0.334	0.749	0.499
	16	0.422	0.281	0.598	0.398	0.470	0.313	0.673	0.448	0.516	0.344	0.752	0.500
	17	0.435	0.289	0.600	0.399	0.484	0.322	0.675	0.449	0.532	0.354	0.755	0.502
	18	0.448	0.298	0.602	0.400	0.500	0.332	0.677	0.451	0.550	0.366	0.758	0.504
	19	0.463	0.308	0.604	0.402	0.516	0.344	0.680	0.452	0.569	0.378	0.761	0.506
	20	0.479	0.319	0.606	0.403	0.535	0.356	0.682	0.454	0.590	0.392	0.764	0.508
	22	0.516	0.343	0.610	0.406	0.577	0.384	0.687	0.457	0.637	0.424	0.770	0.512
	24	0.559	0.372	0.614	0.408	0.627	0.417	0.692	0.461	0.693	0.461	0.776	0.516
	26	0.610	0.406	0.618	0.411	0.686	0.456	0.697	0.464	0.760	0.506	0.782	0.520
	28	0.670	0.446	0.622	0.414	0.756	0.503	0.702	0.467	0.840	0.559	0.788	0.524
	30	0.742	0.494	0.626	0.417	0.839	0.558	0.708	0.471	0.935	0.622	0.795	0.529
	32	0.827	0.550	0.630	0.419	0.938	0.624	0.713	0.474	1.05	0.698	0.801	0.533
	34	0.930	0.619	0.635	0.422	1.06	0.704	0.718	0.478	1.18	0.788	0.808	0.537
	36	1.04	0.694	0.639	0.425	1.19	0.789	0.724	0.481	1.33	0.883	0.814	0.542
	38	1.16	0.773	0.644	0.428	1.32	0.879	0.729	0.485	1.48	0.984	0.821	0.546
	40	1.29	0.856	0.648	0.431	1.46	0.974	0.735	0.489	1.64	1.09	0.828	0.551

Other Constants and Properties

	336[h] ASD	336[h] LRFD	305[h] ASD	305[h] LRFD	279[h] ASD	279[h] LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	1.30	0.865	1.46	0.971	1.62	1.08
$t_y \times 10^3$, (kips)$^{-1}$	0.338	0.225	0.373	0.248	0.408	0.271
$t_r \times 10^3$, (kips)$^{-1}$	0.415	0.277	0.458	0.306	0.501	0.334
r_x / r_y	1.85		1.84		1.82	
r_y, in.	3.47		3.42		3.38	

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

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W12

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W12×											
		252^h				230^h				210			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	0.451	0.300	0.832	0.554	0.493	0.328	0.923	0.614	0.540	0.360	1.02	0.681
	6	0.466	0.310	0.832	0.554	0.511	0.340	0.923	0.614	0.560	0.372	1.02	0.681
	7	0.472	0.314	0.832	0.554	0.517	0.344	0.923	0.614	0.567	0.377	1.02	0.681
	8	0.479	0.319	0.832	0.554	0.525	0.349	0.923	0.614	0.575	0.383	1.02	0.681
	9	0.487	0.324	0.832	0.554	0.533	0.355	0.923	0.614	0.585	0.389	1.02	0.681
	10	0.495	0.330	0.832	0.554	0.543	0.361	0.923	0.614	0.596	0.397	1.02	0.681
	11	0.505	0.336	0.832	0.554	0.554	0.369	0.923	0.614	0.608	0.405	1.02	0.681
	12	0.516	0.344	0.833	0.554	0.567	0.377	0.924	0.615	0.622	0.414	1.03	0.683
	13	0.529	0.352	0.837	0.557	0.580	0.386	0.928	0.618	0.638	0.424	1.03	0.686
	14	0.542	0.361	0.840	0.559	0.596	0.396	0.933	0.621	0.655	0.436	1.04	0.689
	15	0.557	0.371	0.844	0.561	0.612	0.407	0.937	0.623	0.674	0.448	1.04	0.693
	16	0.574	0.382	0.847	0.564	0.631	0.420	0.941	0.626	0.694	0.462	1.05	0.696
	17	0.592	0.394	0.851	0.566	0.651	0.433	0.946	0.629	0.717	0.477	1.05	0.700
	18	0.612	0.407	0.854	0.568	0.674	0.448	0.950	0.632	0.742	0.494	1.06	0.703
	19	0.634	0.422	0.858	0.571	0.698	0.464	0.954	0.635	0.769	0.512	1.06	0.707
	20	0.657	0.437	0.862	0.573	0.725	0.482	0.959	0.638	0.799	0.532	1.07	0.710
	22	0.712	0.474	0.869	0.578	0.786	0.523	0.968	0.644	0.868	0.577	1.08	0.718
	24	0.776	0.516	0.877	0.583	0.858	0.571	0.977	0.650	0.950	0.632	1.09	0.725
	26	0.853	0.568	0.884	0.588	0.945	0.629	0.986	0.656	1.05	0.697	1.10	0.733
	28	0.945	0.629	0.892	0.594	1.05	0.697	0.996	0.663	1.16	0.775	1.11	0.741
	30	1.05	0.701	0.900	0.599	1.17	0.780	1.01	0.669	1.30	0.868	1.13	0.749
	32	1.19	0.790	0.908	0.604	1.32	0.880	1.02	0.676	1.48	0.982	1.14	0.757
	34	1.34	0.891	0.916	0.610	1.49	0.993	1.03	0.682	1.67	1.11	1.15	0.765
	36	1.50	0.999	0.925	0.615	1.67	1.11	1.04	0.689	1.87	1.24	1.16	0.774
	38	1.67	1.11	0.933	0.621	1.87	1.24	1.05	0.696	2.08	1.38	1.18	0.782
	40	1.85	1.23	0.942	0.627	2.07	1.37	1.06	0.704	2.31	1.53	1.19	0.791
Other Constants and Properties													
$b_y \times 10^3$, $(\text{kip-ft})^{-1}$		1.82		1.21		2.01		1.34		2.24		1.49	
$t_y \times 10^3$, $(\text{kips})^{-1}$		0.451		0.300		0.493		0.328		0.540		0.360	
$t_r \times 10^3$, $(\text{kips})^{-1}$		0.554		0.369		0.606		0.404		0.664		0.443	
r_x/r_y		1.81				1.80				1.80			
r_y, in.		3.34				3.31				3.28			

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

W12

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

***F*$_y$ = 50 ksi**

Shape	W12×											
	190				170				152			
Design	$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	0.596	0.397	1.15	0.762	0.668	0.444	1.30	0.862	0.747	0.497	1.47	0.975
6	0.618	0.411	1.15	0.762	0.693	0.461	1.30	0.862	0.776	0.516	1.47	0.975
7	0.626	0.417	1.15	0.762	0.702	0.467	1.30	0.862	0.786	0.523	1.47	0.975
8	0.636	0.423	1.15	0.762	0.713	0.474	1.30	0.862	0.798	0.531	1.47	0.975
9	0.647	0.430	1.15	0.762	0.725	0.483	1.30	0.862	0.813	0.541	1.47	0.975
10	0.659	0.438	1.15	0.762	0.739	0.492	1.30	0.862	0.829	0.551	1.47	0.975
11	0.673	0.448	1.15	0.762	0.755	0.503	1.30	0.862	0.847	0.563	1.47	0.975
12	0.688	0.458	1.15	0.764	0.773	0.514	1.30	0.865	0.867	0.577	1.47	0.980
13	0.706	0.470	1.16	0.768	0.793	0.528	1.31	0.870	0.890	0.592	1.48	0.987
14	0.725	0.482	1.16	0.773	0.815	0.542	1.32	0.876	0.915	0.609	1.49	0.994
15	0.746	0.497	1.17	0.777	0.839	0.559	1.32	0.881	0.943	0.627	1.50	1.00
16	0.770	0.512	1.17	0.781	0.866	0.576	1.33	0.887	0.974	0.648	1.51	1.01
17	0.796	0.529	1.18	0.786	0.896	0.596	1.34	0.892	1.01	0.670	1.52	1.01
18	0.824	0.548	1.19	0.790	0.928	0.618	1.35	0.898	1.04	0.695	1.54	1.02
19	0.855	0.569	1.19	0.794	0.964	0.641	1.36	0.903	1.09	0.722	1.55	1.03
20	0.889	0.591	1.20	0.799	1.00	0.667	1.37	0.909	1.13	0.752	1.56	1.04
22	0.966	0.643	1.21	0.808	1.09	0.727	1.38	0.921	1.23	0.820	1.58	1.05
24	1.06	0.705	1.23	0.817	1.20	0.798	1.40	0.932	1.36	0.902	1.60	1.07
26	1.17	0.778	1.24	0.827	1.33	0.883	1.42	0.945	1.50	1.00	1.63	1.08
28	1.30	0.867	1.26	0.837	1.48	0.985	1.44	0.957	1.68	1.12	1.65	1.10
30	1.46	0.973	1.27	0.847	1.67	1.11	1.46	0.970	1.90	1.26	1.68	1.12
32	1.66	1.10	1.29	0.857	1.89	1.26	1.48	0.983	2.16	1.43	1.70	1.13
34	1.87	1.25	1.30	0.867	2.14	1.42	1.50	0.997	2.43	1.62	1.73	1.15
36	2.10	1.40	1.32	0.878	2.39	1.59	1.52	1.01	2.73	1.82	1.76	1.17
38	2.34	1.56	1.34	0.889	2.67	1.78	1.54	1.03	3.04	2.02	1.79	1.19
40	2.59	1.72	1.35	0.900	2.96	1.97	1.56	1.04	3.37	2.24	1.82	1.21

Other Constants and Properties

	190 ASD	190 LRFD	170 ASD	170 LRFD	152 ASD	152 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	2.49	1.66	2.83	1.88	3.21	2.14
$t_y \times 10^3$, (kips)$^{-1}$	0.596	0.397	0.668	0.444	0.747	0.497
$t_r \times 10^3$, (kips)$^{-1}$	0.733	0.488	0.821	0.547	0.918	0.612
r_x / r_y	1.79		1.78		1.77	
r_y, in.	3.25		3.22		3.19	

W12

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		**W12×**											
		136				**120**				**106**			
Design		**$p\times10^3$ (kips)$^{-1}$**		**$b_x\times10^3$ (kip-ft)$^{-1}$**		**$p\times10^3$ (kips)$^{-1}$**		**$b_x\times10^3$ (kip-ft)$^{-1}$**		**$p\times10^3$ (kips)$^{-1}$**		**$b_x\times10^3$ (kip-ft)$^{-1}$**	
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	**0**	0.837	0.557	1.66	1.11	0.949	0.631	1.92	1.27	1.07	0.712	2.17	1.45
	6	0.869	0.578	1.66	1.11	0.986	0.656	1.92	1.27	1.11	0.741	2.17	1.45
	7	0.881	0.586	1.66	1.11	1.00	0.665	1.92	1.27	1.13	0.751	2.17	1.45
	8	0.896	0.596	1.66	1.11	1.02	0.676	1.92	1.27	1.15	0.764	2.17	1.45
	9	0.912	0.607	1.66	1.11	1.04	0.689	1.92	1.27	1.17	0.778	2.17	1.45
	10	0.930	0.619	1.66	1.11	1.06	0.703	1.92	1.27	1.19	0.794	2.17	1.45
	11	0.951	0.633	1.66	1.11	1.08	0.719	1.92	1.27	1.22	0.813	2.17	1.45
	12	0.974	0.648	1.68	1.11	1.11	0.737	1.93	1.28	1.25	0.833	2.19	1.46
	13	1.00	0.666	1.69	1.12	1.14	0.757	1.95	1.30	1.29	0.856	2.22	1.47
	14	1.03	0.685	1.70	1.13	1.17	0.779	1.96	1.31	1.33	0.882	2.24	1.49
	15	1.06	0.706	1.71	1.14	1.21	0.804	1.98	1.32	1.37	0.910	2.26	1.50
	16	1.10	0.730	1.73	1.15	1.25	0.831	2.00	1.33	1.41	0.941	2.28	1.52
	17	1.14	0.755	1.74	1.16	1.29	0.861	2.02	1.34	1.47	0.976	2.31	1.53
	18	1.18	0.784	1.76	1.17	1.34	0.894	2.04	1.35	1.52	1.01	2.33	1.55
	19	1.22	0.815	1.77	1.18	1.40	0.931	2.05	1.37	1.59	1.06	2.35	1.57
	20	1.28	0.849	1.78	1.19	1.46	0.970	2.07	1.38	1.65	1.10	2.38	1.58
	22	1.39	0.928	1.81	1.21	1.60	1.06	2.11	1.41	1.81	1.21	2.43	1.62
	24	1.54	1.02	1.84	1.23	1.76	1.17	2.15	1.43	2.00	1.33	2.48	1.65
	26	1.71	1.14	1.87	1.25	1.96	1.31	2.19	1.46	2.23	1.49	2.54	1.69
	28	1.91	1.27	1.91	1.27	2.20	1.47	2.24	1.49	2.51	1.67	2.60	1.73
	30	2.16	1.44	1.94	1.29	2.50	1.66	2.28	1.52	2.86	1.90	2.66	1.77
	32	2.46	1.64	1.97	1.31	2.84	1.89	2.33	1.55	3.25	2.16	2.72	1.81
	34	2.78	1.85	2.01	1.34	3.21	2.14	2.38	1.58	3.67	2.44	2.79	1.86
	36	3.12	2.07	2.05	1.36	3.60	2.40	2.43	1.62	4.11	2.74	2.86	1.90
	38	3.47	2.31	2.09	1.39	4.01	2.67	2.48	1.65	4.58	3.05	2.93	1.95
	40	3.85	2.56	2.13	1.41	4.44	2.96	2.54	1.69	5.08	3.38	3.01	2.00

Other Constants and Properties

	136 ASD	136 LRFD	120 ASD	120 LRFD	106 ASD	106 LRFD
$b_y\times10^3$, (kip-ft)$^{-1}$	3.64	2.42	4.17	2.78	4.74	3.16
$t_y\times10^3$, (kips)$^{-1}$	0.837	0.557	0.949	0.631	1.07	0.712
$t_r\times10^3$, (kips)$^{-1}$	1.03	0.685	1.17	0.777	1.31	0.877
r_x/r_y	1.77		1.76		1.76	
r_y, in.	3.16		3.13		3.11	

W12

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W12×											
		96				87				79			
Design		$p\times10^3$ (kips)$^{-1}$		$b_x\times10^3$ (kip-ft)$^{-1}$		$p\times10^3$ (kips)$^{-1}$		$b_x\times10^3$ (kip-ft)$^{-1}$		$p\times10^3$ (kips)$^{-1}$		$b_x\times10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	1.18	0.788	2.42	1.61	1.30	0.868	2.70	1.80	1.44	0.958	2.99	1.99
	6	1.23	0.820	2.42	1.61	1.36	0.904	2.70	1.80	1.50	0.998	2.99	1.99
	7	1.25	0.832	2.42	1.61	1.38	0.917	2.70	1.80	1.52	1.01	2.99	1.99
	8	1.27	0.846	2.42	1.61	1.40	0.932	2.70	1.80	1.55	1.03	2.99	1.99
	9	1.30	0.862	2.42	1.61	1.43	0.950	2.70	1.80	1.58	1.05	2.99	1.99
	10	1.32	0.880	2.42	1.61	1.46	0.971	2.70	1.80	1.61	1.07	2.99	1.99
	11	1.35	0.901	2.43	1.61	1.49	0.994	2.70	1.80	1.65	1.10	3.00	2.00
	12	1.39	0.924	2.45	1.63	1.53	1.02	2.74	1.82	1.69	1.13	3.04	2.02
	13	1.43	0.949	2.48	1.65	1.58	1.05	2.77	1.84	1.74	1.16	3.08	2.05
	14	1.47	0.978	2.50	1.67	1.62	1.08	2.80	1.86	1.80	1.20	3.12	2.08
	15	1.52	1.01	2.53	1.68	1.68	1.12	2.84	1.89	1.86	1.24	3.16	2.11
	16	1.57	1.05	2.56	1.70	1.74	1.16	2.87	1.91	1.92	1.28	3.21	2.13
	17	1.63	1.08	2.59	1.72	1.80	1.20	2.91	1.93	2.00	1.33	3.25	2.16
	18	1.69	1.13	2.62	1.74	1.87	1.25	2.94	1.96	2.08	1.38	3.30	2.19
	19	1.76	1.17	2.65	1.76	1.95	1.30	2.98	1.98	2.17	1.44	3.34	2.22
	20	1.84	1.22	2.68	1.78	2.04	1.36	3.02	2.01	2.26	1.51	3.39	2.26
	22	2.02	1.34	2.74	1.83	2.24	1.49	3.10	2.06	2.49	1.66	3.49	2.32
	24	2.24	1.49	2.81	1.87	2.48	1.65	3.19	2.12	2.76	1.84	3.60	2.40
	26	2.50	1.66	2.88	1.92	2.78	1.85	3.28	2.18	3.09	2.06	3.71	2.47
	28	2.81	1.87	2.95	1.97	3.13	2.08	3.37	2.24	3.50	2.33	3.84	2.55
	30	3.20	2.13	3.03	2.02	3.57	2.38	3.47	2.31	4.00	2.66	3.96	2.64
	32	3.64	2.42	3.11	2.07	4.07	2.71	3.58	2.38	4.55	3.02	4.10	2.73
	34	4.11	2.74	3.20	2.13	4.59	3.05	3.69	2.46	5.13	3.41	4.25	2.83
	36	4.61	3.07	3.29	2.19	5.15	3.42	3.81	2.54	5.75	3.83	4.41	2.93
	38	5.14	3.42	3.39	2.26	5.73	3.81	3.94	2.62	6.41	4.26	4.58	3.05
	40	5.69	3.79	3.49	2.32	6.35	4.23	4.08	2.72	7.10	4.73	4.78	3.18

Other Constants and Properties

	96 ASD	96 LRFD	87 ASD	87 LRFD	79 ASD	79 LRFD
$b_y\times10^3$, (kip-ft)$^{-1}$	5.28	3.51	5.90	3.92	6.56	4.37
$t_y\times10^3$, (kips)$^{-1}$	1.18	0.788	1.30	0.868	1.44	0.958
$t_r\times10^3$, (kips)$^{-1}$	1.45	0.970	1.60	1.07	1.77	1.18
r_x/r_y	1.76		1.75		1.75	
r_y, in.	3.09		3.07		3.05	

W12

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W12×											
		72				65^f				58			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	1.58	1.05	3.30	2.19	1.75	1.16	3.75	2.50	1.96	1.31	4.12	2.74
	6	1.65	1.10	3.30	2.19	1.82	1.21	3.75	2.50	2.09	1.39	4.12	2.74
	7	1.67	1.11	3.30	2.19	1.85	1.23	3.75	2.50	2.13	1.42	4.12	2.74
	8	1.70	1.13	3.30	2.19	1.88	1.25	3.75	2.50	2.19	1.45	4.12	2.74
	9	1.74	1.16	3.30	2.19	1.92	1.28	3.75	2.50	2.25	1.50	4.13	2.75
	10	1.77	1.18	3.30	2.19	1.96	1.31	3.75	2.50	2.32	1.54	4.21	2.80
	11	1.82	1.21	3.31	2.20	2.01	1.34	3.75	2.50	2.41	1.60	4.28	2.85
	12	1.87	1.24	3.36	2.23	2.06	1.37	3.75	2.50	2.50	1.66	4.36	2.90
	13	1.92	1.28	3.40	2.27	2.13	1.41	3.81	2.54	2.61	1.73	4.45	2.96
	14	1.98	1.32	3.45	2.30	2.19	1.46	3.87	2.58	2.73	1.81	4.53	3.02
	15	2.05	1.36	3.50	2.33	2.27	1.51	3.93	2.62	2.86	1.90	4.62	3.07
	16	2.12	1.41	3.56	2.37	2.35	1.56	4.00	2.66	3.01	2.01	4.71	3.14
	17	2.20	1.46	3.61	2.40	2.44	1.62	4.06	2.70	3.18	2.12	4.81	3.20
	18	2.29	1.52	3.67	2.44	2.54	1.69	4.13	2.75	3.38	2.25	4.91	3.27
	19	2.39	1.59	3.72	2.48	2.65	1.77	4.20	2.80	3.59	2.39	5.01	3.34
	20	2.50	1.66	3.78	2.52	2.77	1.85	4.27	2.84	3.83	2.55	5.12	3.41
	22	2.75	1.83	3.91	2.60	3.06	2.03	4.43	2.95	4.41	2.94	5.36	3.56
	24	3.05	2.03	4.04	2.69	3.40	2.26	4.59	3.06	5.15	3.43	5.61	3.74
	26	3.42	2.28	4.18	2.78	3.82	2.54	4.77	3.17	6.05	4.02	5.90	3.92
	28	3.87	2.57	4.33	2.88	4.32	2.88	4.96	3.30	7.01	4.67	6.21	4.13
	30	4.42	2.94	4.49	2.99	4.95	3.29	5.17	3.44	8.05	5.36	6.57	4.37
	32	5.03	3.35	4.67	3.10	5.63	3.75	5.39	3.59	9.16	6.09	7.12	4.74
	34	5.68	3.78	4.86	3.23	6.36	4.23	5.64	3.75	10.3	6.88	7.66	5.10
	36	6.37	4.24	5.06	3.37	7.13	4.74	5.97	3.98	11.6	7.71	8.21	5.46
	38	7.09	4.72	5.32	3.54	7.94	5.28	6.39	4.25	12.9	8.59	8.75	5.82
	40	7.86	5.23	5.66	3.76	8.80	5.85	6.81	4.53	14.3	9.52	9.29	6.18

Other Constants and Properties

	72 ASD	72 LRFD	65^f ASD	65^f LRFD	58 ASD	58 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	7.24	4.82	8.31	5.53	11.0	7.29
$t_y \times 10^3$, (kips)$^{-1}$	1.58	1.05	1.75	1.16	1.96	1.31
$t_r \times 10^3$, (kips)$^{-1}$	1.94	1.30	2.15	1.43	2.41	1.61
r_x/r_y	1.75		1.75		2.10	
r_y, in.	3.04		3.02		2.51	

[f] Shape does not meet compact limit for flexure for F_y = 50 ksi.

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W12

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W12×											
		53				50				45			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	2.14	1.42	4.57	3.04	2.29	1.52	4.96	3.30	2.55	1.70	5.55	3.69
	6	2.28	1.52	4.57	3.04	2.52	1.68	4.96	3.30	2.82	1.87	5.55	3.69
	7	2.33	1.55	4.57	3.04	2.62	1.74	4.96	3.30	2.92	1.94	5.56	3.70
	8	2.39	1.59	4.57	3.04	2.73	1.81	5.08	3.38	3.04	2.03	5.70	3.79
	9	2.46	1.64	4.59	3.06	2.86	1.90	5.19	3.46	3.19	2.12	5.84	3.89
	10	2.54	1.69	4.68	3.12	3.01	2.00	5.32	3.54	3.36	2.24	6.00	3.99
	11	2.63	1.75	4.77	3.18	3.19	2.12	5.45	3.62	3.56	2.37	6.15	4.09
	12	2.74	1.82	4.87	3.24	3.39	2.26	5.58	3.72	3.80	2.53	6.32	4.21
	13	2.86	1.90	4.97	3.31	3.64	2.42	5.73	3.81	4.07	2.71	6.50	4.32
	14	2.99	1.99	5.07	3.38	3.91	2.60	5.88	3.91	4.39	2.92	6.69	4.45
	15	3.15	2.09	5.18	3.45	4.24	2.82	6.04	4.02	4.75	3.16	6.88	4.58
	16	3.32	2.21	5.29	3.52	4.61	3.07	6.20	4.13	5.18	3.45	7.09	4.72
	17	3.51	2.34	5.41	3.60	5.05	3.36	6.38	4.25	5.68	3.78	7.32	4.87
	18	3.73	2.48	5.53	3.68	5.56	3.70	6.57	4.37	6.25	4.16	7.56	5.03
	19	3.97	2.64	5.66	3.77	6.17	4.10	6.77	4.50	6.94	4.62	7.81	5.20
	20	4.25	2.83	5.80	3.86	6.83	4.55	6.98	4.64	7.69	5.12	8.08	5.38
	22	4.90	3.26	6.09	4.05	8.27	5.50	7.45	4.95	9.31	6.19	8.69	5.78
	24	5.75	3.83	6.41	4.26	9.84	6.55	8.01	5.33	11.1	7.37	9.66	6.43
	26	6.75	4.49	6.77	4.50	11.5	7.68	8.84	5.88	13.0	8.65	10.7	7.11
	28	7.83	5.21	7.16	4.77	13.4	8.91	9.67	6.44	15.1	10.0	11.7	7.80
	30	8.99	5.98	7.81	5.20	15.4	10.2	10.5	6.99	17.3	11.5	12.8	8.48
	32	10.2	6.80	8.48	5.64	17.5	11.6	11.3	7.53	19.7	13.1	13.8	9.16
	34	11.5	7.68	9.15	6.09								
	36	12.9	8.61	9.81	6.53								
	38	14.4	9.59	10.5	6.97								
	40	16.0	10.6	11.1	7.41								
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		12.2		8.15		16.7		11.1		18.8		12.5	
$t_y \times 10^3$, (kips)$^{-1}$		2.14		1.42		2.29		1.52		2.55		1.70	
$t_r \times 10^3$, (kips)$^{-1}$		2.63		1.75		2.81		1.87		3.13		2.09	
r_x/r_y		2.11				2.64				2.64			
r_y, in.		2.48				1.96				1.95			

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

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W12

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W12×											
		40				35^c				30^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	2.85	1.90	6.25	4.16	3.25	2.16	6.96	4.63	3.93	2.62	8.27	5.50
	6	3.16	2.10	6.25	4.16	3.80	2.53	7.09	4.72	4.55	3.03	8.46	5.63
	7	3.27	2.18	6.27	4.17	4.03	2.68	7.34	4.89	4.79	3.19	8.79	5.85
	8	3.41	2.27	6.44	4.29	4.31	2.87	7.61	5.07	5.09	3.39	9.14	6.08
	9	3.58	2.38	6.62	4.40	4.65	3.09	7.90	5.26	5.50	3.66	9.53	6.34
	10	3.78	2.51	6.80	4.53	5.05	3.36	8.22	5.47	5.99	3.99	9.94	6.62
	11	4.00	2.66	7.00	4.66	5.55	3.69	8.56	5.69	6.60	4.39	10.4	6.92
	12	4.27	2.84	7.21	4.79	6.15	4.09	8.93	5.94	7.32	4.87	10.9	7.25
	13	4.58	3.05	7.43	4.94	6.87	4.57	9.33	6.21	8.21	5.46	11.5	7.62
	14	4.94	3.29	7.66	5.10	7.74	5.15	9.77	6.50	9.28	6.18	12.1	8.02
	15	5.36	3.56	7.91	5.26	8.82	5.87	10.3	6.82	10.6	7.06	12.7	8.48
	16	5.84	3.89	8.18	5.44	10.0	6.68	10.8	7.18	12.1	8.04	13.7	9.13
	17	6.41	4.26	8.46	5.63	11.3	7.54	11.5	7.66	13.6	9.07	15.0	10.0
	18	7.07	4.70	8.77	5.83	12.7	8.45	12.5	8.30	15.3	10.2	16.4	10.9
	19	7.85	5.23	9.10	6.05	14.2	9.42	13.4	8.94	17.0	11.3	17.7	11.8
	20	8.70	5.79	9.45	6.29	15.7	10.4	14.4	9.59	18.9	12.6	19.0	12.7
	22	10.5	7.01	10.5	6.96	19.0	12.6	16.3	10.9	22.8	15.2	21.7	14.5
	24	12.5	8.34	11.8	7.83	22.6	15.0	18.3	12.2	27.2	18.1	24.4	16.3
	26	14.7	9.79	13.1	8.69								
	28	17.1	11.3	14.4	9.56								
	30	19.6	13.0	15.7	10.4								
	32	22.3	14.8	16.9	11.3								

Other Constants and Properties

	40 ASD	40 LRFD	35^c ASD	35^c LRFD	30^c ASD	30^c LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	21.2	14.1	31.0	20.6	37.3	24.8
$t_y \times 10^3$, (kips)$^{-1}$	2.85	1.90	3.24	2.16	3.80	2.53
$t_r \times 10^3$, (kips)$^{-1}$	3.51	2.34	3.98	2.66	4.67	3.11
r_x / r_y	2.64		3.41		3.43	
r_y, in.	1.94		1.54		1.52	

[c] Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c / r_y equal to or greater than 200.

W12

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W12×											
		26^c				22^c				19^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	4.64	3.09	9.58	6.37	5.40	3.59	12.2	8.09	6.50	4.33	14.4	9.60
	1	4.66	3.10	9.58	6.37	5.47	3.64	12.2	8.09	6.59	4.38	14.4	9.60
	2	4.72	3.14	9.58	6.37	5.67	3.78	12.2	8.09	6.85	4.56	14.4	9.60
	3	4.82	3.21	9.58	6.37	6.04	4.02	12.2	8.09	7.31	4.87	14.5	9.66
	4	4.96	3.30	9.58	6.37	6.59	4.38	13.0	8.65	8.02	5.33	15.6	10.4
	5	5.15	3.42	9.58	6.37	7.43	4.95	14.0	9.28	9.02	6.00	16.9	11.2
	6	5.38	3.58	9.83	6.54	8.73	5.81	15.1	10.0	10.5	6.99	18.4	12.2
	7	5.68	3.78	10.2	6.81	10.6	7.03	16.4	10.9	12.9	8.56	20.2	13.4
	8	6.04	4.02	10.7	7.11	13.2	8.75	17.9	11.9	16.3	10.8	22.3	14.9
	9	6.47	4.31	11.2	7.43	16.7	11.1	19.8	13.1	20.6	13.7	25.7	17.1
	10	7.00	4.66	11.7	7.79	20.6	13.7	23.0	15.3	25.5	16.9	30.4	20.2
	11	7.63	5.08	12.3	8.17	24.9	16.6	26.5	17.6	30.8	20.5	35.2	23.4
	12	8.49	5.65	12.9	8.60	29.6	19.7	30.0	20.0	36.7	24.4	40.1	26.7
	13	9.53	6.34	13.6	9.08	34.7	23.1	33.5	22.3	43.0	28.6	45.1	30.0
	14	10.8	7.18	14.4	9.61	40.3	26.8	37.1	24.7				
	15	12.4	8.22	15.4	10.3								
	16	14.1	9.36	17.1	11.4								
	17	15.9	10.6	18.8	12.5								
	18	17.8	11.8	20.6	13.7								
	19	19.8	13.2	22.3	14.9								
	20	22.0	14.6	24.1	16.0								
	21	24.2	16.1	25.9	17.2								
	22	26.6	17.7	27.7	18.4								
	23	29.1	19.3	29.5	19.6								
	24	31.6	21.0	31.3	20.8								
	25	34.3	22.8	33.1	22.0								
Other Constants and Properties													
$b_y \times 10^3$, (kip-ft)$^{-1}$		43.6		29.0		97.3		64.8		120		79.5	
$t_y \times 10^3$, (kips)$^{-1}$		4.37		2.90		5.15		3.43		6.00		3.99	
$t_r \times 10^3$, (kips)$^{-1}$		5.36		3.58		6.33		4.22		7.37		4.91	
r_x / r_y		3.42				5.79				5.86			
r_y, in.		1.51				0.848				0.822			

^{c} Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c / r_y equal to or greater than 200.

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W12

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W12×							
		16[c]				14[c,v]			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	7.95	5.29	17.7	11.8	9.35	6.22	20.5	13.6
	1	8.06	5.37	17.7	11.8	9.49	6.31	20.5	13.6
	2	8.42	5.60	17.7	11.8	9.93	6.61	20.5	13.6
	3	9.05	6.02	18.1	12.0	10.7	7.13	21.0	14.0
	4	10.0	6.67	19.6	13.1	11.9	7.93	22.9	15.2
	5	11.4	7.59	21.4	14.3	13.7	9.08	25.1	16.7
	6	13.4	8.91	23.6	15.7	16.1	10.7	27.8	18.5
	7	16.8	11.2	26.3	17.5	19.9	13.3	31.2	20.7
	8	21.8	14.5	29.6	19.7	26.0	17.3	36.4	24.2
	9	27.6	18.3	36.1	24.0	32.9	21.9	44.6	29.7
	10	34.0	22.6	42.9	28.5	40.6	27.0	53.3	35.5
	11	41.2	27.4	50.0	33.3	49.1	32.7	62.4	41.5
	12	49.0	32.6	57.2	38.1	58.5	38.9	71.8	47.8

Other Constants and Properties

	16 ASD	16 LRFD	14 ASD	14 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	158	105	188	125
$t_y \times 10^3$, (kips)$^{-1}$	7.09	4.72	8.03	5.34
$t_r \times 10^3$, (kips)$^{-1}$	8.71	5.81	9.86	6.57
r_x/r_y	6.04		6.14	
r_y, in.	0.773		0.753	

[c] Shape is slender for compression for F_y = 50 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(1) with F_y = 50 ksi; therefore, ϕ_v = 0.90 and Ω_v = 1.67.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W10

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W10×											
		112				100				88			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	1.02	0.675	2.42	1.61	1.14	0.758	2.74	1.82	1.28	0.855	3.15	2.10
	6	1.07	0.712	2.42	1.61	1.20	0.800	2.74	1.82	1.36	0.903	3.15	2.10
	7	1.09	0.726	2.42	1.61	1.23	0.816	2.74	1.82	1.38	0.921	3.15	2.10
	8	1.12	0.742	2.42	1.61	1.25	0.835	2.74	1.82	1.42	0.942	3.15	2.10
	9	1.14	0.761	2.42	1.61	1.29	0.856	2.74	1.82	1.45	0.967	3.15	2.10
	10	1.18	0.782	2.43	1.62	1.32	0.881	2.75	1.83	1.50	0.995	3.17	2.11
	11	1.21	0.807	2.45	1.63	1.37	0.909	2.78	1.85	1.54	1.03	3.20	2.13
	12	1.25	0.834	2.47	1.64	1.41	0.941	2.80	1.86	1.60	1.06	3.23	2.15
	13	1.30	0.865	2.49	1.66	1.47	0.977	2.82	1.88	1.66	1.11	3.27	2.17
	14	1.35	0.900	2.51	1.67	1.53	1.02	2.85	1.90	1.73	1.15	3.30	2.19
	15	1.41	0.939	2.53	1.68	1.60	1.06	2.87	1.91	1.81	1.20	3.33	2.22
	16	1.48	0.983	2.55	1.69	1.67	1.11	2.90	1.93	1.90	1.26	3.36	2.24
	17	1.55	1.03	2.56	1.71	1.76	1.17	2.92	1.94	1.99	1.33	3.40	2.26
	18	1.63	1.09	2.59	1.72	1.85	1.23	2.95	1.96	2.10	1.40	3.43	2.28
	19	1.72	1.15	2.61	1.73	1.96	1.30	2.98	1.98	2.23	1.48	3.47	2.31
	20	1.82	1.21	2.63	1.75	2.08	1.38	3.00	2.00	2.36	1.57	3.50	2.33
	22	2.06	1.37	2.67	1.78	2.36	1.57	3.06	2.03	2.68	1.79	3.58	2.38
	24	2.36	1.57	2.71	1.80	2.70	1.80	3.11	2.07	3.09	2.05	3.65	2.43
	26	2.74	1.82	2.76	1.83	3.15	2.09	3.17	2.11	3.60	2.40	3.73	2.48
	28	3.18	2.11	2.80	1.87	3.65	2.43	3.23	2.15	4.18	2.78	3.82	2.54
	30	3.65	2.43	2.85	1.90	4.19	2.79	3.30	2.19	4.79	3.19	3.90	2.60
	32	4.15	2.76	2.90	1.93	4.77	3.17	3.36	2.24	5.46	3.63	4.00	2.66
	34	4.69	3.12	2.95	1.97	5.38	3.58	3.43	2.28	6.16	4.10	4.09	2.72
	36	5.25	3.50	3.01	2.00	6.03	4.01	3.50	2.33	6.90	4.59	4.19	2.79
	38	5.85	3.90	3.06	2.04	6.72	4.47	3.58	2.38	7.69	5.12	4.30	2.86
	40	6.49	4.32	3.12	2.08	7.45	4.96	3.66	2.43	8.52	5.67	4.41	2.94

Other Constants and Properties

	112 ASD	112 LRFD	100 ASD	100 LRFD	88 ASD	88 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	5.15	3.43	5.84	3.89	6.71	4.46
$t_y \times 10^3$, (kips)$^{-1}$	1.02	0.675	1.14	0.758	1.28	0.855
$t_r \times 10^3$, (kips)$^{-1}$	1.25	0.831	1.40	0.933	1.58	1.05
r_x/r_y	1.74		1.74		1.73	
r_y, in.	2.68		2.65		2.63	

W10

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W10×											
		77				68				60			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	1.47	0.979	3.65	2.43	1.68	1.12	4.18	2.78	1.89	1.26	4.78	3.18
	6	1.56	1.04	3.65	2.43	1.78	1.18	4.18	2.78	2.00	1.33	4.78	3.18
	7	1.59	1.06	3.65	2.43	1.81	1.21	4.18	2.78	2.04	1.36	4.78	3.18
	8	1.63	1.08	3.65	2.43	1.86	1.23	4.18	2.78	2.09	1.39	4.78	3.18
	9	1.67	1.11	3.65	2.43	1.91	1.27	4.18	2.78	2.15	1.43	4.78	3.18
	10	1.72	1.14	3.68	2.45	1.96	1.31	4.22	2.81	2.21	1.47	4.84	3.22
	11	1.78	1.18	3.72	2.48	2.03	1.35	4.27	2.84	2.29	1.52	4.90	3.26
	12	1.84	1.23	3.76	2.50	2.10	1.40	4.32	2.88	2.37	1.58	4.97	3.31
	13	1.91	1.27	3.80	2.53	2.19	1.46	4.38	2.91	2.47	1.64	5.04	3.36
	14	2.00	1.33	3.85	2.56	2.28	1.52	4.44	2.95	2.58	1.72	5.12	3.41
	15	2.09	1.39	3.89	2.59	2.39	1.59	4.49	2.99	2.70	1.80	5.19	3.46
	16	2.19	1.46	3.94	2.62	2.51	1.67	4.55	3.03	2.84	1.89	5.27	3.51
	17	2.31	1.54	3.98	2.65	2.64	1.76	4.61	3.07	2.99	1.99	5.35	3.56
	18	2.44	1.62	4.03	2.68	2.79	1.86	4.67	3.11	3.16	2.10	5.43	3.62
	19	2.58	1.72	4.08	2.71	2.96	1.97	4.74	3.15	3.36	2.23	5.52	3.67
	20	2.74	1.83	4.13	2.74	3.14	2.09	4.80	3.20	3.57	2.38	5.61	3.73
	22	3.13	2.08	4.23	2.81	3.59	2.39	4.94	3.29	4.08	2.72	5.79	3.85
	24	3.61	2.40	4.33	2.88	4.15	2.76	5.08	3.38	4.73	3.14	5.99	3.99
	26	4.22	2.81	4.45	2.96	4.85	3.23	5.24	3.49	5.54	3.69	6.20	4.13
	28	4.89	3.26	4.56	3.04	5.63	3.74	5.40	3.59	6.42	4.27	6.43	4.28
	30	5.62	3.74	4.69	3.12	6.46	4.30	5.57	3.71	7.38	4.91	6.67	4.44
	32	6.39	4.25	4.82	3.21	7.35	4.89	5.76	3.83	8.39	5.58	6.94	4.61
	34	7.22	4.80	4.96	3.30	8.30	5.52	5.96	3.96	9.47	6.30	7.22	4.80
	36	8.09	5.38	5.11	3.40	9.30	6.19	6.17	4.10	10.6	7.07	7.53	5.01
	38	9.02	6.00	5.26	3.50	10.4	6.90	6.40	4.26	11.8	7.87	7.96	5.30
	40	9.99	6.65	5.43	3.61	11.5	7.64	6.64	4.42	13.1	8.72	8.43	5.61
Other Constants and Properties													
$b_y \times 10^3$, $(\text{kip-ft})^{-1}$		7.76		5.16		8.88		5.91		10.2		6.77	
$t_y \times 10^3$, $(\text{kips})^{-1}$		1.47		0.979		1.68		1.12		1.89		1.26	
$t_r \times 10^3$, $(\text{kips})^{-1}$		1.81		1.20		2.06		1.37		2.32		1.55	
r_x/r_y		1.73				1.71				1.71			
r_y, in.		2.60				2.59				2.57			

W10

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W10×											
		54				49				45			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	2.11	1.41	5.35	3.56	2.32	1.54	5.90	3.92	2.51	1.67	6.49	4.32
	6	2.24	1.49	5.35	3.56	2.46	1.64	5.90	3.92	2.76	1.84	6.49	4.32
	7	2.29	1.52	5.35	3.56	2.51	1.67	5.90	3.92	2.85	1.90	6.49	4.32
	8	2.34	1.56	5.35	3.56	2.57	1.71	5.90	3.92	2.97	1.97	6.60	4.39
	9	2.41	1.60	5.35	3.56	2.65	1.76	5.90	3.93	3.10	2.06	6.73	4.48
	10	2.48	1.65	5.43	3.61	2.73	1.82	6.00	3.99	3.26	2.17	6.87	4.57
	11	2.57	1.71	5.51	3.67	2.83	1.88	6.10	4.06	3.44	2.29	7.00	4.66
	12	2.66	1.77	5.60	3.72	2.93	1.95	6.20	4.13	3.65	2.43	7.15	4.76
	13	2.77	1.85	5.69	3.78	3.06	2.03	6.31	4.20	3.90	2.60	7.30	4.86
	14	2.90	1.93	5.78	3.85	3.19	2.12	6.42	4.27	4.19	2.78	7.46	4.96
	15	3.03	2.02	5.88	3.91	3.35	2.23	6.54	4.35	4.51	3.00	7.63	5.07
	16	3.19	2.12	5.97	3.97	3.52	2.34	6.66	4.43	4.89	3.26	7.80	5.19
	17	3.36	2.24	6.08	4.04	3.72	2.47	6.78	4.51	5.33	3.55	7.98	5.31
	18	3.56	2.37	6.18	4.11	3.94	2.62	6.91	4.60	5.84	3.89	8.17	5.44
	19	3.78	2.51	6.29	4.19	4.18	2.78	7.04	4.69	6.44	4.28	8.37	5.57
	20	4.02	2.67	6.40	4.26	4.46	2.96	7.18	4.78	7.13	4.75	8.58	5.71
	22	4.60	3.06	6.64	4.42	5.11	3.40	7.48	4.98	8.63	5.74	9.03	6.01
	24	5.33	3.55	6.90	4.59	5.94	3.95	7.80	5.19	10.3	6.83	9.53	6.34
	26	6.25	4.16	7.18	4.78	6.97	4.64	8.15	5.42	12.1	8.02	10.1	6.71
	28	7.25	4.83	7.48	4.98	8.08	5.38	8.53	5.68	14.0	9.30	10.9	7.22
	30	8.33	5.54	7.81	5.20	9.28	6.17	8.95	5.96	16.0	10.7	11.7	7.82
	32	9.47	6.30	8.17	5.43	10.6	7.03	9.47	6.30	18.3	12.1	12.6	8.41
	34	10.7	7.12	8.60	5.72	11.9	7.93	10.2	6.77				
	36	12.0	7.98	9.19	6.11	13.4	8.89	10.9	7.24				
	38	13.4	8.89	9.77	6.50	14.9	9.91	11.6	7.71				
	40	14.8	9.85	10.4	6.89	16.5	11.0	12.3	8.18				

Other Constants and Properties

	54 ASD	54 LRFD	49 ASD	49 LRFD	45 ASD	45 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	11.4	7.57	12.6	8.38	17.6	11.7
$t_y \times 10^3$, (kips)$^{-1}$	2.11	1.41	2.32	1.54	2.51	1.67
$t_r \times 10^3$, (kips)$^{-1}$	2.60	1.73	2.85	1.90	3.08	2.06
r_x/r_y	1.71		1.71		2.15	
r_y, in.	2.56		2.54		2.01	

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W10

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W10×											
		39				33				30			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	2.90	1.93	7.61	5.06	3.44	2.29	9.18	6.11	3.78	2.51	9.73	6.48
	6	3.20	2.13	7.61	5.06	3.80	2.53	9.18	6.11	4.62	3.08	10.1	6.74
	7	3.31	2.20	7.61	5.07	3.95	2.62	9.22	6.13	4.97	3.31	10.5	6.99
	8	3.45	2.29	7.78	5.18	4.11	2.74	9.45	6.29	5.41	3.60	10.9	7.25
	9	3.61	2.40	7.96	5.29	4.31	2.87	9.70	6.45	5.95	3.96	11.3	7.53
	10	3.80	2.53	8.14	5.41	4.55	3.03	9.96	6.62	6.62	4.41	11.8	7.84
	11	4.02	2.67	8.33	5.54	4.83	3.21	10.2	6.81	7.45	4.96	12.3	8.17
	12	4.28	2.84	8.53	5.67	5.15	3.42	10.5	7.00	8.47	5.64	12.8	8.54
	13	4.57	3.04	8.74	5.81	5.52	3.67	10.8	7.20	9.76	6.49	13.4	8.93
	14	4.92	3.27	8.96	5.96	5.95	3.96	11.2	7.42	11.3	7.53	14.1	9.37
	15	5.31	3.54	9.19	6.12	6.45	4.29	11.5	7.65	13.0	8.64	14.8	9.85
	16	5.78	3.84	9.44	6.28	7.04	4.68	11.9	7.89	14.8	9.83	15.6	10.4
	17	6.31	4.20	9.70	6.45	7.72	5.14	12.3	8.15	16.7	11.1	16.8	11.2
	18	6.93	4.61	9.97	6.63	8.51	5.67	12.7	8.43	18.7	12.4	18.1	12.1
	19	7.67	5.10	10.3	6.82	9.46	6.30	13.1	8.73	20.8	13.9	19.4	12.9
	20	8.50	5.66	10.6	7.03	10.5	6.98	13.6	9.05	23.1	15.4	20.7	13.8
	22	10.3	6.84	11.2	7.47	12.7	8.44	14.8	9.82	27.9	18.6	23.2	15.4
	24	12.2	8.14	12.0	7.98	15.1	10.0	16.5	11.0				
	26	14.4	9.56	13.2	8.77	17.7	11.8	18.3	12.2				
	28	16.7	11.1	14.4	9.58	20.6	13.7	20.1	13.4				
	30	19.1	12.7	15.6	10.4	23.6	15.7	21.9	14.5				
	32	21.8	14.5	16.8	11.2	26.8	17.9	23.6	15.7				

Other Constants and Properties

	39 ASD	39 LRFD	33 ASD	33 LRFD	30 ASD	30 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	20.7	13.8	25.4	16.9	40.3	26.8
$t_y \times 10^3$, (kips)$^{-1}$	2.90	1.93	3.44	2.29	3.78	2.51
$t_r \times 10^3$, (kips)$^{-1}$	3.57	2.38	4.23	2.82	4.64	3.09
r_x / r_y	2.16		2.16		3.20	
r_y, in.	1.98		1.94		1.37	

Note: Heavy line indicates L_c / r_y equal to or greater than 200.

W10

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W10×											
		26				22^c				19			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	4.39	2.92	11.4	7.57	5.18	3.45	13.7	9.12	5.94	3.95	16.5	11.0
	1	4.41	2.94	11.4	7.57	5.21	3.46	13.7	9.12	6.03	4.01	16.5	11.0
	2	4.49	2.99	11.4	7.57	5.29	3.52	13.7	9.12	6.28	4.18	16.5	11.0
	3	4.62	3.07	11.4	7.57	5.43	3.61	13.7	9.12	6.73	4.48	16.5	11.0
	4	4.81	3.20	11.4	7.57	5.66	3.77	13.7	9.12	7.41	4.93	17.4	11.6
	5	5.06	3.37	11.5	7.63	5.97	3.97	13.9	9.23	8.39	5.58	18.6	12.4
	6	5.39	3.58	11.9	7.93	6.38	4.24	14.5	9.64	9.76	6.49	19.9	13.2
	7	5.80	3.86	12.4	8.25	6.89	4.58	15.1	10.1	11.7	7.77	21.4	14.3
	8	6.32	4.20	12.9	8.59	7.53	5.01	15.9	10.6	14.4	9.55	23.2	15.4
	9	6.96	4.63	13.5	8.97	8.33	5.55	16.7	11.1	18.1	12.0	25.3	16.8
	10	7.76	5.16	14.1	9.38	9.33	6.21	17.6	11.7	22.3	14.8	28.2	18.8
	11	8.74	5.81	14.8	9.84	10.6	7.04	18.5	12.3	27.0	18.0	32.3	21.5
	12	9.96	6.63	15.5	10.3	12.1	8.07	19.6	13.1	32.1	21.4	36.4	24.2
	13	11.5	7.65	16.4	10.9	14.1	9.38	20.9	13.9	37.7	25.1	40.5	26.9
	14	13.3	8.88	17.3	11.5	16.4	10.9	22.5	15.0	43.7	29.1	44.6	29.7
	15	15.3	10.2	18.4	12.2	18.8	12.5	25.0	16.6				
	16	17.4	11.6	20.1	13.4	21.4	14.2	27.4	18.2				
	17	19.7	13.1	21.8	14.5	24.1	16.0	29.9	19.9				
	18	22.1	14.7	23.6	15.7	27.0	18.0	32.4	21.6				
	19	24.6	16.3	25.3	16.8	30.1	20.0	34.9	23.2				
	20	27.2	18.1	27.0	18.0	33.4	22.2	37.4	24.9				
	21	30.0	20.0	28.7	19.1	36.8	24.5	39.9	26.5				
	22	32.9	21.9	30.5	20.3	40.4	26.9	42.4	28.2				

Other Constants and Properties

	W10×26 ASD	W10×26 LRFD	W10×22^c ASD	W10×22^c LRFD	W10×19 ASD	W10×19 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	47.5	31.6	58.4	38.9	106	70.8
$t_y \times 10^3$, (kips)$^{-1}$	4.39	2.92	5.15	3.42	5.94	3.95
$t_r \times 10^3$, (kips)$^{-1}$	5.39	3.59	6.32	4.21	7.30	4.87
r_x / r_y	3.20		3.21		4.74	
r_y, in.	1.36		1.33		0.874	

^{c} Shape is slender for compression for F_y = 50 ksi.

Note: Heavy line indicates L_c / r_y equal to or greater than 200.

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W10

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape		W10×											
		17^c				15^c				$12^{c,f}$			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	6.75	4.49	19.1	12.7	7.75	5.15	22.3	14.8	10.3	6.85	28.5	19.0
	1	6.83	4.55	19.1	12.7	7.85	5.22	22.3	14.8	10.4	6.95	28.5	19.0
	2	7.10	4.72	19.1	12.7	8.18	5.44	22.3	14.8	10.9	7.25	28.5	19.0
	3	7.64	5.09	19.1	12.7	8.75	5.82	22.5	15.0	11.7	7.78	28.8	19.1
	4	8.47	5.64	20.4	13.6	9.79	6.51	24.2	16.1	12.9	8.60	31.1	20.7
	5	9.68	6.44	21.9	14.5	11.3	7.53	26.1	17.4	14.7	9.78	33.9	22.6
	6	11.4	7.57	23.6	15.7	13.5	8.98	28.4	18.9	17.5	11.6	37.3	24.8
	7	13.8	9.17	25.6	17.0	16.6	11.1	31.2	20.7	21.8	14.5	41.3	27.5
	8	17.2	11.4	28.0	18.6	21.2	14.1	34.5	22.9	28.1	18.7	46.4	30.9
	9	21.8	14.5	30.9	20.6	26.8	17.8	39.6	26.4	35.6	23.7	56.5	37.6
	10	26.9	17.9	36.0	23.9	33.1	22.0	46.8	31.1	43.9	29.2	67.2	44.7
	11	32.5	21.6	41.4	27.5	40.1	26.7	54.0	35.9	53.1	35.4	78.3	52.1
	12	38.7	25.8	46.8	31.2	47.7	31.7	61.4	40.9	63.2	42.1	89.6	59.6
	13	45.4	30.2	52.3	34.8	56.0	37.2	68.8	45.8	74.2	49.4	101	67.3
	14	52.7	35.1	57.8	38.5								

Other Constants and Properties

$b_y \times 10^3$, $(\text{kip-ft})^{-1}$	127	84.7	155	103	207	138
$t_y \times 10^3$, $(\text{kips})^{-1}$	6.69	4.45	7.57	5.04	9.44	6.28
$t_r \times 10^3$, $(\text{kips})^{-1}$	8.22	5.48	9.30	6.20	11.6	7.73
r_x/r_y	4.79		4.88		4.97	
r_y, in.	0.845		0.810		0.785	

[c] Shape is slender for compression for F_y = 50 ksi.

[f] Shape does not meet compact limit for flexure for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W8

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W8×											
		67				58				48			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	1.70	1.13	5.08	3.38	1.95	1.30	5.96	3.96	2.37	1.58	7.27	4.84
	6	1.84	1.23	5.08	3.38	2.13	1.42	5.96	3.96	2.59	1.72	7.27	4.84
	7	1.90	1.27	5.08	3.38	2.20	1.46	5.96	3.96	2.67	1.78	7.27	4.84
	8	1.97	1.31	5.11	3.40	2.28	1.51	6.00	3.99	2.77	1.84	7.34	4.88
	9	2.05	1.36	5.16	3.43	2.37	1.58	6.07	4.04	2.88	1.92	7.44	4.95
	10	2.14	1.43	5.21	3.47	2.48	1.65	6.14	4.08	3.02	2.01	7.55	5.02
	11	2.25	1.50	5.27	3.50	2.61	1.73	6.21	4.13	3.18	2.12	7.65	5.09
	12	2.38	1.58	5.32	3.54	2.75	1.83	6.29	4.18	3.36	2.24	7.77	5.17
	13	2.52	1.68	5.38	3.58	2.92	1.95	6.36	4.23	3.57	2.38	7.88	5.24
	14	2.68	1.79	5.43	3.61	3.12	2.08	6.44	4.29	3.82	2.54	8.00	5.32
	15	2.87	1.91	5.49	3.65	3.34	2.22	6.52	4.34	4.10	2.73	8.12	5.41
	16	3.09	2.05	5.55	3.69	3.60	2.39	6.61	4.40	4.42	2.94	8.25	5.49
	17	3.34	2.22	5.61	3.73	3.89	2.59	6.69	4.45	4.79	3.18	8.38	5.58
	18	3.62	2.41	5.67	3.77	4.23	2.82	6.78	4.51	5.21	3.47	8.52	5.67
	19	3.95	2.63	5.74	3.82	4.62	3.08	6.87	4.57	5.70	3.79	8.66	5.76
	20	4.33	2.88	5.80	3.86	5.08	3.38	6.96	4.63	6.28	4.18	8.80	5.85
	22	5.24	3.48	5.93	3.95	6.15	4.09	7.15	4.76	7.60	5.06	9.10	6.06
	24	6.23	4.15	6.07	4.04	7.32	4.87	7.35	4.89	9.05	6.02	9.43	6.27
	26	7.31	4.87	6.22	4.14	8.59	5.71	7.57	5.03	10.6	7.06	9.77	6.50
	28	8.48	5.64	6.38	4.24	9.96	6.63	7.79	5.19	12.3	8.19	10.1	6.75
	30	9.74	6.48	6.54	4.35	11.4	7.61	8.03	5.35	14.1	9.40	10.6	7.02
	32	11.1	7.37	6.71	4.46	13.0	8.66	8.29	5.52	16.1	10.7	11.0	7.31
	34	12.5	8.32	6.89	4.58	14.7	9.77	8.56	5.70	18.2	12.1	11.5	7.63

Other Constants and Properties

	67		58		48	
$b_y \times 10^3$, (kip-ft)$^{-1}$	10.9	7.25	12.8	8.50	15.6	10.4
$t_y \times 10^3$, (kips)$^{-1}$	1.70	1.13	1.95	1.30	2.37	1.58
$t_r \times 10^3$, (kips)$^{-1}$	2.08	1.39	2.40	1.60	2.91	1.94
r_x/r_y	1.75		1.74		1.74	
r_y, in.	2.12		2.10		2.08	

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W8

Table IV-5 (continued)

Combined Flexure and Axial Force

W-Shapes

F_y = 50 ksi

Shape	W8×											
	40				35				31[f]			
Design	$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	2.85	1.90	8.95	5.96	3.24	2.16	10.3	6.83	3.66	2.43	11.7	7.80
6	3.13	2.08	8.95	5.96	3.56	2.37	10.3	6.83	4.01	2.67	11.7	7.80
7	3.23	2.15	8.95	5.96	3.68	2.45	10.3	6.83	4.15	2.76	11.7	7.80
8	3.36	2.23	9.07	6.03	3.82	2.54	10.4	6.94	4.32	2.87	11.9	7.94
9	3.50	2.33	9.22	6.14	3.99	2.65	10.6	7.07	4.51	3.00	12.2	8.11
10	3.68	2.45	9.38	6.24	4.19	2.79	10.8	7.21	4.74	3.15	12.5	8.29
11	3.88	2.58	9.55	6.35	4.42	2.94	11.1	7.36	5.00	3.33	12.7	8.48
12	4.11	2.73	9.72	6.47	4.68	3.12	11.3	7.51	5.30	3.53	13.0	8.67
13	4.38	2.91	9.90	6.59	4.99	3.32	11.5	7.67	5.66	3.76	13.3	8.88
14	4.69	3.12	10.1	6.71	5.35	3.56	11.8	7.83	6.07	4.04	13.7	9.09
15	5.04	3.36	10.3	6.84	5.76	3.83	12.0	8.00	6.54	4.35	14.0	9.32
16	5.46	3.63	10.5	6.97	6.24	4.15	12.3	8.18	7.08	4.71	14.4	9.56
17	5.93	3.95	10.7	7.11	6.79	4.51	12.6	8.37	7.71	5.13	14.7	9.81
18	6.48	4.31	10.9	7.25	7.42	4.94	12.9	8.56	8.44	5.62	15.1	10.1
19	7.12	4.73	11.1	7.40	8.16	5.43	13.2	8.77	9.29	6.18	15.6	10.3
20	7.87	5.24	11.4	7.55	9.03	6.01	13.5	8.99	10.3	6.84	16.0	10.6
22	9.52	6.34	11.8	7.88	10.9	7.27	14.2	9.45	12.4	8.28	17.0	11.3
24	11.3	7.54	12.4	8.24	13.0	8.65	15.0	9.97	14.8	9.86	18.0	12.0
26	13.3	8.85	13.0	8.64	15.3	10.2	15.8	10.5	17.4	11.6	19.6	13.1
28	15.4	10.3	13.6	9.07	17.7	11.8	17.0	11.3	20.2	13.4	21.4	14.3
30	17.7	11.8	14.4	9.57	20.3	13.5	18.4	12.3	23.1	15.4	23.3	15.5
32	20.1	13.4	15.4	10.3	23.1	15.4	19.8	13.2	26.3	17.5	25.1	16.7
34	22.7	15.1	16.5	11.0								

Other Constants and Properties

	40		35		31[f]	
	ASD	LRFD	ASD	LRFD	ASD	LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	19.3	12.8	22.1	14.7	25.3	16.8
$t_y \times 10^3$, (kips)$^{-1}$	2.85	1.90	3.24	2.16	3.66	2.43
$t_r \times 10^3$, (kips)$^{-1}$	3.51	2.34	3.98	2.66	4.49	3.00
r_x/r_y	1.73		1.73		1.72	
r_y, in.	2.04		2.03		2.02	

[f] Shape does not meet compact limit for flexure for F_y = 50 ksi.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

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W8

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W8×			
		28			
Design		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	4.05	2.69	13.1	8.71
	6	4.68	3.11	13.2	8.77
	7	4.93	3.28	13.5	9.00
	8	5.23	3.48	13.9	9.23
	9	5.60	3.73	14.2	9.48
	10	6.05	4.02	14.6	9.74
	11	6.58	4.38	15.0	10.0
	12	7.21	4.80	15.5	10.3
	13	7.98	5.31	15.9	10.6
	14	8.89	5.91	16.4	10.9
	15	9.98	6.64	17.0	11.3
	16	11.3	7.54	17.5	11.7
	17	12.8	8.51	18.1	12.0
	18	14.3	9.54	18.7	12.5
	19	16.0	10.6	19.4	12.9
	20	17.7	11.8	20.2	13.4
	22	21.4	14.2	22.1	14.7
	24	25.5	17.0	24.5	16.3
	26	29.9	19.9	26.9	17.9
	28				
	30				
	32				
	34				
	36				
	38				
	40				

Other Constants and Properties

	ASD	LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	35.3	23.5
$t_y \times 10^3$, (kips)$^{-1}$	4.05	2.69
$t_r \times 10^3$, (kips)$^{-1}$	4.97	3.32
r_x/r_y	2.13	
r_y, in.	1.62	

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W8

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W8×											
		24				21				18			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	4.72	3.14	15.4	10.3	5.42	3.61	17.5	11.6	6.35	4.22	21.0	13.9
	1	4.74	3.15	15.4	10.3	5.46	3.63	17.5	11.6	6.39	4.25	21.0	13.9
	2	4.79	3.19	15.4	10.3	5.57	3.70	17.5	11.6	6.53	4.34	21.0	13.9
	3	4.89	3.26	15.4	10.3	5.76	3.83	17.5	11.6	6.76	4.50	21.0	13.9
	4	5.03	3.35	15.4	10.3	6.03	4.01	17.5	11.6	7.10	4.72	21.0	13.9
	5	5.22	3.47	15.4	10.3	6.40	4.26	17.8	11.9	7.56	5.03	21.5	14.3
	6	5.46	3.63	15.6	10.4	6.88	4.58	18.5	12.3	8.16	5.43	22.5	15.0
	7	5.76	3.83	16.0	10.6	7.50	4.99	19.2	12.8	8.93	5.94	23.5	15.6
	8	6.12	4.07	16.5	11.0	8.29	5.51	20.0	13.3	9.91	6.60	24.6	16.4
	9	6.56	4.36	17.0	11.3	9.28	6.17	20.9	13.9	11.2	7.42	25.9	17.2
	10	7.08	4.71	17.5	11.7	10.5	7.00	21.9	14.5	12.7	8.47	27.3	18.1
	11	7.71	5.13	18.1	12.0	12.1	8.05	22.9	15.2	14.7	9.81	28.8	19.2
	12	8.47	5.63	18.7	12.4	14.1	9.39	24.1	16.0	17.3	11.5	30.5	20.3
	13	9.37	6.24	19.3	12.9	16.6	11.0	25.3	16.8	20.3	13.5	32.5	21.6
	14	10.5	6.96	20.0	13.3	19.2	12.8	26.7	17.8	23.6	15.7	35.3	23.5
	15	11.8	7.83	20.8	13.8	22.0	14.7	28.5	18.9	27.1	18.0	38.8	25.8
	16	13.4	8.89	21.6	14.4	25.1	16.7	30.9	20.6	30.8	20.5	42.4	28.2
	17	15.1	10.0	22.5	14.9	28.3	18.8	33.4	22.2	34.8	23.1	45.9	30.5
	18	16.9	11.3	23.4	15.6	31.7	21.1	35.9	23.9	39.0	26.0	49.4	32.9
	19	18.8	12.5	24.5	16.3	35.4	23.5	38.3	25.5	43.5	28.9	52.9	35.2
	20	20.9	13.9	26.1	17.4	39.2	26.1	40.7	27.1	48.2	32.0	56.4	37.5
	21	23.0	15.3	27.8	18.5	43.2	28.7	43.2	28.7				
	22	25.3	16.8	29.4	19.6								
	23	27.6	18.4	31.0	20.6								
	24	30.1	20.0	32.6	21.7								
	25	32.6	21.7	34.2	22.8								

Other Constants and Properties

	W8×24 ASD	W8×24 LRFD	W8×21 ASD	W8×21 LRFD	W8×18 ASD	W8×18 LRFD
$b_y \times 10^3$, (kip-ft)$^{-1}$	41.6	27.7	62.6	41.7	76.5	50.9
$t_y \times 10^3$, (kips)$^{-1}$	4.72	3.14	5.42	3.61	6.35	4.22
$t_r \times 10^3$, (kips)$^{-1}$	5.79	3.86	6.66	4.44	7.80	5.20
r_x/r_y	2.12		2.77		2.79	
r_y, in.	1.61		1.26		1.23	

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W8

Table IV-5 (continued)
Combined Flexure and Axial Force
W-Shapes

F_y = 50 ksi

Shape		W8×											
		15				13				$10^{c,f}$			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to the least radius of gyration, r_y, or Unbraced Length, L_b (ft), for X-X axis bending	0	7.52	5.01	26.2	17.4	8.70	5.79	31.3	20.8	11.7	7.76	40.6	27.0
	1	7.63	5.07	26.2	17.4	8.83	5.87	31.3	20.8	11.8	7.86	40.6	27.0
	2	7.95	5.29	26.2	17.4	9.23	6.14	31.3	20.8	12.3	8.17	40.6	27.0
	3	8.51	5.66	26.2	17.4	9.94	6.61	31.3	20.8	13.1	8.71	40.6	27.0
	4	9.37	6.23	27.6	18.4	11.0	7.34	33.4	22.2	14.3	9.53	43.2	28.8
	5	10.6	7.05	29.4	19.5	12.6	8.38	35.7	23.8	16.4	10.9	46.7	31.1
	6	12.3	8.20	31.3	20.8	14.8	9.86	38.5	25.6	19.3	12.8	50.8	33.8
	7	14.7	9.80	33.6	22.4	18.0	12.0	41.7	27.7	23.4	15.6	55.7	37.0
	8	18.1	12.0	36.2	24.1	22.5	14.9	45.4	30.2	29.3	19.5	61.6	41.0
	9	22.8	15.2	39.3	26.1	28.4	18.9	50.0	33.2	37.1	24.7	71.3	47.4
	10	28.1	18.7	42.9	28.6	35.1	23.4	57.4	38.2	45.8	30.4	84.3	56.1
	11	34.0	22.6	48.9	32.5	42.5	28.3	65.8	43.8	55.4	36.8	97.6	64.9
	12	40.5	26.9	54.9	36.5	50.6	33.6	74.3	49.4	65.9	43.8	111	73.9
	13	47.5	31.6	60.9	40.5	59.3	39.5	82.7	55.0	77.3	51.5	125	83.0
	14	55.1	36.7	66.9	44.5	68.8	45.8	91.2	60.7	89.7	59.7	139	92.2

Other Constants and Properties

	15		13		$10^{c,f}$	
$b_y \times 10^3$, (kip-ft)$^{-1}$	133	88.8	166	110	218	145
$t_y \times 10^3$, (kips)$^{-1}$	7.52	5.01	8.70	5.79	11.3	7.51
$t_r \times 10^3$, (kips)$^{-1}$	9.24	6.16	10.7	7.12	13.9	9.24
r_x / r_y	3.76		3.81		3.83	
r_y, in.	0.876		0.843		0.841	

[c] Shape is slender for compression for F_y = 50 ksi.

[f] Shape does not meet compact limit for flexure for F_y = 50 ksi.

Note: Heavy line indicates L_c / r_y equal to or greater than 200.

Table IV-6A
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W44

F_y = 65 ksi
F_u = 80 ksi

W44×						Shape	W44×					
335^c		290^c		262^c		lb/ft	335		290		262^v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
3630	5460	3010	4530	2650	3990	0	5250	7900	4570	6870	4120	6190
3510	5270	2910	4380	2560	3850	6	5250	7900	4570	6870	4120	6190
3470	5210	2870	4320	2530	3800	7	5250	7900	4570	6870	4120	6190
3420	5130	2830	4260	2490	3750	8	5250	7900	4570	6870	4120	6190
3360	5050	2790	4190	2450	3680	9	5250	7900	4570	6870	4120	6190
3300	4960	2740	4110	2410	3620	10	5250	7900	4570	6870	4120	6190
3230	4860	2680	4030	2360	3540	11	5240	7870	4560	6850	4100	6160
3160	4750	2620	3940	2300	3460	12	5140	7730	4470	6720	4020	6040
3090	4640	2560	3840	2250	3380	13	5050	7590	4390	6590	3940	5920
3010	4520	2490	3750	2190	3290	14	4950	7450	4300	6460	3850	5790
2930	4400	2420	3640	2130	3200	15	4860	7310	4210	6330	3770	5670
2840	4270	2350	3530	2060	3100	16	4770	7160	4130	6200	3690	5550
2750	4130	2280	3420	2000	3000	17	4670	7020	4040	6080	3610	5430
2660	4000	2200	3310	1930	2900	18	4580	6880	3960	5950	3530	5310
2560	3840	2120	3190	1860	2800	19	4480	6740	3870	5820	3450	5180
2450	3680	2040	3070	1790	2690	20	4390	6600	3780	5690	3370	5060
2230	3340	1880	2830	1650	2480	22	4200	6320	3610	5430	3200	4820
2010	3020	1720	2590	1510	2270	24	4010	6030	3440	5170	3040	4570
1790	2700	1560	2340	1370	2050	26	3830	5750	3270	4910	2880	4330
1590	2390	1380	2070	1230	1850	28	3640	5470	3100	4660	2720	4080
1390	2090	1210	1810	1080	1620	30	3450	5180	2930	4400	2550	3840
1220	1840	1060	1590	948	1420	32	3260	4900	2710	4070	2310	3480
1080	1630	939	1410	839	1260	34	3000	4510	2460	3700	2090	3150
966	1450	838	1260	749	1130	36	2750	4140	2250	3380	1910	2870
867	1300	752	1130	672	1010	38	2540	3820	2070	3110	1750	2630
783	1180	679	1020	606	911	40	2360	3550	1920	2880	1620	2430
710	1070	616	925	550	827	42	2200	3310	1780	2680	1500	2260
647	972	561	843	501	753	44	2060	3100	1660	2500	1400	2100
592	890	513	771	459	689	46	1940	2910	1560	2350	1310	1970
544	817	471	708	421	633	48	1830	2750	1470	2210	1230	1850
501	753	434	653	388	583	50	1730	2600	1390	2090	1160	1740

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3830	5760	3320	5000	3000	4520
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2960	4430	2560	3840	2320	3470
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1180	1770	981	1470	794	1190
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
765	1150	665	999	590	887

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.8	32.6	10.8	31.3	10.7	30.4
Area, in.^{2}					
98.5		85.4		77.2	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
31100	1200	27000	1040	24100	923
r_y, in.					
3.49		3.49		3.47	
r_x/r_y					
5.10		5.10		5.10	

[c] Shape is slender for compression with F_y = 65 ksi.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.
Note: Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W44–W40

F_y = 65 ksi
F_u = 80 ksi

W44×		W40×				Shape	W44×		W40×			
230^c		655^h		593^h		lb/ft	230^v		655^h		593^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2260	3400	7510	11300	6770	10200	0	3570	5360	9990	15000	8950	13500
2180	3280	7270	10900	6550	9840	6	3570	5360	9990	15000	8950	13500
2150	3240	7180	10800	6470	9720	7	3570	5360	9990	15000	8950	13500
2120	3190	7080	10600	6370	9580	8	3570	5360	9990	15000	8950	13500
2080	3130	6970	10500	6270	9430	9	3570	5360	9990	15000	8950	13500
2040	3070	6850	10300	6160	9260	10	3570	5360	9990	15000	8950	13500
2000	3010	6720	10100	6040	9080	11	3540	5320	9990	15000	8950	13500
1960	2940	6580	9890	5910	8880	12	3470	5210	9990	15000	8930	13400
1910	2870	6430	9670	5770	8670	13	3400	5100	9890	14900	8840	13300
1860	2790	6270	9430	5620	8450	14	3320	5000	9790	14700	8740	13100
1800	2710	6110	9180	5470	8220	15	3250	4890	9700	14600	8650	13000
1750	2630	5940	8920	5310	7990	16	3180	4780	9600	14400	8550	12900
1690	2540	5760	8660	5150	7740	17	3110	4670	9510	14300	8460	12700
1630	2450	5580	8380	4980	7490	18	3030	4560	9410	14100	8370	12600
1570	2360	5390	8100	4810	7230	19	2960	4450	9320	14000	8270	12400
1510	2270	5200	7820	4640	6970	20	2890	4340	9220	13900	8180	12300
1390	2090	4820	7240	4280	6430	22	2740	4120	9030	13600	7990	12000
1270	1910	4430	6650	3920	5900	24	2600	3910	8840	13300	7800	11700
1150	1730	4040	6070	3570	5360	26	2450	3690	8650	13000	7610	11400
1030	1550	3660	5490	3220	4840	28	2310	3470	8450	12700	7420	11200
917	1380	3290	4940	2890	4340	30	2130	3200	8260	12400	7240	10900
813	1220	2930	4410	2560	3850	32	1910	2870	8070	12100	7050	10600
720	1080	2600	3900	2270	3410	34	1720	2590	7880	11800	6860	10300
642	966	2320	3480	2020	3040	36	1570	2350	7690	11600	6670	10000
577	867	2080	3120	1820	2730	38	1430	2150	7500	11300	6480	9740
520	782	1880	2820	1640	2460	40	1320	1980	7310	11000	6290	9460
472	709	1700	2560	1490	2230	42	1220	1830	7110	10700	6110	9180
430	646	1550	2330	1350	2040	44	1130	1700	6920	10400	5920	8900
393	591	1420	2130	1240	1860	46	1060	1590	6730	10100	5730	8610
361	543	1300	1960	1140	1710	48	991	1490	6540	9830	5540	8330
333	501	1200	1800	1050	1580	50	932	1400	6350	9540	5350	8050

Properties

Available Strength in Tensile Yielding, kips							Limiting Unbraced Lengths, ft					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		L_p	L_r	L_p	L_r	L_p	L_r
2640	3970	7510	11300	6770	10200		10.6	29.5	12.0	54.9	11.8	50.4
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips							Area, in.^{2}					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		67.8		193		174	
2030	3050	5790	8690	5220	7830		Moment of Inertia, in.^{4}					
Available Strength in Shear, kips							I_x	I_y	I_x	I_y	I_x	I_y
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$		20800	796	56500	2870	50400	2520
697	1050	2230	3350	2000	3000		r_y, in.					
Available Strength in Flexure about Y-Y Axis, kip-ft							3.43		3.86		3.80	
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$		r_x/r_y					
509	765	1760	2640	1560	2340		5.10		4.43		4.47	

[c] Shape is slender for compression with F_y = 65 ksi.
[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.
Note: Confirm ASTM A913 material availability before specifying.

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W40

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W40×						Shape	W40×					
503^h		431^h		397^h		lb/ft	503^h		431^h		397^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
5760	8660	4940	7430	4550	6840	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending 0	7520	11300	6360	9560	5840	8780
5560	8360	4760	7160	4390	6590	6	7520	11300	6360	9560	5840	8780
5490	8250	4700	7060	4330	6510	7	7520	11300	6360	9560	5840	8780
5410	8130	4630	6960	4260	6410	8	7520	11300	6360	9560	5840	8780
5320	7990	4550	6840	4190	6300	9	7520	11300	6360	9560	5840	8780
5220	7840	4460	6700	4110	6170	10	7520	11300	6360	9560	5840	8780
5110	7680	4370	6560	4020	6040	11	7520	11300	6360	9560	5840	8780
5000	7510	4260	6410	3920	5900	12	7480	11200	6300	9460	5780	8680
4870	7330	4160	6250	3820	5750	13	7390	11100	6210	9330	5690	8550
4750	7130	4040	6070	3720	5590	14	7300	11000	6120	9200	5610	8420
4610	6930	3920	5900	3610	5420	15	7200	10800	6030	9060	5520	8300
4470	6720	3800	5710	3500	5250	16	7110	10700	5940	8930	5430	8170
4330	6510	3670	5520	3380	5080	17	7020	10500	5850	8800	5350	8040
4180	6280	3540	5330	3260	4900	18	6920	10400	5760	8660	5260	7910
4030	6060	3410	5130	3140	4710	19	6830	10300	5680	8530	5180	7780
3880	5830	3280	4930	3010	4530	20	6740	10100	5590	8400	5090	7650
3570	5360	3010	4520	2760	4150	22	6550	9850	5410	8130	4920	7390
3260	4900	2740	4110	2510	3780	24	6370	9570	5230	7870	4750	7140
2950	4440	2470	3710	2270	3400	26	6180	9290	5060	7600	4580	6880
2650	3990	2210	3320	2030	3050	28	6000	9010	4880	7330	4410	6620
2370	3550	1960	2950	1800	2700	30	5810	8730	4700	7070	4240	6370
2090	3140	1720	2590	1580	2380	32	5620	8450	4520	6800	4060	6110
1850	2780	1530	2300	1400	2100	34	5440	8170	4350	6530	3890	5850
1650	2480	1360	2050	1250	1880	36	5250	7890	4170	6270	3720	5590
1480	2230	1220	1840	1120	1680	38	5070	7620	3990	6000	3550	5340
1340	2010	1100	1660	1010	1520	40	4880	7340	3810	5720	3320	4990
1210	1820	1000	1500	917	1380	42	4700	7060	3580	5370	3110	4680
1100	1660	912	1370	836	1260	44	4510	6780	3370	5070	2930	4410
1010	1520	835	1250	765	1150	46	4280	6430	3190	4800	2770	4170
928	1390	767	1150	702	1060	48	4070	6110	3030	4550	2630	3950
855	1290	706	1060	647	973	50	3880	5830	2880	4330	2500	3760

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
5760	8660	4940	7430	4550	6840
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4440	6660	3810	5720	3510	5270
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1690	2530	1440	2160	1300	1950
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
1280	1920	1060	1600	973	1460

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.5	44.2	11.3	39.8	11.3	38.1
Area, in.^{2}					
148		127		117	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
41600	2040	34800	1690	32000	1540
r_y, in.					
3.72		3.65		3.64	
r_x/r_y					
4.52		4.55		4.56	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

F_y = 65 ksi
F_u = 80 ksi

W40

W-Shapes

W40× 372^h		W40× 362^h		W40× 324^c		Shape lb/ft		W40× 372^h		W40× 362^h		W40× 324	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending		ASD	LRFD	ASD	LRFD	ASD	LRFD
4280	6430	4130	6200	3630	5460		0	5450	8190	5320	8000	4740	7120
4120	6190	3970	5970	3510	5280		6	5450	8190	5320	8000	4740	7120
4070	6110	3920	5890	3470	5220		7	5450	8190	5320	8000	4740	7120
4000	6010	3860	5800	3420	5140		8	5450	8190	5320	8000	4740	7120
3930	5910	3790	5690	3370	5060		9	5450	8190	5320	8000	4740	7120
3850	5790	3710	5580	3310	4970		10	5450	8190	5320	8000	4740	7120
3770	5660	3630	5460	3240	4880		11	5450	8190	5320	8000	4740	7120
3680	5530	3540	5330	3180	4770		12	5380	8080	5250	7890	4660	7010
3580	5380	3450	5190	3100	4650		13	5290	7960	5170	7760	4580	6890
3480	5230	3350	5040	3010	4520		14	5210	7830	5080	7640	4510	6770
3380	5070	3250	4890	2920	4380		15	5130	7700	5000	7510	4430	6650
3270	4910	3150	4730	2820	4240		16	5040	7580	4910	7390	4350	6540
3160	4740	3040	4570	2720	4090		17	4960	7450	4830	7260	4270	6420
3040	4570	2930	4400	2620	3940		18	4870	7320	4750	7140	4190	6300
2920	4400	2820	4240	2520	3790		19	4790	7200	4660	7010	4110	6180
2810	4220	2700	4060	2420	3640		20	4700	7070	4580	6880	4030	6060
2570	3860	2470	3720	2210	3320		22	4540	6820	4410	6630	3870	5820
2330	3500	2250	3370	2010	3010		24	4370	6560	4250	6380	3720	5590
2100	3150	2020	3040	1800	2710		26	4200	6310	4080	6130	3560	5350
1870	2810	1800	2710	1610	2410		28	4030	6060	3910	5880	3400	5110
1650	2490	1590	2390	1420	2130		30	3860	5810	3740	5630	3240	4870
1450	2180	1400	2100	1250	1870		32	3690	5550	3580	5380	3080	4640
1290	1930	1240	1860	1100	1660		34	3530	5300	3410	5130	2930	4400
1150	1730	1110	1660	984	1480		36	3360	5050	3240	4870	2700	4060
1030	1550	993	1490	883	1330		38	3140	4710	3020	4530	2500	3760
930	1400	896	1350	797	1200		40	2930	4400	2810	4230	2330	3500
844	1270	813	1220	723	1090		42	2740	4120	2640	3960	2180	3270
769	1160	741	1110	659	990		44	2580	3880	2480	3730	2050	3070
703	1060	678	1020	603	906		46	2440	3660	2340	3520	1930	2900
646	971	622	935	553	832		48	2310	3470	2220	3330	1820	2740
595	895	574	862	510	766		50	2190	3300	2110	3170	1730	2600

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4280	6440	4130	6200	3710	5580
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3300	4950	3180	4770	2860	4290
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1220	1840	1180	1770	1050	1570
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
898	1350	876	1320	775	1170

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.2	36.5	11.2	36.2	11.1	34.3
Area, in.^{2}					
110		106		95.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
29600	1420	28900	1380	25600	1220
r_y, in.					
3.60		3.60		3.58	
r_x/r_y					
4.58		4.58		4.58	

[c] Shape is slender for compression with F_y = 65 ksi.
[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
Note: Confirm ASTM A913 material availability before specifying.

W40

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W40×						Shape	W40×					
297^c		277^c		249^c		lb/ft	297		277		249	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
3270	4910	2970	4470	2620	3930	0	4310	6480	4050	6090	3630	5460
3160	4740	2870	4320	2530	3800	6	4310	6480	4050	6090	3630	5460
3120	4680	2840	4270	2500	3750	7	4310	6480	4050	6090	3630	5460
3070	4620	2800	4210	2460	3700	8	4310	6480	4050	6090	3630	5460
3020	4540	2750	4140	2420	3640	9	4310	6480	4050	6090	3630	5460
2970	4460	2710	4070	2380	3570	10	4310	6480	4050	6090	3630	5460
2910	4370	2650	3990	2330	3500	11	4310	6480	4050	6090	3630	5460
2850	4280	2600	3900	2280	3430	12	4240	6370	3990	5990	3570	5360
2780	4180	2540	3810	2230	3350	13	4160	6250	3920	5890	3500	5260
2710	4070	2470	3720	2170	3260	14	4080	6140	3840	5780	3430	5160
2640	3960	2410	3620	2110	3170	15	4010	6030	3770	5670	3360	5060
2560	3850	2340	3510	2050	3080	16	3930	5910	3700	5560	3300	4950
2480	3720	2270	3410	1980	2980	17	3860	5800	3630	5450	3230	4850
2390	3580	2190	3300	1920	2880	18	3780	5690	3550	5340	3160	4750
2290	3440	2120	3180	1850	2780	19	3710	5570	3480	5230	3090	4650
2200	3300	2040	3070	1780	2680	20	3630	5460	3410	5130	3030	4550
2000	3010	1890	2840	1650	2480	22	3480	5230	3270	4910	2890	4350
1810	2720	1710	2580	1510	2270	24	3330	5000	3120	4690	2760	4150
1620	2440	1540	2320	1370	2060	26	3180	4780	2980	4470	2620	3940
1440	2170	1370	2060	1220	1840	28	3030	4550	2830	4260	2490	3740
1270	1910	1210	1820	1070	1610	30	2880	4320	2690	4040	2350	3540
1120	1680	1060	1600	944	1420	32	2730	4100	2540	3820	2190	3300
988	1480	943	1420	836	1260	34	2530	3800	2340	3520	1990	2990
881	1320	841	1260	746	1120	36	2320	3490	2150	3220	1820	2740
791	1190	755	1130	670	1010	38	2150	3220	1980	2970	1680	2520
714	1070	681	1020	604	908	40	1990	3000	1840	2760	1550	2330
647	973	618	929	548	824	42	1860	2800	1710	2570	1440	2170
590	887	563	846	499	751	44	1740	2620	1600	2410	1350	2030
540	811	515	774	457	687	46	1640	2470	1510	2260	1270	1900
496	745	473	711	420	631	48	1550	2330	1420	2140	1190	1790
457	687	436	655	387	581	50	1470	2210	1340	2020	1130	1690

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3400	5110	3170	4770	2860	4300
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2620	3930	2450	3670	2210	3310
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
962	1440	857	1290	768	1150
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
697	1050	662	995	590	887

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.0	32.9	11.1	32.6	11.0	31.5
Area, in.^{2}					
87.3		81.5		73.5	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
23200	1090	21900	1040	19600	926
r_y, in.					
3.54		3.58		3.55	
r_x/r_y					
4.60		4.58		4.59	

^{c} Shape is slender for compression with F_y = 65 ksi.

Note: Confirm ASTM A913 material availability before specifying.

W40

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W40×						Shape		W40×					
215^c		199^c		392^h		lb/ft		215^v		199^v		392^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
2190	3290	2010	3010	4510	6790	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	3130	4700	2820	4240	5550	8340
2110	3180	1930	2910	4210	6320		6	3130	4700	2820	4240	5550	8340
2090	3140	1910	2870	4100	6160		7	3130	4700	2820	4240	5550	8340
2060	3090	1880	2830	3980	5980		8	3130	4700	2820	4240	5550	8340
2020	3040	1850	2780	3850	5790		9	3130	4700	2820	4240	5460	8210
1990	2980	1810	2720	3710	5580		10	3130	4700	2820	4240	5360	8060
1950	2920	1770	2670	3560	5350		11	3120	4700	2800	4210	5260	7910
1900	2860	1730	2610	3400	5110		12	3060	4610	2740	4120	5160	7760
1860	2790	1690	2540	3240	4870		13	3000	4510	2690	4040	5060	7610
1810	2720	1650	2470	3070	4620		14	2940	4420	2630	3950	4960	7460
1760	2640	1600	2400	2900	4360		15	2880	4330	2570	3870	4860	7310
1710	2570	1550	2330	2730	4100		16	2820	4240	2520	3780	4760	7160
1650	2490	1500	2250	2560	3850		17	2760	4150	2460	3690	4660	7010
1600	2400	1450	2180	2390	3590		18	2700	4060	2400	3610	4560	6850
1540	2320	1390	2100	2220	3340		19	2640	3970	2340	3520	4460	6700
1490	2230	1340	2020	2060	3090		20	2580	3880	2290	3440	4360	6550
1370	2060	1230	1850	1740	2620		22	2460	3700	2170	3270	4160	6250
1260	1890	1120	1690	1470	2200		24	2340	3510	2060	3090	3960	5950
1140	1710	1020	1530	1250	1880		26	2220	3330	1940	2920	3760	5650
1030	1550	914	1370	1080	1620		28	2100	3150	1830	2750	3560	5350
918	1380	812	1220	938	1410		30	1980	2970	1690	2540	3360	5040
811	1220	713	1070	824	1240		32	1790	2690	1510	2270	3120	4690
719	1080	632	950	730	1100		34	1620	2430	1370	2050	2900	4360
641	963	564	847	651	979		36	1470	2220	1240	1870	2710	4070
575	865	506	760	584	878		38	1350	2030	1140	1710	2540	3810
519	780	457	686	527	793		40	1250	1880	1050	1570	2390	3590
471	708	414	622	478	719		42	1160	1740	969	1460	2260	3390
429	645	377	567	436	655		44	1080	1620	901	1350	2140	3220
393	590	345	519				46	1010	1520	841	1260	2040	3060
361	542	317	477				48	947	1420	788	1190	1940	2920
332	499	292	439				50	892	1340	742	1110	1850	2790

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2470	3710	2290	3440	4510	6790
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1910	2860	1760	2650	3480	5220
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
592	890	587	883	1540	2300
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
506	761	444	668	675	1010

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.0	30.4	10.7	29.4	8.18	30.9
Area, in.^{2}					
63.5		58.8		116	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
16700	803	14900	695	29900	803
r_y, in.					
3.54		3.45		2.64	
r_x/r_y					
4.58		4.64		6.10	

^{c} Shape is slender for compression with F_y = 65 ksi.

^{h} Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

^{v} Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W40

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

Shape lb/ft	W40×331^h		W40×327^h		W40×294^c		W40×331^h		W40×327^h		W40×294	
	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Design	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending												
0	3800	5720	3730	5610	3330	5010	4640	6970	4570	6870	4120	6190
6	3530	5300	3470	5210	3110	4670	4640	6970	4570	6870	4120	6190
7	3440	5160	3370	5070	3030	4550	4640	6970	4570	6870	4120	6190
8	3330	5010	3270	4920	2930	4410	4630	6970	4570	6870	4110	6180
9	3220	4830	3160	4750	2830	4250	4540	6820	4480	6730	4020	6040
10	3090	4650	3040	4570	2720	4090	4440	6680	4380	6590	3930	5900
11	2960	4450	2910	4370	2600	3910	4350	6530	4290	6450	3830	5760
12	2820	4240	2780	4170	2480	3720	4250	6390	4190	6300	3740	5620
13	2680	4030	2640	3960	2350	3530	4150	6240	4100	6160	3650	5490
14	2530	3810	2490	3750	2220	3340	4060	6100	4010	6020	3560	5350
15	2390	3590	2350	3530	2090	3140	3960	5950	3910	5880	3470	5210
16	2240	3360	2200	3310	1960	2940	3860	5810	3820	5740	3370	5070
17	2090	3140	2060	3100	1830	2740	3770	5660	3720	5590	3280	4930
18	1940	2920	1920	2880	1700	2550	3670	5520	3630	5450	3190	4790
19	1800	2700	1780	2670	1570	2360	3580	5370	3530	5310	3100	4660
20	1660	2490	1640	2460	1450	2170	3480	5230	3440	5170	3010	4520
22	1390	2090	1380	2070	1210	1820	3290	4940	3250	4880	2820	4240
24	1170	1760	1160	1740	1020	1530	3090	4650	3060	4600	2640	3960
26	996	1500	986	1480	865	1300	2900	4360	2870	4310	2450	3690
28	859	1290	850	1280	746	1120	2690	4050	2660	4000	2210	3320
30	748	1120	740	1110	650	977	2460	3690	2430	3650	2010	3020
32	658	989	651	978	571	859	2260	3400	2230	3360	1850	2770
34	583	876	576	866	506	761	2090	3140	2070	3110	1710	2560
36	520	781	514	773	451	679	1950	2930	1920	2890	1580	2380
38	466	701	461	694	405	609	1820	2740	1800	2710	1480	2220
40	421	633	416	626	366	550	1710	2570	1690	2540	1390	2090
42	382	574	378	568	332	499	1620	2430	1600	2400	1310	1970
44							1530	2300	1510	2270	1240	1860
46							1450	2180	1430	2150	1170	1760
48							1380	2080	1360	2050	1120	1680
50							1320	1980	1300	1960	1060	1600

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3800	5720	3730	5610	3360	5040
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2930	4400	2880	4320	2590	3880
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1290	1940	1250	1880	1110	1670
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
550	827	545	819	485	729

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.96	27.6	7.99	27.5	7.90	26.0
Area, in.^{2}					
97.7		95.9		86.2	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
24700	644	24500	640	21900	562
r_y, in.					
2.57		2.58		2.55	
r_x/r_y					
6.19		6.20		6.24	

[c] Shape is slender for compression with F_y = 65 ksi.

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W40

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W40×						Shape	W40×					
278^c		264^c		235^c		lb/ft	278		264		235	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
3150	4730	2900	4370	2490	3740	0	3860	5800	3670	5510	3280	4920
2950	4430	2720	4090	2330	3510	6	3860	5800	3670	5510	3280	4920
2880	4330	2650	3990	2280	3420	7	3860	5800	3670	5510	3280	4920
2790	4190	2580	3880	2220	3330	8	3840	5780	3650	5480	3270	4910
2690	4040	2500	3760	2150	3230	9	3750	5640	3560	5350	3180	4790
2580	3880	2420	3630	2080	3120	10	3670	5510	3480	5220	3100	4670
2470	3710	2320	3490	2000	3000	11	3580	5380	3390	5090	3020	4550
2350	3530	2210	3320	1920	2880	12	3490	5240	3300	4960	2940	4420
2230	3340	2090	3150	1830	2750	13	3400	5110	3220	4840	2860	4300
2100	3160	1970	2970	1740	2620	14	3310	4980	3130	4710	2780	4180
1970	2960	1850	2790	1650	2490	15	3220	4840	3040	4580	2700	4060
1840	2770	1740	2610	1560	2350	16	3130	4710	2960	4450	2620	3940
1720	2580	1620	2430	1460	2190	17	3050	4580	2870	4320	2540	3820
1590	2390	1500	2250	1350	2030	18	2960	4440	2790	4190	2460	3700
1470	2210	1380	2080	1250	1880	19	2870	4310	2700	4060	2380	3580
1350	2030	1270	1910	1150	1730	20	2780	4180	2610	3930	2300	3460
1130	1690	1060	1590	961	1450	22	2600	3910	2440	3670	2140	3210
947	1420	891	1340	808	1210	24	2430	3650	2270	3410	1970	2960
807	1210	759	1140	688	1030	26	2220	3330	2050	3080	1740	2620
696	1050	654	984	594	892	28	2000	3000	1840	2760	1560	2350
606	911	570	857	517	777	30	1810	2730	1670	2510	1410	2120
533	801	501	753	454	683	32	1660	2500	1530	2300	1290	1940
472	709	444	667	403	605	34	1530	2300	1410	2120	1180	1780
421	633	396	595	359	540	36	1420	2140	1310	1960	1100	1650
378	568	355	534	322	484	38	1330	2000	1220	1830	1020	1530
341	512	321	482	291	437	40	1250	1870	1140	1710	953	1430
309	465	291	437	264	396	42	1170	1760	1070	1610	895	1350
						44	1110	1670	1010	1520	844	1270
						46	1050	1580	959	1440	798	1200
						48	998	1500	911	1370	757	1140
						50	951	1430	867	1300	720	1080

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3200	4810	3010	4530	2690	4040
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2470	3700	2320	3480	2070	3110
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1080	1610	998	1500	857	1290
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
452	679	428	644	383	575

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.81	25.2	7.81	24.7	7.87	23.9
Area, in.^{2}					
82.3		77.4		69.1	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
20500	521	19400	493	17400	444
r_y, in.					
2.52		2.52		2.54	
r_x/r_y					
6.27		6.27		6.26	

[c] Shape is slender for compression with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W40

F_y = 65 ksi
F_u = 80 ksi

W40×						Shape	W40×					
211^c		183^c		167^c		lb/ft	211		183^v		167^v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2170	3270	1790	2690	1640	2460	0	2940	4420	2510	3770	2250	3380
2030	3050	1670	2510	1520	2290	6	2940	4420	2510	3770	2250	3380
1980	2980	1630	2450	1480	2230	7	2940	4420	2510	3770	2250	3380
1930	2900	1590	2380	1440	2160	8	2920	4390	2490	3740	2210	3320
1870	2810	1540	2310	1390	2090	9	2850	4280	2420	3640	2150	3230
1800	2710	1480	2230	1340	2010	10	2770	4160	2350	3540	2080	3130
1730	2610	1420	2140	1280	1930	11	2690	4050	2290	3440	2020	3040
1660	2500	1360	2050	1220	1840	12	2620	3930	2220	3330	1960	2940
1590	2380	1300	1950	1160	1750	13	2540	3820	2150	3230	1890	2840
1510	2270	1230	1860	1100	1660	14	2470	3710	2080	3130	1830	2750
1430	2150	1170	1760	1040	1560	15	2390	3590	2010	3030	1760	2650
1350	2030	1100	1660	978	1470	16	2310	3480	1950	2920	1700	2550
1270	1910	1040	1560	915	1380	17	2240	3360	1880	2820	1640	2460
1190	1790	970	1460	853	1280	18	2160	3250	1810	2720	1570	2360
1100	1660	904	1360	792	1190	19	2090	3140	1740	2620	1510	2270
1010	1520	840	1260	732	1100	20	2010	3020	1670	2510	1440	2170
844	1270	713	1070	612	920	22	1860	2790	1540	2310	1280	1930
709	1070	599	900	515	773	24	1660	2500	1330	2000	1110	1660
604	908	510	767	438	659	26	1470	2200	1170	1750	967	1450
521	783	440	661	378	568	28	1310	1970	1040	1560	857	1290
454	682	383	576	329	495	30	1180	1770	929	1400	767	1150
399	599	337	506	289	435	32	1070	1610	842	1270	693	1040
353	531	298	448	256	385	34	985	1480	769	1160	631	949
315	474	266	400	229	344	36	909	1370	707	1060	579	870
283	425	239	359	205	309	38	844	1270	654	982	535	803
255	384	216	324	185	278	40	787	1180	608	914	496	746
						42	738	1110	568	854	463	695
						44	694	1040	533	801	433	651
						46	656	986	502	754	407	612
						48	621	934	474	713	384	578
						50	590	887	449	676	364	547

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2420	3630	2070	3120	1920	2880
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1860	2790	1600	2400	1480	2220
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
768	1150	592	890	586	881
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
341	512	286	430	247	371

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.78	23.0	7.71	22.1	7.44	21.3
Area, in.^{2}					
62.1		53.3		49.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
15500	390	13200	331	11600	283
r_y, in.					
2.51		2.49		2.40	
r_x/r_y					
6.29		6.31		6.38	

[c] Shape is slender for compression with F_y = 65 ksi.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W40–W36

F_y = 65 ksi
F_u = 80 ksi

W40×		W36×				Shape		W40×		W36×			
149[c]		925[h]		853[h]		lb/ft		149[v]		925[h]		853[h]	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
1420	2130	10600	15900	9770	14700	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	1940	2920	13400	20100	12700	19100
1310	1970	10300	15500	9510	14300		6	1940	2920	13400	20100	12700	19100
1270	1910	10200	15300	9420	14200		7	1940	2920	13400	20100	12700	19100
1230	1850	10100	15200	9310	14000		8	1890	2840	13400	20100	12700	19100
1190	1780	9960	15000	9200	13800		9	1830	2750	13400	20100	12700	19100
1140	1710	9820	14800	9070	13600		10	1770	2660	13400	20100	12700	19100
1090	1630	9660	14500	8920	13400		11	1710	2570	13400	20100	12700	19100
1030	1560	9500	14300	8770	13200		12	1650	2480	13400	20100	12700	19100
980	1470	9320	14000	8610	12900		13	1590	2390	13400	20100	12700	19100
925	1390	9130	13700	8440	12700		14	1530	2310	13300	20000	12700	19000
869	1310	8930	13400	8260	12400		15	1480	2220	13200	19900	12600	18900
812	1220	8730	13100	8070	12100		16	1420	2130	13200	19800	12500	18800
757	1140	8510	12800	7870	11800		17	1360	2040	13100	19700	12400	18600
702	1050	8290	12500	7670	11500		18	1300	1950	13000	19500	12300	18500
647	973	8060	12100	7460	11200		19	1240	1860	12900	19400	12200	18400
595	894	7830	11800	7250	10900		20	1180	1780	12800	19300	12200	18300
495	745	7350	11000	6800	10200		22	1010	1510	12700	19000	12000	18000
416	626	6860	10300	6350	9550		24	866	1300	12500	18800	11800	17800
355	533	6360	9560	5900	8860		26	755	1140	12300	18600	11700	17500
306	460	5860	8810	5440	8170		28	667	1000	12200	18300	11500	17300
266	400	5370	8070	4990	7500		30	595	895	12000	18100	11300	17000
234	352	4890	7350	4550	6830		32	537	806	11800	17800	11200	16800
207	312	4430	6650	4120	6190		34	487	733	11700	17600	11000	16500
185	278	3980	5980	3700	5570		36	446	670	11500	17300	10800	16300
166	250	3570	5360	3320	5000		38	411	617	11400	17100	10700	16000
		3220	4840	3000	4510		40	380	572	11200	16800	10500	15800
		2920	4390	2720	4090		42	354	532	11000	16600	10300	15500
		2660	4000	2480	3730		44	331	497	10900	16300	10200	15300
		2430	3660	2270	3410		46	310	466	10700	16100	10000	15000
		2240	3360	2080	3130		48	292	439	10500	15800	9830	14800
		2060	3100	1920	2890		50	276	415	10400	15600	9670	14500

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1700	2560	10600	15900	9770	14700
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1310	1970	8160	12200	7530	11300
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
556	835	3380	5080	2820	4240
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
201	303	2760	4140	2610	3920

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.09	20.3	13.2	82.5	13.3	77.6
Area, in.^{2}					
43.8		272		251	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
9800	229	73000	4940	70000	4600
r_y, in.					
2.29		4.26		4.28	
r_x/r_y					
6.55		3.85		3.90	

[c] Shape is slender for compression with F_y = 65 ksi.

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

W36

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

F_y = 65 ksi
F_u = 80 ksi

W-Shapes

W36×						Shape	W36×					
802^h		723^h		652^h		lb/ft	802^h		723^h		652^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
9190	13800	8290	12500	7470	11200	0	11900	17800	10600	15900	9440	14200
8930	13400	8060	12100	7260	10900	6	11900	17800	10600	15900	9440	14200
8850	13300	7980	12000	7180	10800	7	11900	17800	10600	15900	9440	14200
8740	13100	7880	11800	7090	10700	8	11900	17800	10600	15900	9440	14200
8630	13000	7780	11700	7000	10500	9	11900	17800	10600	15900	9440	14200
8510	12800	7660	11500	6890	10400	10	11900	17800	10600	15900	9440	14200
8370	12600	7540	11300	6770	10200	11	11900	17800	10600	15900	9440	14200
8220	12400	7400	11100	6650	9990	12	11900	17800	10600	15900	9440	14200
8070	12100	7260	10900	6510	9790	13	11900	17800	10600	15900	9410	14200
7900	11900	7110	10700	6370	9580	14	11800	17700	10500	15800	9330	14000
7730	11600	6940	10400	6220	9350	15	11700	17600	10400	15700	9250	13900
7540	11300	6780	10200	6070	9120	16	11600	17500	10400	15600	9170	13800
7360	11100	6600	9930	5910	8880	17	11500	17400	10300	15400	9090	13700
7160	10800	6420	9660	5740	8630	18	11500	17200	10200	15300	9010	13500
6960	10500	6240	9380	5570	8370	19	11400	17100	10100	15200	8930	13400
6750	10200	6050	9090	5400	8110	20	11300	17000	10000	15100	8850	13300
6330	9520	5660	8510	5040	7570	22	11100	16700	9860	14800	8690	13100
5900	8870	5270	7920	4680	7030	24	11000	16500	9700	14600	8530	12800
5460	8210	4870	7320	4310	6480	26	10800	16200	9540	14300	8370	12600
5030	7560	4470	6720	3950	5930	28	10600	16000	9370	14100	8210	12300
4600	6910	4080	6140	3590	5400	30	10500	15700	9210	13800	8050	12100
4180	6280	3700	5570	3250	4880	32	10300	15500	9050	13600	7890	11900
3780	5680	3340	5020	2910	4380	34	10100	15300	8880	13400	7730	11600
3380	5090	2980	4480	2600	3910	36	9980	15000	8720	13100	7570	11400
3040	4570	2680	4020	2330	3510	38	9820	14800	8560	12900	7410	11100
2740	4120	2420	3630	2110	3160	40	9650	14500	8390	12600	7250	10900
2490	3740	2190	3290	1910	2870	42	9490	14300	8230	12400	7090	10700
2270	3410	2000	3000	1740	2620	44	9320	14000	8070	12100	6930	10400
2070	3120	1830	2750	1590	2390	46	9160	13800	7900	11900	6770	10200
1900	2860	1680	2520	1460	2200	48	8990	13500	7740	11600	6610	9940
1750	2640	1550	2320	1350	2030	50	8830	13300	7580	11400	6450	9700

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
9190	13800	8290	12500	7470	11200
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
7080	10600	6390	9590	5760	8640
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
2640	3950	2360	3540	2110	3160
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
2410	3630	2130	3210	1880	2830

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
13.1	73.4	12.9	66.6	12.7	60.8
Area, in.^{2}					
236		213		192	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
64800	4210	57300	3700	50600	3230
r_y, in.					
4.22		4.17		4.10	
r_x/r_y					
3.93		3.93		3.95	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W36

F_y = 65 ksi
F_u = 80 ksi

W36×						Shape	W36×					
529^h		487^h		441^h		lb/ft	529^h		487^h		441^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
6070	9130	5570	8370	5060	7600	0	7560	11400	6910	10400	6200	9310
5890	8850	5390	8110	4900	7370	6	7560	11400	6910	10400	6200	9310
5820	8750	5330	8020	4840	7280	7	7560	11400	6910	10400	6200	9310
5750	8640	5260	7910	4780	7180	8	7560	11400	6910	10400	6200	9310
5670	8520	5190	7790	4710	7080	9	7560	11400	6910	10400	6200	9310
5570	8380	5100	7670	4630	6960	10	7560	11400	6910	10400	6200	9310
5470	8230	5010	7530	4540	6830	11	7560	11400	6910	10400	6200	9310
5370	8070	4910	7380	4450	6690	12	7560	11400	6910	10400	6200	9310
5250	7900	4800	7220	4350	6540	13	7510	11300	6850	10300	6130	9210
5130	7720	4690	7050	4250	6390	14	7430	11200	6770	10200	6050	9100
5010	7530	4570	6870	4140	6220	15	7350	11000	6700	10100	5980	8990
4880	7330	4450	6690	4030	6050	16	7270	10900	6620	9950	5900	8870
4740	7130	4320	6500	3910	5880	17	7190	10800	6540	9830	5830	8760
4600	6920	4190	6300	3790	5700	18	7120	10700	6460	9720	5750	8650
4460	6700	4060	6100	3670	5510	19	7040	10600	6390	9600	5680	8530
4310	6480	3930	5900	3540	5330	20	6960	10500	6310	9480	5600	8420
4010	6030	3650	5480	3290	4940	22	6800	10200	6150	9250	5450	8190
3710	5580	3370	5060	3030	4550	24	6640	9980	6000	9020	5300	7970
3410	5120	3090	4640	2770	4160	26	6480	9750	5840	8780	5150	7740
3100	4670	2810	4220	2520	3780	28	6330	9510	5690	8550	5000	7510
2810	4230	2540	3810	2270	3410	30	6170	9270	5530	8320	4850	7280
2530	3800	2280	3420	2030	3050	32	6010	9030	5380	8080	4700	7060
2250	3390	2020	3040	1800	2710	34	5850	8800	5220	7850	4540	6830
2010	3020	1810	2710	1610	2420	36	5700	8560	5070	7620	4390	6600
1800	2710	1620	2440	1440	2170	38	5540	8320	4910	7380	4240	6380
1630	2450	1460	2200	1300	1960	40	5380	8090	4760	7150	4090	6150
1480	2220	1330	1990	1180	1780	42	5220	7850	4600	6920	3940	5920
1350	2020	1210	1820	1080	1620	44	5060	7610	4450	6680	3790	5700
1230	1850	1110	1660	985	1480	46	4910	7370	4290	6450	3600	5410
1130	1700	1020	1530	905	1360	48	4750	7140	4130	6200	3420	5140
1040	1570	936	1410	834	1250	50	4590	6900	3930	5910	3260	4890

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
6070	9130	5570	8370	5060	7610
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4680	7020	4290	6440	3900	5850
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1670	2500	1530	2300	1380	2060
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
1470	2210	1340	2010	1190	1790

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.4	50.9	12.3	47.8	12.1	44.6
Area, in.^{2}					
156		143		130	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
39600	2490	36000	2250	32100	1990
r_y, in.					
4.00		3.96		3.92	
r_x/r_y					
4.00		3.99		4.01	

^{h} Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W36

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W36×						Shape	W36×					
395^h		361^h		330		lb/ft	395^h		361^h		330	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
4510	6790	4130	6200	3770	5670	0	5550	8340	5030	7560	4570	6870
4370	6570	3990	6000	3650	5480	6	5550	8340	5030	7560	4570	6870
4320	6490	3940	5930	3600	5420	7	5550	8340	5030	7560	4570	6870
4260	6400	3890	5850	3550	5340	8	5550	8340	5030	7560	4570	6870
4190	6300	3830	5750	3500	5260	9	5550	8340	5030	7560	4570	6870
4120	6200	3760	5650	3440	5160	10	5550	8340	5030	7560	4570	6870
4040	6080	3690	5550	3370	5060	11	5550	8340	5030	7560	4570	6870
3960	5950	3610	5430	3300	4960	12	5550	8340	5020	7550	4560	6860
3870	5820	3530	5310	3220	4840	13	5470	8230	4950	7440	4500	6760
3780	5680	3440	5170	3140	4720	14	5400	8120	4880	7330	4430	6650
3680	5530	3350	5040	3060	4600	15	5330	8010	4810	7230	4360	6550
3580	5380	3260	4900	2970	4460	16	5250	7890	4740	7120	4290	6450
3470	5220	3160	4750	2880	4330	17	5180	7780	4670	7010	4220	6340
3360	5050	3060	4600	2790	4190	18	5100	7670	4590	6900	4150	6240
3250	4890	2960	4440	2690	4050	19	5030	7560	4520	6800	4080	6140
3140	4720	2850	4290	2600	3900	20	4960	7450	4450	6690	4020	6040
2910	4370	2640	3970	2400	3610	22	4810	7230	4310	6470	3880	5830
2670	4020	2420	3640	2200	3310	24	4660	7010	4170	6260	3740	5620
2440	3670	2210	3320	2010	3020	26	4510	6780	4020	6050	3600	5420
2210	3330	2000	3010	1810	2730	28	4370	6560	3880	5830	3470	5210
1990	2990	1800	2700	1630	2450	30	4220	6340	3740	5620	3330	5010
1780	2670	1600	2410	1450	2180	32	4070	6120	3590	5400	3190	4800
1580	2370	1420	2130	1280	1930	34	3920	5900	3450	5190	3060	4590
1410	2110	1270	1900	1140	1720	36	3780	5670	3310	4970	2920	4390
1260	1900	1140	1710	1030	1540	38	3630	5450	3160	4760	2760	4150
1140	1710	1030	1540	927	1390	40	3480	5230	3000	4510	2570	3870
1030	1550	930	1400	841	1260	42	3310	4970	2810	4230	2410	3620
942	1420	847	1270	766	1150	44	3120	4690	2650	3980	2260	3400
861	1290	775	1160	701	1050	46	2950	4430	2500	3750	2130	3200
791	1190	712	1070	644	968	48	2800	4200	2370	3560	2020	3030
729	1100	656	986	593	892	50	2660	4000	2250	3380	1910	2880

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4510	6790	4130	6200	3770	5670
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3480	5220	3180	4770	2910	4360
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1220	1830	1110	1660	1000	1500
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
1050	1580	950	1430	860	1290

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.0	41.3	11.9	39.4	11.9	37.5
Area, in.^{2}					
116		106		96.9	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
28500	1750	25700	1570	23300	1420
r_y, in.					
3.88		3.85		3.83	
r_x/r_y					
4.05		4.05		4.05	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

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W36

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W36×						Shape	W36×					
302^c		282^c		262^c		lb/ft	302		282		262	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
3400	5120	3120	4690	2870	4310	0	4150	6240	3860	5800	3570	5360
3300	4970	3030	4550	2780	4180	6	4150	6240	3860	5800	3570	5360
3270	4910	3000	4500	2750	4130	7	4150	6240	3860	5800	3570	5360
3230	4850	2960	4450	2710	4080	8	4150	6240	3860	5800	3570	5360
3180	4780	2920	4380	2680	4020	9	4150	6240	3860	5800	3570	5360
3130	4710	2870	4320	2630	3960	10	4150	6240	3860	5800	3570	5360
3080	4630	2820	4240	2590	3890	11	4150	6240	3860	5800	3570	5360
3020	4540	2770	4160	2540	3810	12	4140	6220	3850	5780	3550	5330
2960	4440	2710	4070	2480	3730	13	4080	6130	3780	5690	3490	5240
2880	4330	2650	3980	2430	3650	14	4010	6030	3720	5590	3430	5150
2800	4220	2590	3890	2370	3560	15	3950	5930	3660	5500	3360	5060
2720	4100	2520	3790	2310	3470	16	3880	5830	3590	5400	3300	4970
2640	3970	2450	3680	2240	3370	17	3820	5740	3530	5310	3240	4880
2560	3840	2370	3570	2180	3270	18	3750	5640	3470	5210	3180	4780
2470	3710	2290	3440	2110	3170	19	3690	5540	3410	5120	3120	4690
2380	3580	2210	3320	2040	3070	20	3620	5440	3340	5030	3060	4600
2200	3310	2040	3070	1880	2830	22	3490	5250	3220	4840	2940	4420
2020	3030	1870	2810	1720	2590	24	3360	5050	3090	4650	2820	4240
1840	2760	1700	2560	1560	2350	26	3230	4860	2970	4460	2700	4060
1660	2500	1530	2310	1410	2110	28	3100	4660	2840	4270	2580	3870
1490	2240	1370	2070	1260	1890	30	2970	4470	2720	4080	2460	3690
1320	1990	1220	1830	1110	1670	32	2840	4270	2590	3890	2340	3510
1170	1760	1080	1620	985	1480	34	2710	4080	2460	3700	2210	3330
1050	1570	964	1450	879	1320	36	2580	3880	2310	3470	2030	3050
939	1410	865	1300	789	1190	38	2400	3600	2130	3210	1870	2810
847	1270	781	1170	712	1070	40	2230	3350	1980	2970	1730	2600
768	1160	708	1060	646	971	42	2080	3130	1850	2770	1610	2420
700	1050	645	970	588	884	44	1950	2930	1730	2600	1510	2270
641	963	591	888	538	809	46	1840	2760	1630	2440	1420	2130
588	884	542	815	494	743	48	1730	2610	1530	2300	1330	2000
542	815	500	751	456	685	50	1640	2470	1450	2180	1260	1900

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3460	5210	3230	4850	3000	4520
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2670	4010	2490	3730	2320	3470
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
916	1370	854	1280	806	1210
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
782	1170	723	1090	662	995

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.8	36.3	11.8	35.3	11.6	34.1
Area, in.^{2}					
89.0		82.9		77.2	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
21100	1300	19600	1200	17900	1090
r_y, in.					
3.82		3.80		3.76	
r_x/r_y					
4.03		4.05		4.07	

^{c} Shape is slender for compression with F_y = 65 ksi.

Note: Confirm ASTM A913 material availability before specifying.

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W36

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W36×						Shape	W36×					
247^c		231^c		256^c		lb/ft	247		231		256	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2660	4000	2470	3710	2870	4320	0	3340	5020	3120	4690	3370	5070
2580	3870	2390	3600	2700	4060	6	3340	5020	3120	4690	3370	5070
2550	3830	2370	3560	2650	3980	7	3340	5020	3120	4690	3370	5070
2520	3780	2330	3510	2580	3880	8	3340	5020	3120	4690	3370	5070
2480	3730	2300	3460	2500	3760	9	3340	5020	3120	4690	3310	4980
2440	3670	2260	3400	2410	3620	10	3340	5020	3120	4690	3240	4870
2400	3600	2220	3340	2320	3480	11	3340	5020	3120	4690	3160	4760
2350	3530	2180	3270	2210	3330	12	3320	4980	3100	4650	3090	4640
2300	3460	2130	3200	2110	3170	13	3260	4900	3040	4570	3010	4530
2250	3380	2080	3130	2000	3010	14	3200	4810	2980	4480	2940	4420
2190	3290	2030	3050	1890	2840	15	3140	4720	2930	4400	2860	4310
2130	3210	1970	2970	1780	2670	16	3080	4630	2870	4320	2790	4190
2070	3120	1920	2880	1670	2510	17	3020	4550	2820	4230	2710	4080
2010	3030	1860	2800	1560	2340	18	2970	4460	2760	4150	2640	3970
1950	2930	1800	2710	1450	2180	19	2910	4370	2700	4060	2570	3860
1890	2830	1740	2620	1340	2020	20	2850	4280	2650	3980	2490	3740
1750	2640	1620	2430	1140	1710	22	2730	4110	2540	3810	2340	3520
1610	2410	1490	2250	958	1440	24	2620	3930	2420	3640	2190	3290
1460	2190	1360	2040	817	1230	26	2500	3760	2310	3480	2040	3070
1310	1970	1220	1830	704	1060	28	2380	3580	2200	3310	1840	2760
1170	1760	1080	1630	613	922	30	2270	3410	2090	3140	1670	2510
1030	1550	957	1440	539	810	32	2150	3230	1980	2970	1530	2290
916	1380	848	1270	477	718	34	2000	3010	1820	2730	1410	2120
817	1230	756	1140	426	640	36	1830	2750	1660	2490	1310	1960
733	1100	679	1020	382	575	38	1680	2530	1520	2290	1220	1830
662	994	612	920	345	518	40	1560	2340	1410	2120	1140	1710
600	902	555	835	313	470	42	1450	2180	1310	1960	1070	1610
547	822	506	761	285	429	44	1350	2030	1220	1830	1010	1520
500	752	463	696			46	1270	1910	1140	1720	960	1440
459	691	425	639			48	1200	1800	1070	1610	912	1370
423	636	392	589			50	1130	1700	1010	1520	868	1310

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2820	4240	2650	3990	2930	4410
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2180	3260	2050	3070	2260	3390
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
763	1150	721	1080	934	1400
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
616	926	571	858	444	668

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.6	33.3	11.5	32.7	8.21	26.1
Area, in.^{2}					
72.5		68.2		75.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
16700	1010	15600	940	16800	528
r_y, in.					
3.74		3.71		2.65	
r_x/r_y					
4.06		4.07		5.62	

^{c} Shape is slender for compression with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W36

F_y = 65 ksi
F_u = 80 ksi

W36×						Shape	W36×					
232^c		210^c		194^c		lb/ft	232		210		194	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2520	3790	2260	3390	2030	3050	0	3040	4560	2700	4060	2490	3740
2370	3560	2120	3180	1900	2860	6	3040	4560	2700	4060	2490	3740
2320	3480	2070	3110	1860	2790	7	3040	4560	2700	4060	2490	3740
2260	3400	2020	3030	1810	2720	8	3040	4560	2700	4060	2480	3730
2190	3300	1960	2940	1760	2640	9	2970	4470	2630	3960	2420	3640
2120	3190	1890	2850	1700	2550	10	2900	4360	2570	3860	2360	3540
2050	3080	1830	2750	1640	2460	11	2830	4260	2500	3760	2290	3450
1970	2960	1750	2640	1570	2360	12	2760	4150	2430	3660	2230	3350
1890	2840	1680	2520	1500	2260	13	2690	4040	2370	3560	2170	3260
1790	2690	1600	2410	1430	2150	14	2620	3940	2300	3460	2100	3160
1690	2540	1520	2280	1360	2040	15	2550	3830	2230	3360	2040	3070
1590	2390	1420	2140	1290	1940	16	2480	3720	2170	3260	1980	2970
1490	2240	1330	2000	1210	1820	17	2410	3620	2100	3160	1910	2870
1390	2080	1240	1860	1130	1690	18	2340	3510	2040	3060	1850	2780
1290	1940	1150	1720	1040	1570	19	2270	3410	1970	2960	1790	2680
1190	1790	1060	1590	962	1450	20	2200	3300	1900	2860	1720	2590
1010	1510	889	1340	806	1210	22	2050	3090	1770	2660	1600	2400
846	1270	747	1120	677	1020	24	1910	2870	1640	2460	1440	2170
721	1080	636	956	577	867	26	1740	2610	1440	2170	1270	1910
621	934	549	825	497	748	28	1560	2340	1290	1940	1130	1700
541	814	478	718	433	651	30	1410	2120	1160	1750	1020	1530
476	715	420	631	381	572	32	1290	1930	1060	1590	925	1390
421	633	372	559	337	507	34	1180	1780	970	1460	847	1270
376	565	332	499	301	452	36	1100	1650	895	1350	780	1170
337	507	298	448	270	406	38	1020	1530	831	1250	723	1090
305	458	269	404	244	366	40	954	1430	776	1170	674	1010
276	415	244	366	221	332	42	896	1350	727	1090	630	948
						44	844	1270	684	1030	593	891
						46	799	1200	646	971	559	840
						48	758	1140	612	920	529	795
						50	721	1080	581	874	502	754

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2650	3980	2410	3620	2220	3330
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2040	3060	1860	2790	1710	2570
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
839	1260	792	1190	726	1090
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
396	595	347	522	317	476

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.12	25.1	7.99	24.0	7.93	23.4
Area, in.^{2}					
68.0		61.9		57.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
15000	468	13200	411	12100	375
r_y, in.					
2.62		2.58		2.56	
r_x/r_y					
5.65		5.66		5.70	

^{c} Shape is slender for compression with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

W36

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W36×						Shape	W36×					
182^c		170^c		160^c		lb/ft	182		170^v		160^v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1870	2810	1710	2580	1590	2380	0	2330	3500	2170	3260	2020	3040
1760	2640	1600	2410	1480	2230	6	2330	3500	2170	3260	2020	3040
1720	2580	1570	2360	1450	2180	7	2330	3500	2170	3260	2020	3040
1670	2510	1530	2290	1410	2120	8	2320	3490	2160	3240	2010	3020
1620	2440	1480	2220	1360	2050	9	2260	3400	2100	3160	1950	2940
1570	2350	1430	2150	1320	1980	10	2200	3310	2040	3070	1900	2860
1510	2270	1380	2070	1270	1900	11	2140	3220	1980	2980	1840	2770
1450	2180	1320	1980	1210	1820	12	2080	3130	1930	2900	1790	2690
1390	2080	1260	1900	1160	1740	13	2020	3030	1870	2810	1730	2610
1320	1980	1200	1810	1100	1660	14	1960	2940	1810	2720	1680	2520
1250	1880	1140	1710	1040	1570	15	1900	2850	1750	2630	1620	2440
1190	1780	1080	1620	987	1480	16	1840	2760	1700	2550	1570	2360
1120	1680	1020	1530	928	1400	17	1780	2670	1640	2460	1510	2280
1050	1580	953	1430	870	1310	18	1720	2580	1580	2370	1460	2190
976	1470	891	1340	813	1220	19	1650	2490	1520	2290	1400	2110
899	1350	827	1240	756	1140	20	1590	2400	1460	2200	1350	2030
752	1130	690	1040	634	952	22	1470	2210	1350	2030	1240	1860
632	949	580	872	532	800	24	1310	1970	1180	1780	1070	1610
538	809	494	743	454	682	26	1150	1730	1040	1560	936	1410
464	697	426	640	391	588	28	1020	1540	920	1380	829	1250
404	608	371	558	341	512	30	921	1380	825	1240	742	1120
355	534	326	490	299	450	32	835	1250	746	1120	670	1010
315	473	289	434	265	399	34	763	1150	681	1020	610	917
281	422	258	387	237	356	36	702	1050	625	940	560	841
252	379	231	348	212	319	38	650	976	578	869	516	776
227	342	209	314	192	288	40	604	908	537	807	479	720
206	310	189	285			42	565	849	501	753	447	671
						44	530	797	470	706	418	629
						46	500	751	442	665	393	591
						48	473	710	418	628	371	557
						50	448	674	396	595	351	528

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2090	3140	1950	2930	1830	2750
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1610	2410	1500	2250	1410	2120
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
684	1030	575	864	546	821
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
294	442	272	409	251	377

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.90	23.0	7.84	22.5	7.74	22.1
Area, in.^{2}					
53.6		50.0		47.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
11300	347	10500	320	9760	295
r_y, in.					
2.55		2.53		2.50	
r_x/r_y					
5.69		5.73		5.76	

^{c} Shape is slender for compression with F_y = 65 ksi.

^{v} Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W36–W33

F_y = 65 ksi
F_u = 80 ksi

W36×				W33×		Shape		W36×				W33×	
150^c		135^c		387^h		lb/ft		150^v		135^v		387^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending		ASD	LRFD	ASD	LRFD	ASD	LRFD
1470	2210	1290	1940	4440	6670		0	1880	2830	1650	2480	5060	7610
1370	2070	1200	1810	4290	6440		6	1880	2830	1650	2480	5060	7610
1340	2020	1170	1760	4230	6360		7	1880	2830	1650	2480	5060	7610
1300	1960	1140	1710	4170	6270		8	1870	2800	1620	2440	5060	7610
1260	1900	1100	1650	4100	6170		9	1810	2730	1570	2360	5060	7610
1220	1830	1060	1590	4030	6060		10	1760	2650	1520	2290	5060	7610
1170	1760	1010	1520	3950	5940		11	1710	2570	1480	2220	5060	7610
1120	1680	967	1450	3860	5810		12	1660	2490	1430	2150	5040	7570
1070	1610	920	1380	3770	5670		13	1600	2410	1380	2070	4980	7480
1020	1530	871	1310	3670	5520		14	1550	2330	1330	2000	4910	7380
961	1440	822	1240	3570	5370		15	1500	2250	1280	1930	4850	7290
907	1360	773	1160	3470	5210		16	1440	2170	1230	1860	4780	7190
852	1280	723	1090	3360	5050		17	1390	2090	1190	1780	4720	7090
798	1200	674	1010	3250	4880		18	1340	2010	1140	1710	4650	7000
744	1120	626	941	3130	4710		19	1290	1930	1090	1640	4590	6900
691	1040	578	869	3020	4540		20	1230	1860	1040	1560	4530	6800
583	876	487	733	2780	4180		22	1120	1680	909	1370	4400	6610
490	736	410	616	2550	3830		24	960	1440	780	1170	4270	6420
417	627	349	525	2310	3480		26	838	1260	679	1020	4140	6230
360	541	301	452	2090	3130		28	741	1110	598	899	4010	6030
313	471	262	394	1870	2800		30	662	994	533	801	3890	5840
275	414	230	346	1650	2480		32	597	897	479	720	3760	5650
244	367	204	307	1460	2200		34	542	815	435	653	3630	5450
218	327	182	274	1300	1960		36	497	746	397	597	3500	5260
195	294	163	246	1170	1760		38	457	688	365	548	3370	5070
176	265			1060	1590		40	424	637	337	507	3240	4880
				959	1440		42	395	593	313	471	3120	4680
				874	1310		44	369	555	292	439	2960	4450
				799	1200		46	346	521	274	412	2810	4220
				734	1100		48	326	491	258	387	2670	4010
				676	1020		50	309	464	243	365	2540	3820

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1720	2590	1550	2330	4440	6670
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1330	1990	1200	1800	3420	5130
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
524	788	495	744	1180	1770
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
230	346	194	291	1010	1520

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.65	21.7	7.37	20.9	11.7	42.8
Area, in.^{2}					
44.3		39.9		114	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
9040	270	7800	225	24300	1620
r_y, in.					
2.47		2.38		3.77	
r_x/r_y					
5.79		5.88		3.87	

[c] Shape is slender for compression with F_y = 65 ksi.

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W33

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W33×						Shape		W33×					
354^h		318		291		lb/ft		354^h		318		291	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
4050	6080	3650	5480	3330	5010	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	4610	6920	4120	6190	3760	5660
3910	5870	3520	5290	3210	4830		6	4610	6920	4120	6190	3760	5660
3860	5800	3470	5220	3170	4770		7	4610	6920	4120	6190	3760	5660
3800	5710	3420	5140	3120	4690		8	4610	6920	4120	6190	3760	5660
3740	5620	3360	5060	3070	4610		9	4610	6920	4120	6190	3760	5660
3670	5520	3300	4960	3010	4530		10	4610	6920	4120	6190	3760	5660
3600	5400	3230	4860	2950	4430		11	4610	6920	4120	6190	3760	5660
3520	5280	3160	4750	2880	4330		12	4580	6880	4090	6150	3730	5600
3430	5160	3080	4630	2810	4220		13	4520	6790	4030	6050	3670	5510
3340	5020	3000	4510	2730	4110		14	4460	6700	3970	5960	3610	5430
3250	4880	2920	4380	2650	3990		15	4390	6600	3910	5870	3550	5340
3150	4740	2830	4250	2570	3870		16	4330	6510	3850	5780	3490	5250
3050	4590	2740	4110	2490	3740		17	4270	6420	3790	5690	3430	5160
2950	4430	2640	3970	2400	3610		18	4210	6320	3730	5600	3380	5070
2840	4270	2550	3830	2310	3480		19	4140	6230	3670	5510	3320	4990
2740	4110	2450	3680	2220	3340		20	4080	6140	3610	5420	3260	4900
2520	3790	2250	3390	2040	3070		22	3960	5950	3490	5240	3140	4720
2300	3460	2060	3090	1860	2800		24	3830	5760	3370	5060	3020	4540
2090	3140	1860	2800	1680	2530		26	3710	5570	3240	4880	2910	4370
1880	2820	1670	2510	1510	2270		28	3580	5390	3120	4700	2790	4190
1680	2520	1490	2240	1340	2020		30	3460	5200	3000	4510	2670	4020
1480	2230	1310	1980	1180	1780		32	3340	5010	2880	4330	2550	3840
1310	1970	1160	1750	1050	1570		34	3210	4830	2760	4150	2440	3660
1170	1760	1040	1560	934	1400		36	3090	4640	2640	3970	2320	3490
1050	1580	932	1400	838	1260		38	2960	4450	2520	3790	2150	3240
949	1430	841	1260	756	1140		40	2840	4260	2350	3540	2010	3020
861	1290	763	1150	686	1030		42	2670	4020	2210	3320	1880	2830
784	1180	695	1050	625	939		44	2520	3790	2080	3120	1770	2660
718	1080	636	956	572	859		46	2390	3590	1960	2950	1670	2510
659	991	584	878	525	789		48	2270	3400	1860	2800	1580	2370
607	913	538	809	484	727		50	2160	3240	1770	2660	1500	2260

Properties												
Available Strength in Tensile Yielding, kips							Limiting Unbraced Lengths, ft					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		L_p	L_r	L_p	L_r	L_p	L_r
4050	6080	3650	5480	3330	5010		11.6	40.3	11.5	38.0	11.4	36.1
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips							Area, in.^{2}					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		104		93.7		85.6	
3120	4680	2810	4220	2570	3850		Moment of Inertia, in.^{4}					
Available Strength in Shear, kips							I_x	I_y	I_x	I_y	I_x	I_y
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$		22000	1460	19500	1290	17700	1160
1070	1610	952	1430	869	1300		r_y, in.					
Available Strength in Flexure about Y-Y Axis, kip-ft							3.74		3.71		3.68	
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$		r_x/r_y					
915	1370	811	1220	733	1100		3.88		3.91		3.91	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

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W33

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W33×						Shape	W33×					
263^c		241^c		221^c		lb/ft	263		241		221	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2950	4440	2680	4030	2420	3640	0	3370	5070	3050	4580	2780	4180
2860	4300	2590	3900	2340	3520	6	3370	5070	3050	4580	2780	4180
2830	4250	2560	3850	2310	3470	7	3370	5070	3050	4580	2780	4180
2790	4190	2530	3800	2280	3420	8	3370	5070	3050	4580	2780	4180
2740	4130	2490	3740	2240	3370	9	3370	5070	3050	4580	2780	4180
2700	4060	2450	3680	2200	3310	10	3370	5070	3050	4580	2780	4180
2650	3980	2400	3610	2160	3250	11	3370	5070	3050	4580	2780	4180
2590	3900	2350	3530	2120	3180	12	3340	5010	3010	4520	2740	4110
2530	3810	2300	3450	2070	3110	13	3280	4930	2950	4440	2690	4040
2470	3710	2240	3370	2020	3030	14	3230	4850	2900	4360	2640	3960
2390	3600	2180	3280	1960	2950	15	3170	4770	2850	4280	2590	3890
2320	3490	2120	3180	1910	2860	16	3120	4680	2800	4200	2540	3810
2240	3370	2050	3080	1850	2780	17	3060	4600	2740	4130	2490	3740
2160	3250	1970	2970	1790	2690	18	3000	4520	2690	4050	2440	3660
2080	3130	1900	2850	1730	2600	19	2950	4430	2640	3970	2390	3580
2000	3010	1820	2740	1660	2500	20	2890	4350	2590	3890	2330	3510
1840	2760	1670	2510	1520	2280	22	2780	4180	2480	3730	2230	3360
1670	2510	1520	2280	1380	2070	24	2670	4020	2380	3570	2130	3210
1510	2270	1370	2050	1240	1860	26	2560	3850	2270	3410	2030	3060
1350	2030	1220	1830	1110	1660	28	2450	3680	2170	3260	1930	2910
1200	1810	1080	1620	976	1470	30	2340	3520	2060	3100	1830	2760
1060	1590	950	1430	858	1290	32	2230	3350	1960	2940	1730	2610
936	1410	841	1260	760	1140	34	2120	3180	1830	2740	1580	2380
835	1260	750	1130	678	1020	36	1970	2950	1680	2520	1450	2180
749	1130	674	1010	608	914	38	1820	2730	1550	2330	1330	2010
676	1020	608	914	549	825	40	1690	2540	1440	2160	1240	1860
614	922	551	829	498	748	42	1580	2380	1340	2010	1150	1730
559	840	502	755	454	682	44	1490	2230	1260	1890	1080	1620
511	769	460	691	415	624	46	1400	2100	1180	1780	1010	1520
470	706	422	634	381	573	48	1320	1990	1110	1680	953	1430
433	651	389	585	351	528	50	1260	1890	1060	1590	901	1350

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3010	4530	2770	4160	2540	3820
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2320	3480	2130	3200	1960	2940
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
780	1170	738	1110	683	1020
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
655	985	590	887	532	800

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.3	34.6	11.2	33.3	11.1	32.2
Area, in.^{2}					
77.4		71.1		65.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
15900	1040	14200	933	12900	840
r_y, in.					
3.66		3.62		3.59	
r_x/r_y					
3.91		3.90		3.93	

^{c} Shape is slender for compression with F_y = 65 ksi.

Note: Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W33

F_y = 65 ksi
F_u = 80 ksi

W33× 201^c P_n/Ω_c ASD	W33× 201^c $\phi_c P_n$ LRFD	W33× 169^c P_n/Ω_c ASD	W33× 169^c $\phi_c P_n$ LRFD	W33× 152^c P_n/Ω_c ASD	W33× 152^c $\phi_c P_n$ LRFD	Shape lb/ft — Design: Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	W33× 201 M_{nx}/Ω_b ASD	W33× 201 $\phi_b M_{nx}$ LRFD	W33× 169 M_{nx}/Ω_b ASD	W33× 169 $\phi_b M_{nx}$ LRFD	W33× 152 M_{nx}/Ω_b ASD	W33× 152 $\phi_b M_{nx}$ LRFD
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
2140	3220	1750	2620	1550	2330	0	2510	3770	2040	3070	1810	2730
2070	3110	1630	2450	1450	2180	6	2510	3770	2040	3070	1810	2730
2050	3080	1590	2390	1410	2120	7	2510	3770	2040	3070	1810	2730
2020	3030	1550	2330	1370	2060	8	2510	3770	2030	3050	1800	2700
1990	2980	1500	2250	1330	2000	9	2510	3770	1970	2970	1750	2620
1950	2930	1450	2170	1280	1920	10	2510	3770	1920	2890	1700	2550
1910	2870	1390	2090	1230	1850	11	2510	3770	1870	2800	1650	2480
1870	2810	1330	2000	1180	1770	12	2460	3700	1810	2720	1600	2400
1830	2750	1270	1910	1120	1690	13	2410	3630	1760	2640	1550	2330
1780	2680	1210	1810	1070	1600	14	2370	3560	1710	2560	1500	2250
1730	2600	1140	1720	1010	1520	15	2320	3490	1650	2480	1450	2180
1680	2530	1080	1620	950	1430	16	2270	3420	1600	2400	1400	2100
1630	2450	1010	1520	892	1340	17	2230	3350	1550	2320	1350	2030
1580	2370	948	1420	834	1250	18	2180	3270	1490	2240	1300	1950
1520	2290	874	1310	777	1170	19	2130	3200	1440	2160	1250	1880
1470	2210	802	1210	712	1070	20	2080	3130	1390	2080	1200	1800
1360	2040	667	1000	591	888	22	1990	2990	1280	1920	1100	1650
1230	1860	561	843	496	746	24	1900	2850	1130	1700	949	1430
1110	1670	478	718	423	636	26	1800	2710	997	1500	834	1250
986	1480	412	619	365	548	28	1710	2570	890	1340	741	1110
869	1310	359	539	318	478	30	1610	2420	803	1210	666	1000
763	1150	315	474	279	420	32	1490	2240	730	1100	604	908
676	1020	279	420	247	372	34	1350	2030	670	1010	552	830
603	907	249	375	221	332	36	1240	1860	618	929	508	763
541	814	224	336	198	298	38	1140	1710	574	862	470	707
489	734	202	303	179	269	40	1050	1580	535	805	438	658
443	666					42	976	1470	502	754	409	615
404	607					44	911	1370	472	710	384	577
369	555					46	854	1280	446	670	362	544
339	510					48	804	1210	422	635	342	514
313	470					50	759	1140	401	603	324	488

Properties

Available Strength in Tensile Yielding, kips

201 P_n/Ω_t	201 $\phi_t P_n$	169 P_n/Ω_t	169 $\phi_t P_n$	152 P_n/Ω_t	152 $\phi_t P_n$
2300	3460	1930	2900	1750	2630

Available Strength in Tensile Rupture (A_e = 0.75A_g), kips

201 P_n/Ω_t	201 $\phi_t P_n$	169 P_n/Ω_t	169 $\phi_t P_n$	152 P_n/Ω_t	152 $\phi_t P_n$
1770	2660	1490	2230	1350	2020

Available Strength in Shear, kips

201 V_n/Ω_v	201 $\phi_v V_n$	169 V_n/Ω_v	169 $\phi_v V_n$	152 V_n/Ω_v	152 $\phi_v V_n$
626	940	589	883	553	830

Available Strength in Flexure about Y-Y Axis, kip-ft

201 M_{ny}/Ω_b	201 $\phi_b M_{ny}$	169 M_{ny}/Ω_b	169 $\phi_b M_{ny}$	152 M_{ny}/Ω_b	152 $\phi_b M_{ny}$
477	717	274	411	240	360

Limiting Unbraced Lengths, ft

201 L_p	201 L_r	169 L_p	169 L_r	152 L_p	152 L_r
11.0	31.2	7.74	22.6	7.65	21.9

	201	169	152
Area, in.^{2}	59.1	49.5	44.9
r_y, in.	3.56	2.50	2.47
r_x/r_y	3.93	5.48	5.47

Moment of Inertia, in.^{4}

201 I_x	201 I_y	169 I_x	169 I_y	152 I_x	152 I_y
11600	749	9290	310	8160	273

^c Shape is slender for compression with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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W33

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W33×						Shape	W33×					
141^c		130^c		118^c		lb/ft	141^v		130^v		118^v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1410	2120	1280	1920	1130	1690	0	1670	2510	1510	2280	1350	2020
1310	1970	1190	1780	1040	1570	6	1670	2510	1510	2280	1350	2020
1280	1920	1150	1740	1020	1530	7	1670	2510	1510	2280	1350	2020
1240	1860	1120	1680	983	1480	8	1650	2470	1490	2240	1310	1970
1200	1800	1080	1630	948	1430	9	1600	2400	1440	2170	1270	1910
1160	1740	1040	1570	911	1370	10	1550	2330	1400	2100	1230	1850
1110	1670	998	1500	871	1310	11	1500	2260	1360	2040	1190	1790
1060	1590	953	1430	829	1250	12	1460	2190	1310	1970	1150	1730
1010	1520	906	1360	786	1180	13	1410	2120	1270	1910	1110	1670
957	1440	858	1290	743	1120	14	1360	2050	1220	1840	1070	1610
904	1360	809	1220	698	1050	15	1320	1980	1180	1770	1030	1540
850	1280	760	1140	654	983	16	1270	1910	1140	1710	987	1480
797	1200	711	1070	610	916	17	1220	1840	1090	1640	946	1420
744	1120	663	996	566	851	18	1180	1770	1050	1570	906	1360
692	1040	615	925	523	787	19	1130	1700	1000	1510	865	1300
639	961	568	854	481	723	20	1080	1630	959	1440	824	1240
528	794	472	709	403	605	22	968	1460	837	1260	700	1050
444	667	396	596	338	509	24	836	1260	721	1080	601	903
378	569	338	508	288	433	26	732	1100	630	946	524	787
326	490	291	438	249	374	28	649	976	557	837	462	694
284	427	254	381	217	326	30	582	875	498	749	412	619
250	375	223	335	190	286	32	527	792	450	676	371	558
221	333	198	297	169	253	34	480	722	409	615	337	506
197	297	176	265	150	226	36	441	663	375	564	308	463
177	266	158	238	135	203	38	408	613	346	520	283	426
160	240					40	379	569	321	482	262	394
						42	353	531	299	449	244	366
						44	331	498	280	420	228	342
						46	312	468	263	395	213	321
						48	294	442	248	372	201	302
						50	279	419	234	352	190	285

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1620	2430	1490	2240	1350	2030
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1250	1870	1150	1720	1040	1560
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
470	707	448	674	416	626
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
217	326	193	290	166	250

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.53	21.4	7.40	20.8	7.19	20.2
Area, in.^{2}					
41.5		38.3		34.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
7450	246	6710	218	5900	187
r_y, in.					
2.43		2.39		2.32	
r_x/r_y					
5.51		5.52		5.60	

[c] Shape is slender for compression with F_y = 65 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W30

F_y = 65 ksi
F_u = 80 ksi

W30×						Shape	W30×					
391^h		357^h		326^h		lb/ft	391^h		357^h		326^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
4480	6730	4090	6140	3730	5610	0	4700	7070	4280	6440	3860	5800
4320	6490	3940	5920	3590	5400	6	4700	7070	4280	6440	3860	5800
4260	6400	3890	5840	3540	5330	7	4700	7070	4280	6440	3860	5800
4190	6300	3830	5750	3490	5240	8	4700	7070	4280	6440	3860	5800
4120	6200	3760	5650	3430	5150	9	4700	7070	4280	6440	3860	5800
4040	6080	3690	5540	3360	5050	10	4700	7070	4280	6440	3860	5800
3960	5950	3610	5420	3280	4940	11	4700	7070	4280	6440	3860	5800
3870	5810	3520	5290	3210	4820	12	4670	7020	4240	6380	3820	5740
3770	5670	3430	5160	3120	4690	13	4620	6940	4190	6300	3770	5660
3670	5510	3340	5020	3030	4560	14	4560	6860	4140	6220	3720	5580
3560	5350	3240	4870	2940	4420	15	4510	6780	4090	6140	3660	5510
3450	5190	3140	4720	2850	4280	16	4460	6700	4030	6060	3610	5430
3340	5020	3030	4560	2750	4130	17	4400	6620	3980	5980	3560	5350
3220	4840	2920	4400	2650	3980	18	4350	6540	3930	5900	3510	5280
3100	4660	2810	4230	2550	3830	19	4300	6460	3870	5820	3460	5200
2980	4480	2700	4060	2450	3680	20	4250	6380	3820	5740	3410	5130
2740	4110	2480	3730	2240	3360	22	4140	6220	3720	5590	3310	4970
2490	3750	2250	3390	2030	3050	24	4030	6060	3610	5430	3210	4820
2250	3380	2030	3060	1830	2750	26	3930	5900	3500	5270	3100	4670
2020	3030	1820	2730	1630	2450	28	3820	5740	3400	5110	3000	4510
1790	2700	1610	2420	1440	2170	30	3710	5580	3290	4950	2900	4360
1580	2370	1420	2130	1270	1900	32	3610	5420	3190	4790	2800	4210
1400	2100	1260	1890	1120	1690	34	3500	5260	3080	4630	2700	4060
1250	1880	1120	1680	1000	1500	36	3400	5100	2980	4480	2600	3900
1120	1680	1010	1510	898	1350	38	3290	4940	2870	4320	2490	3750
1010	1520	908	1360	811	1220	40	3180	4790	2770	4160	2390	3600
917	1380	823	1240	735	1110	42	3080	4630	2660	4000	2270	3400
835	1260	750	1130	670	1010	44	2970	4470	2540	3820	2140	3220
764	1150	686	1030	613	921	46	2870	4310	2410	3630	2030	3050
702	1050	630	947	563	846	48	2740	4110	2300	3450	1930	2900
647	972	581	873	519	780	50	2610	3930	2190	3290	1840	2770

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4480	6730	4090	6140	3730	5610
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3450	5180	3150	4730	2880	4320
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1170	1760	1060	1590	960	1440
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
1010	1510	905	1360	817	1230

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.4	46.5	11.3	43.4	11.2	40.6
Area, in.^{2}					
115		105		95.9	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
20700	1550	18700	1390	16800	1240
r_y, in.					
3.67		3.64		3.60	
r_x/r_y					
3.65		3.65		3.67	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W30

F_y = 65 ksi
F_u = 80 ksi

W30×						Shape		W30×					
292		261		235^c		lb/ft		292		261		235	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
3350	5030	3000	4500	2680	4030	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	3440	5170	3060	4600	2750	4130
3220	4840	2880	4330	2590	3890		6	3440	5170	3060	4600	2750	4130
3180	4770	2840	4270	2550	3840		7	3440	5170	3060	4600	2750	4130
3130	4700	2790	4200	2510	3780		8	3440	5170	3060	4600	2750	4130
3070	4610	2740	4120	2470	3710		9	3440	5170	3060	4600	2750	4130
3010	4520	2690	4040	2410	3630		10	3440	5170	3060	4600	2750	4130
2940	4420	2620	3940	2360	3540		11	3440	5170	3060	4590	2740	4120
2870	4310	2560	3850	2300	3450		12	3390	5100	3010	4520	2700	4050
2790	4200	2490	3740	2240	3360		13	3340	5030	2960	4450	2650	3980
2720	4080	2420	3630	2170	3260		14	3290	4950	2910	4380	2600	3910
2630	3960	2340	3520	2100	3160		15	3250	4880	2860	4300	2560	3850
2550	3830	2260	3400	2030	3050		16	3200	4800	2820	4230	2510	3780
2460	3690	2180	3280	1960	2940		17	3150	4730	2770	4160	2470	3710
2370	3560	2100	3160	1880	2830		18	3100	4660	2720	4090	2420	3640
2280	3420	2020	3030	1810	2710		19	3050	4580	2670	4020	2380	3570
2180	3280	1930	2900	1730	2600		20	3000	4510	2620	3950	2330	3500
2000	3000	1760	2650	1580	2370		22	2900	4360	2530	3800	2240	3370
1810	2720	1590	2390	1420	2140		24	2800	4210	2430	3660	2150	3230
1630	2440	1430	2140	1270	1910		26	2700	4060	2340	3510	2060	3090
1450	2180	1270	1900	1130	1700		28	2600	3910	2240	3370	1970	2960
1280	1920	1110	1670	990	1490		30	2510	3770	2150	3230	1870	2820
1120	1690	978	1470	870	1310		32	2410	3620	2050	3080	1780	2680
995	1500	866	1300	771	1160		34	2310	3470	1960	2940	1690	2540
888	1330	773	1160	688	1030		36	2210	3320	1850	2780	1560	2340
797	1200	694	1040	617	928		38	2110	3170	1720	2580	1450	2170
719	1080	626	941	557	837		40	1980	2970	1610	2410	1350	2030
652	980	568	853	505	759		42	1860	2790	1510	2270	1260	1900
594	893	517	778	460	692		44	1750	2640	1420	2130	1190	1790
544	817	473	711	421	633		46	1660	2500	1340	2020	1120	1690
499	751	435	653	387	581		48	1580	2370	1270	1910	1060	1600
460	692	401	602	356	536		50	1500	2260	1210	1820	1010	1520

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3350	5030	3000	4500	2700	4050
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2580	3870	2310	3470	2080	3120
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
849	1270	764	1150	675	1010
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
723	1090	636	956	568	853

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.1	38.0	10.9	35.5	10.9	33.9
Area, in.^{2}					
86.0		77.0		69.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
14900	1100	13100	959	11700	855
r_y, in.					
3.58		3.53		3.51	
r_x/r_y					
3.69		3.71		3.70	

[c] Shape is slender for compression with F_y = 65 ksi.

Note: Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W30

F_y = 65 ksi
F_u = 80 ksi

W30×						Shape	W30×					
211^c		191^c		173^c		lb/ft	211		191		173	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2370	3570	2090	3150	1860	2800	0	2440	3660	2190	3290	1970	2960
2290	3440	2020	3030	1790	2700	6	2440	3660	2190	3290	1970	2960
2260	3400	1990	2990	1770	2660	7	2440	3660	2190	3290	1970	2960
2230	3350	1960	2950	1740	2620	8	2440	3660	2190	3290	1970	2960
2190	3290	1930	2900	1710	2570	9	2440	3660	2190	3290	1970	2960
2150	3230	1890	2840	1680	2520	10	2440	3660	2190	3290	1970	2960
2100	3160	1850	2780	1640	2470	11	2430	3650	2180	3270	1950	2940
2060	3090	1810	2720	1610	2410	12	2380	3580	2140	3210	1920	2880
2010	3010	1760	2650	1560	2350	13	2340	3520	2100	3150	1880	2820
1950	2920	1720	2580	1520	2290	14	2300	3450	2060	3090	1840	2760
1880	2830	1670	2510	1480	2220	15	2260	3390	2010	3030	1800	2710
1820	2730	1620	2430	1430	2150	16	2210	3330	1970	2970	1760	2650
1750	2630	1560	2350	1380	2080	17	2170	3260	1930	2910	1720	2590
1680	2530	1510	2270	1330	2000	18	2130	3200	1890	2840	1690	2540
1620	2430	1450	2170	1280	1930	19	2080	3130	1850	2780	1650	2480
1550	2330	1380	2080	1230	1850	20	2040	3070	1810	2720	1610	2420
1410	2120	1260	1890	1120	1690	22	1950	2940	1730	2600	1530	2310
1270	1910	1130	1700	1010	1520	24	1870	2810	1650	2480	1460	2190
1130	1710	1010	1520	898	1350	26	1780	2680	1570	2350	1380	2080
1000	1510	891	1340	792	1190	28	1700	2550	1480	2230	1310	1960
880	1320	779	1170	690	1040	30	1610	2420	1400	2110	1230	1850
773	1160	685	1030	607	912	32	1520	2290	1300	1950	1110	1670
685	1030	606	911	538	808	34	1400	2100	1180	1770	1010	1510
611	919	541	813	479	721	36	1290	1940	1080	1630	922	1390
549	824	485	730	430	647	38	1190	1790	1000	1500	850	1280
495	744	438	659	388	584	40	1110	1670	928	1400	787	1180
449	675	397	597	352	529	42	1040	1560	866	1300	733	1100
409	615	362	544	321	482	44	973	1460	812	1220	685	1030
374	563	331	498	294	441	46	917	1380	763	1150	643	967
344	517	304	457	270	405	48	866	1300	720	1080	606	911
317	476	280	421	249	374	50	822	1230	682	1030	573	861

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2420	3640	2180	3280	1980	2980
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1870	2800	1680	2520	1530	2290
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
623	934	567	850	518	777
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
503	756	448	673	399	600

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.8	32.3	10.7	31.0	10.6	30.0
Area, in.^{2}					
62.3		56.1		50.9	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
10300	757	9200	673	8230	598
r_y, in.					
3.49		3.46		3.42	
r_x/r_y					
3.70		3.70		3.71	

^{c} Shape is slender for compression with F_y = 65 ksi.

Note: Confirm ASTM A913 material availability before specifying.

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W30

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

Shape	W30× 148^c^		132^c^		124^c^		Shape lb/ft	W30× 148		132		124	
	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips						Design	Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	1570	2360	1370	2060	1270	1900	0	1620	2440	1420	2130	1320	1990
	1450	2170	1260	1890	1160	1750	6	1620	2440	1420	2130	1320	1990
	1400	2110	1220	1840	1130	1700	7	1620	2440	1420	2130	1320	1980
	1360	2040	1180	1770	1090	1640	8	1580	2370	1370	2070	1280	1920
	1310	1960	1130	1710	1050	1570	9	1530	2310	1330	2000	1240	1860
	1250	1880	1090	1630	1000	1510	10	1490	2240	1290	1940	1200	1800
	1190	1790	1040	1560	953	1430	11	1440	2170	1250	1880	1160	1740
	1130	1700	982	1480	904	1360	12	1400	2100	1210	1810	1120	1680
	1070	1610	927	1390	852	1280	13	1350	2030	1170	1750	1080	1620
	1010	1510	871	1310	800	1200	14	1310	1960	1120	1690	1040	1560
	938	1410	815	1220	748	1120	15	1260	1900	1080	1630	999	1500
	865	1300	756	1140	696	1050	16	1220	1830	1040	1560	959	1440
	793	1190	691	1040	641	964	17	1170	1760	998	1500	919	1380
	723	1090	629	945	582	875	18	1130	1690	956	1440	879	1320
	655	985	568	854	525	789	19	1080	1620	914	1370	839	1260
	591	889	513	770	474	712	20	1040	1560	872	1310	794	1190
	489	735	424	637	391	588	22	919	1380	751	1130	677	1020
	411	617	356	535	329	494	24	805	1210	654	983	588	884
	350	526	303	456	280	421	26	714	1070	578	868	518	779
	302	454	262	393	242	363	28	641	963	516	776	462	695
	263	395	228	342	211	316	30	581	874	466	701	417	626
	231	347	200	301	185	278	32	531	799	425	638	379	569
	205	308	177	267	164	246	34	489	736	390	586	347	522
	183	274	158	238	146	220	36	454	682	360	541	320	481
	164	246					38	423	635	335	503	297	446
							40	396	595	312	469	277	416
							42	372	559	293	440	259	390
							44	351	528	276	415	244	367
							46	332	500	261	392	230	346
							48	316	474	247	371	218	328
							50	300	452	235	353	207	311

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1700	2550	1510	2270	1420	2140
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1310	1960	1160	1750	1100	1640
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
519	778	484	727	459	689
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
221	332	189	285	175	263

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.06	21.0	6.97	20.2	6.91	19.8
Area, in.^{2}					
43.6		38.8		36.5	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
6680	227	5770	196	5360	181
r_y, in.					
2.28		2.25		2.23	
r_x/r_y					
5.44		5.42		5.43	

[c] Shape is slender for compression with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W30

F_y = 65 ksi
F_u = 80 ksi

W30×						Shape	W30×					
116^c		108^c		99^c		lb/ft	116^v		108^v		99^v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1170	1760	1070	1600	954	1430	0	1230	1840	1120	1690	1010	1520
1070	1610	974	1460	869	1310	6	1230	1840	1120	1690	1010	1520
1040	1560	943	1420	840	1260	7	1220	1830	1110	1670	995	1500
1000	1510	908	1360	808	1210	8	1180	1770	1070	1610	962	1450
961	1450	871	1310	773	1160	9	1140	1720	1040	1560	928	1390
918	1380	830	1250	736	1110	10	1100	1660	1000	1510	894	1340
873	1310	788	1180	697	1050	11	1070	1600	966	1450	860	1290
826	1240	745	1120	657	988	12	1030	1540	930	1400	826	1240
778	1170	700	1050	616	926	13	990	1490	894	1340	793	1190
729	1100	655	984	575	864	14	952	1430	857	1290	759	1140
680	1020	609	916	533	802	15	914	1370	821	1230	725	1090
631	948	564	848	493	740	16	876	1320	785	1180	691	1040
582	875	520	781	452	680	17	838	1260	749	1130	658	988
528	794	472	710	412	619	18	799	1200	713	1070	624	938
474	713	424	637	370	556	19	761	1140	675	1010	575	865
428	643	382	575	334	502	20	707	1060	617	927	525	790
354	532	316	475	276	415	22	602	904	524	788	445	669
297	447	266	399	232	348	24	521	784	453	681	384	577
253	381	226	340	197	297	26	458	689	397	597	336	504
218	328	195	293	170	256	28	408	613	353	530	297	447
190	286	170	255	148	223	30	367	552	317	476	266	400
167	251	149	224	130	196	32	333	501	287	431	241	362
148	223	132	199	115	174	34	305	458	262	393	219	329
132	199					36	281	422	241	362	201	302
						38	260	391	222	334	186	279
						40	242	364	207	311	172	259
						42	226	340	193	290	161	241
						44	213	320	181	272	150	226
						46	201	301	171	257	142	213
						48	190	285	161	242	134	201
						50	180	271	153	230	126	190

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1330	2000	1230	1850	1130	1700
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1030	1540	951	1430	870	1310
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
396	595	379	570	361	542
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
160	240	142	214	125	188

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
6.78	19.4	6.66	18.9	6.51	18.4
Area, in.^{2}					
34.2		31.7		29.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
4930	164	4470	146	3990	128
r_y, in.					
2.19		2.15		2.10	
r_x/r_y					
5.48		5.53		5.57	

^{c} Shape is slender for compression with F_y = 65 ksi.

^{v} Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W30–W27

F_y = 65 ksi
F_u = 80 ksi

Shape	W30×		W27×				Shape	W30×		W27×			
lb/ft	90^c		539^h		368^h		lb/ft	$90^{f,v}$		539^h		368^h	
	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips						Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
	838	1260	6190	9300	4240	6380	0	904	1360	6130	9210	4020	6050
	762	1140	5960	8960	4070	6120	6	904	1360	6130	9210	4020	6050
	736	1110	5880	8840	4010	6030	7	902	1350	6130	9210	4020	6050
	708	1060	5790	8710	3950	5930	8	870	1310	6130	9210	4020	6050
	677	1020	5690	8560	3870	5820	9	839	1260	6130	9210	4020	6050
	644	968	5580	8390	3790	5700	10	808	1210	6130	9210	4020	6050
	609	916	5470	8210	3700	5560	11	777	1170	6130	9210	4010	6030
	574	863	5340	8020	3610	5420	12	745	1120	6100	9170	3970	5970
	538	808	5200	7820	3500	5270	13	714	1070	6050	9100	3930	5900
	501	753	5060	7600	3400	5110	14	683	1030	6010	9030	3880	5840
	465	698	4910	7380	3290	4940	15	652	980	5960	8970	3840	5770
	429	644	4760	7150	3180	4770	16	621	933	5920	8900	3800	5710
	393	591	4600	6910	3060	4600	17	589	886	5880	8830	3760	5650
	359	539	4440	6670	2940	4420	18	558	839	5830	8760	3710	5580
	328	493	4270	6420	2820	4240	19	507	762	5790	8700	3670	5520
	300	451	4100	6170	2700	4060	20	462	695	5740	8630	3630	5450
	248	372	3760	5660	2450	3690	22	390	587	5650	8490	3540	5330
	208	313	3420	5150	2210	3330	24	335	504	5560	8360	3460	5200
	177	267	3090	4640	1980	2970	26	293	440	5470	8220	3370	5070
	153	230	2770	4160	1750	2630	28	259	389	5380	8090	3290	4940
	133	200	2450	3690	1530	2300	30	231	347	5290	7950	3200	4810
	117	176	2160	3250	1350	2020	32	208	313	5200	7820	3120	4680
	104	156	1910	2870	1190	1790	34	189	284	5110	7690	3030	4560
			1710	2560	1060	1600	36	173	260	5020	7550	2950	4430
			1530	2300	954	1430	38	159	240	4930	7420	2860	4300
			1380	2080	861	1290	40	148	222	4840	7280	2770	4170
			1250	1880	781	1170	42	137	206	4750	7150	2690	4040
			1140	1720	712	1070	44	128	193	4670	7010	2600	3910
			1040	1570	651	979	46	121	181	4580	6880	2520	3790
			960	1440	598	899	48	114	171	4490	6740	2430	3660
			884	1330	551	828	50	107	161	4400	6610	2330	3510

Properties

W30×90		W27×539		W27×368	
Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1020	1540	6190	9300	4240	6380
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
789	1180	4770	7160	3270	4910
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
302	454	1660	2500	1090	1640
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
110	166	1420	2130	905	1360

W30×90		W27×539		W27×368	
Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
6.91	18.1	11.3	68.6	10.8	48.6
Area, in.^{2}					
26.3		159		109	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
3610	115	25600	2110	16200	1310
r_y, in.					
2.09		3.65		3.48	
r_x/r_y					
5.60		3.48		3.51	

[c] Shape is slender for compression with F_y = 65 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 65 ksi.
[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.
Note: Heavy line indicates L_c/r equal to or greater than 200.

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Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W27

F_y = 65 ksi
F_u = 80 ksi

W27×						Shape	W27×					
336^h		307^h		281		lb/ft	336^h		307^h		281	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
3860	5800	3510	5280	3230	4860	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending 0	3670	5510	3340	5020	3040	4560
3700	5570	3370	5060	3100	4660	6	3670	5510	3340	5020	3040	4560
3650	5490	3310	4980	3050	4590	7	3670	5510	3340	5020	3040	4560
3590	5390	3260	4890	3000	4500	8	3670	5510	3340	5020	3040	4560
3520	5290	3190	4800	2940	4410	9	3670	5510	3340	5020	3040	4560
3440	5170	3120	4690	2870	4320	10	3670	5510	3340	5020	3040	4560
3360	5050	3040	4580	2800	4210	11	3650	5490	3320	4990	3020	4530
3270	4920	2960	4450	2720	4100	12	3610	5420	3280	4930	2970	4470
3180	4780	2880	4320	2640	3980	13	3570	5360	3240	4870	2930	4410
3080	4630	2790	4190	2560	3850	14	3520	5300	3190	4800	2890	4350
2980	4480	2690	4050	2470	3720	15	3480	5230	3150	4740	2850	4280
2880	4320	2600	3900	2380	3580	16	3440	5170	3110	4670	2810	4220
2770	4160	2500	3760	2290	3450	17	3400	5110	3070	4610	2770	4160
2660	4000	2400	3600	2200	3300	18	3350	5040	3020	4550	2730	4100
2550	3830	2300	3450	2100	3160	19	3310	4980	2980	4480	2680	4040
2440	3660	2190	3300	2010	3020	20	3270	4910	2940	4420	2640	3970
2210	3330	1990	2980	1820	2730	22	3180	4790	2850	4290	2560	3850
1990	2990	1780	2680	1630	2450	24	3100	4660	2770	4160	2480	3730
1770	2670	1580	2380	1450	2170	26	3010	4530	2680	4030	2400	3600
1570	2360	1400	2100	1270	1910	28	2930	4400	2600	3910	2310	3480
1370	2060	1220	1830	1110	1660	30	2840	4270	2510	3780	2230	3350
1200	1810	1070	1610	973	1460	32	2760	4150	2430	3650	2150	3230
1070	1600	947	1420	862	1300	34	2670	4020	2340	3520	2070	3100
951	1430	845	1270	769	1160	36	2590	3890	2260	3390	1980	2980
853	1280	758	1140	690	1040	38	2500	3760	2170	3270	1900	2860
770	1160	684	1030	623	936	40	2420	3630	2090	3140	1810	2710
699	1050	621	933	565	849	42	2330	3510	2000	3000	1700	2560
637	957	565	850	515	774	44	2250	3380	1890	2850	1610	2420
582	875	517	778	471	708	46	2150	3230	1800	2700	1530	2300
535	804	475	714	433	650	48	2050	3080	1710	2580	1460	2190
493	741	438	658	399	599	50	1960	2940	1640	2460	1390	2090

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3860	5800	3510	5280	3230	4860
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2980	4460	2710	4060	2490	3740
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
983	1470	893	1340	808	1210
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
817	1230	736	1110	668	1000

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.7	45.0	10.6	41.7	10.5	39.3
Area, in.^{2}					
99.2		90.2		83.1	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
14600	1180	13100	1050	11900	953
r_y, in.					
3.45		3.41		3.39	
r_x/r_y					
3.51		3.52		3.54	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W27

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W27×						Shape		W27×					
258		235		217		lb/ft		258		235		217	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
2960	4450	2700	4060	2490	3740	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	2760	4150	2500	3760	2310	3470
2840	4260	2580	3880	2380	3570		6	2760	4150	2500	3760	2310	3470
2790	4200	2540	3820	2340	3520		7	2760	4150	2500	3760	2310	3470
2740	4120	2500	3750	2300	3450		8	2760	4150	2500	3760	2310	3470
2680	4040	2440	3670	2250	3380		9	2760	4150	2500	3760	2310	3470
2620	3940	2390	3590	2200	3300		10	2760	4150	2500	3760	2310	3470
2560	3840	2330	3500	2140	3220		11	2740	4120	2480	3720	2280	3430
2490	3740	2260	3400	2080	3130		12	2700	4060	2440	3660	2240	3370
2410	3630	2190	3300	2020	3030		13	2660	4000	2400	3600	2200	3310
2340	3510	2120	3190	1950	2930		14	2620	3940	2360	3550	2160	3250
2250	3390	2050	3080	1880	2830		15	2580	3880	2320	3490	2130	3200
2170	3260	1970	2960	1810	2720		16	2540	3810	2280	3430	2090	3140
2090	3140	1890	2840	1740	2610		17	2500	3750	2240	3370	2050	3080
2000	3010	1810	2720	1660	2500		18	2460	3690	2200	3310	2010	3020
1910	2870	1730	2600	1590	2390		19	2420	3630	2160	3250	1970	2970
1820	2740	1650	2480	1510	2270		20	2380	3570	2120	3190	1940	2910
1650	2480	1490	2230	1360	2050		22	2300	3450	2040	3070	1860	2790
1470	2210	1330	1990	1220	1830		24	2220	3330	1970	2950	1780	2680
1310	1960	1170	1760	1070	1610		26	2130	3210	1890	2840	1710	2560
1140	1720	1020	1540	938	1410		28	2050	3090	1810	2720	1630	2450
996	1500	893	1340	817	1230		30	1970	2970	1730	2600	1550	2340
876	1320	784	1180	718	1080		32	1890	2840	1650	2480	1480	2220
776	1170	695	1040	636	956		34	1810	2720	1570	2360	1390	2090
692	1040	620	932	567	853		36	1730	2600	1470	2220	1290	1940
621	933	556	836	509	765		38	1630	2460	1380	2070	1200	1800
560	842	502	755	459	691		40	1530	2300	1290	1940	1120	1690
508	764	455	684	417	626		42	1450	2170	1210	1820	1060	1590
463	696	415	624	380	571		44	1370	2050	1150	1720	997	1500
424	637	380	571	347	522		46	1300	1950	1090	1630	944	1420
389	585	349	524	319	480		48	1230	1850	1030	1550	896	1350
359	539	321	483	294	442		50	1180	1770	984	1480	853	1280

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2960	4450	2700	4060	2490	3740
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2280	3420	2080	3120	1920	2880
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
739	1110	679	1020	613	919
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
607	912	545	819	500	751

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.4	37.0	10.3	34.9	10.3	33.4
Area, in.^{2}					
76.1		69.4		63.9	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
10800	859	9700	769	8910	704
r_y, in.					
3.36		3.33		3.32	
r_x/r_y					
3.54		3.54		3.55	

Note: Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W27

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W27×						Shape	W27×					
194^c		178^c		161^c		lb/ft	194		178		161	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2220	3330	2020	3040	1800	2700	0	2050	3080	1850	2780	1670	2510
2120	3190	1940	2920	1720	2590	6	2050	3080	1850	2780	1670	2510
2090	3140	1910	2870	1700	2550	7	2050	3080	1850	2780	1670	2510
2050	3080	1880	2820	1670	2510	8	2050	3080	1850	2780	1670	2510
2010	3020	1840	2770	1630	2460	9	2050	3080	1850	2780	1670	2510
1960	2940	1800	2700	1600	2400	10	2050	3080	1850	2780	1670	2510
1910	2870	1750	2630	1560	2340	11	2020	3030	1820	2730	1640	2460
1850	2780	1700	2550	1520	2280	12	1980	2980	1780	2680	1600	2410
1790	2700	1640	2470	1480	2220	13	1940	2920	1750	2630	1570	2360
1730	2610	1590	2380	1430	2150	14	1910	2870	1710	2570	1540	2310
1670	2510	1530	2290	1380	2070	15	1870	2810	1680	2520	1510	2260
1610	2420	1470	2200	1320	1990	16	1840	2760	1640	2470	1470	2210
1540	2320	1410	2110	1270	1910	17	1800	2700	1610	2420	1440	2160
1480	2220	1340	2020	1210	1820	18	1760	2650	1570	2370	1410	2110
1410	2120	1280	1920	1150	1730	19	1730	2590	1540	2310	1370	2070
1340	2010	1220	1830	1100	1650	20	1690	2540	1500	2260	1340	2020
1210	1810	1090	1640	982	1480	22	1620	2430	1440	2160	1280	1920
1070	1610	969	1460	870	1310	24	1540	2320	1370	2050	1210	1820
945	1420	851	1280	763	1150	26	1470	2210	1300	1950	1140	1720
823	1240	738	1110	661	994	28	1400	2100	1230	1840	1080	1620
717	1080	643	967	576	866	30	1330	1990	1160	1740	993	1490
630	947	565	850	506	761	32	1240	1870	1060	1590	900	1350
558	839	501	753	448	674	34	1140	1720	969	1460	823	1240
498	748	447	671	400	601	36	1060	1590	894	1340	757	1140
447	671	401	602	359	540	38	982	1480	829	1250	701	1050
403	606	362	544	324	487	40	918	1380	773	1160	652	980
366	550	328	493	294	442	42	861	1290	724	1090	610	916
333	501	299	449	268	402	44	811	1220	681	1020	572	860
305	458	274	411	245	368	46	767	1150	643	966	539	811
280	421	251	378	225	338	48	727	1090	608	914	510	766
258	388	232	348	207	312	50	692	1040	578	868	484	727

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2220	3340	2040	3070	1850	2780
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1710	2570	1580	2360	1430	2140
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
548	822	524	786	474	710
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
441	663	396	595	354	531

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.2	31.6	10.1	30.3	10.0	29.1
Area, in.^{2}					
57.1		52.5		47.6	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
7860	619	7020	555	6310	497
r_y, in.					
3.29		3.25		3.23	
r_x/r_y					
3.56		3.57		3.56	

^{c} Shape is slender for compression with F_y = 65 ksi.

Note: Confirm ASTM A913 material availability before specifying.

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W27

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W27×						Shape	W27×					
146^c		129^c		114^c		lb/ft	146		129		114	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1600	2400	1380	2080	1200	1800	0	1500	2260	1280	1930	1110	1670
1530	2300	1270	1900	1100	1650	6	1500	2260	1280	1930	1110	1670
1510	2260	1230	1840	1060	1600	7	1500	2260	1280	1920	1100	1660
1480	2220	1180	1780	1020	1540	8	1500	2260	1240	1860	1070	1610
1450	2180	1140	1710	983	1480	9	1500	2260	1200	1810	1040	1560
1420	2130	1080	1630	938	1410	10	1500	2260	1170	1750	1000	1510
1380	2080	1030	1550	890	1340	11	1470	2210	1130	1700	969	1460
1350	2020	975	1460	841	1260	12	1440	2170	1090	1640	936	1410
1310	1960	916	1380	791	1190	13	1410	2120	1050	1590	902	1360
1270	1900	849	1280	740	1110	14	1380	2070	1020	1530	868	1310
1220	1840	783	1180	684	1030	15	1350	2030	981	1470	835	1250
1180	1770	718	1080	626	940	16	1320	1980	944	1420	801	1200
1130	1700	655	984	569	855	17	1290	1930	908	1360	767	1150
1090	1640	593	892	514	773	18	1260	1890	871	1310	734	1100
1040	1560	534	802	462	694	19	1220	1840	834	1250	700	1050
985	1480	482	724	417	626	20	1190	1790	797	1200	657	988
880	1320	398	598	344	518	22	1130	1700	695	1050	564	848
779	1170	335	503	289	435	24	1070	1610	611	918	493	740
681	1020	285	428	247	371	26	1010	1510	544	817	436	655
589	885	246	369	213	320	28	946	1420	489	736	391	587
513	771	214	322	185	278	30	851	1280	445	669	354	532
451	678	188	283	163	245	32	769	1160	408	613	323	485
399	600	167	251	144	217	34	701	1050	376	566	297	446
356	535	149	223	129	193	36	644	967	349	525	275	413
320	481					38	594	893	326	490	256	384
289	434					40	552	830	306	460	239	359
262	393					42	515	774	288	433	225	338
239	358					44	483	725	272	409	212	318
218	328					46	454	682	258	388	200	301
200	301					48	429	644	245	368	190	286
185	278					50	406	610	234	351	181	272

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1680	2530	1470	2210	1310	1970
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1300	1940	1130	1700	1010	1510
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
431	647	438	657	405	607
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
317	476	187	281	160	240

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.91	28.2	6.85	20.4	6.75	19.6
Area, in.^{2}					
43.2		37.8		33.6	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
5660	443	4760	184	4080	159
r_y, in.					
3.20		2.21		2.18	
r_x/r_y					
3.59		5.07		5.05	

^{c} Shape is slender for compression with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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W27

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W27×						Shape	W27×					
102^c		94^c		84^c		lb/ft	102		94^v		84^v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1040	1560	939	1410	819	1230	0	989	1490	902	1360	791	1190
949	1430	855	1280	742	1120	6	989	1490	902	1360	791	1190
918	1380	826	1240	717	1080	7	979	1470	889	1340	776	1170
883	1330	795	1190	688	1030	8	948	1420	860	1290	749	1130
846	1270	760	1140	657	988	9	917	1380	830	1250	722	1090
806	1210	724	1090	624	939	10	886	1330	801	1200	695	1040
765	1150	685	1030	590	887	11	855	1280	772	1160	668	1000
721	1080	646	971	555	834	12	823	1240	742	1120	641	964
677	1020	605	910	519	779	13	792	1190	713	1070	614	924
632	951	565	849	482	725	14	761	1140	684	1030	588	883
588	883	524	787	446	670	15	730	1100	655	984	561	843
543	817	483	727	410	617	16	699	1050	625	940	534	802
496	746	444	667	376	564	17	668	1000	596	896	507	762
447	671	400	601	341	512	18	637	958	567	852	477	717
401	603	359	539	306	460	19	605	910	527	791	433	651
362	544	324	487	276	415	20	555	834	482	725	396	595
299	450	268	402	228	343	22	474	713	411	618	336	505
251	378	225	338	192	288	24	413	620	356	535	290	437
214	322	192	288	163	246	26	364	547	313	471	255	383
185	277	165	248	141	212	28	325	488	279	419	226	340
161	242	144	216	123	184	30	293	441	251	377	203	305
141	212	126	190	108	162	32	267	401	228	343	184	276
125	188	112	168	95.6	144	34	245	368	209	314	167	252
						36	226	340	192	289	154	231
						38	210	315	178	268	142	214
						40	196	294	166	249	132	199
						42	183	276	155	233	123	186
						44	173	260	146	219	116	174
						46	163	245	138	207	109	164
						48	155	232	130	196	103	155
						50	147	221	124	186	97.5	147

Properties

Available Strength in Tensile Yielding, kips							Limiting Unbraced Lengths, ft					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		L_p	L_r	L_p	L_r	L_p	L_r
1170	1760	1070	1610	961	1440		6.66	19.0	6.57	18.5	6.41	17.9
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips							Area, in.^{2}					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		30.0		27.6		24.7	
900	1350	828	1240	741	1110		Moment of Inertia, in.^{4}					
Available Strength in Shear, kips							I_x	I_y	I_x	I_y	I_x	I_y
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$		3620	139	3270	124	2850	106
363	544	308	463	287	431		r_y, in.					
Available Strength in Flexure about Y-Y Axis, kip-ft							2.15		2.12		2.07	
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$		r_x/r_y					
141	212	126	189	108	162		5.12		5.14		5.17	

[c] Shape is slender for compression with F_y = 65 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W24

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W24×						Shape		W24×					
370^h		335^h		306^h		lb/ft		370^h		335^h		306^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
4240	6380	3830	5750	3490	5250	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	3670	5510	3310	4970	2990	4490
4050	6090	3650	5490	3330	5000		6	3670	5510	3310	4970	2990	4490
3980	5990	3590	5390	3270	4910		7	3670	5510	3310	4970	2990	4490
3910	5870	3520	5290	3210	4820		8	3670	5510	3310	4970	2990	4490
3820	5750	3440	5170	3130	4710		9	3670	5510	3310	4970	2990	4490
3730	5610	3360	5040	3050	4590		10	3670	5510	3310	4970	2990	4490
3630	5460	3260	4910	2970	4460		11	3640	5460	3270	4920	2950	4440
3530	5300	3170	4760	2880	4330		12	3600	5410	3240	4870	2920	4390
3420	5140	3070	4610	2790	4190		13	3570	5360	3210	4820	2890	4340
3300	4960	2960	4450	2690	4040		14	3530	5310	3170	4770	2850	4290
3180	4780	2850	4280	2580	3880		15	3500	5260	3140	4710	2820	4240
3060	4590	2730	4110	2480	3730		16	3460	5210	3100	4660	2780	4190
2930	4400	2620	3940	2370	3570		17	3430	5160	3070	4610	2750	4130
2800	4210	2500	3760	2260	3400		18	3400	5100	3030	4560	2720	4080
2670	4020	2380	3580	2150	3240		19	3360	5050	3000	4510	2680	4030
2540	3820	2260	3400	2050	3070		20	3330	5000	2970	4460	2650	3980
2280	3430	2030	3050	1830	2750		22	3260	4900	2900	4350	2580	3880
2030	3050	1800	2700	1620	2430		24	3190	4800	2830	4250	2510	3780
1790	2680	1580	2370	1410	2130		26	3120	4690	2760	4150	2450	3680
1550	2330	1370	2050	1220	1840		28	3050	4590	2690	4040	2380	3580
1350	2030	1190	1790	1070	1600		30	2990	4490	2620	3940	2310	3470
1190	1790	1050	1570	936	1410		32	2920	4390	2550	3840	2240	3370
1050	1580	926	1390	829	1250		34	2850	4280	2480	3730	2180	3270
939	1410	826	1240	740	1110		36	2780	4180	2420	3630	2110	3170
843	1270	741	1110	664	998		38	2710	4080	2350	3530	2040	3070
760	1140	669	1010	599	901		40	2640	3970	2280	3430	1970	2970
690	1040	607	912	544	817		42	2580	3870	2210	3320	1910	2860
628	944	553	831	495	744		44	2510	3770	2140	3220	1840	2760
575	864	506	760	453	681		46	2440	3670	2070	3120	1760	2650
528	794	465	698	416	625		48	2370	3560	2000	3010	1680	2530
487	731	428	644	384	576		50	2300	3460	1930	2900	1610	2420

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4240	6380	3830	5750	3490	5250
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3270	4910	2950	4420	2690	4040
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1110	1660	987	1480	888	1330
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
866	1300	772	1160	694	1040

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.1	53.8	10.0	49.2	9.91	45.4
Area, in.^{2}					
109		98.3		89.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
13400	1160	11900	1030	10700	919
r_y, in.					
3.27		3.23		3.20	
r_x/r_y					
3.39		3.41		3.41	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

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W24

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W24×						Shape		W24×					
279^h		250		229		lb/ft		279^h		250		229	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
3190	4790	2860	4300	2620	3930	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	2710	4070	2410	3630	2190	3290
3040	4560	2720	4090	2490	3740		6	2710	4070	2410	3630	2190	3290
2980	4480	2670	4020	2440	3670		7	2710	4070	2410	3630	2190	3290
2920	4390	2620	3930	2390	3590		8	2710	4070	2410	3630	2190	3290
2850	4290	2560	3840	2330	3510		9	2710	4070	2410	3630	2190	3290
2780	4180	2490	3740	2270	3410		10	2700	4060	2400	3610	2180	3270
2700	4060	2420	3630	2200	3310		11	2670	4010	2370	3560	2150	3220
2620	3940	2340	3520	2130	3210		12	2640	3960	2340	3510	2110	3180
2530	3810	2260	3400	2060	3090		13	2600	3910	2310	3470	2080	3130
2440	3670	2180	3280	1980	2980		14	2570	3860	2270	3420	2050	3080
2350	3530	2090	3150	1900	2860		15	2540	3810	2240	3370	2020	3030
2250	3380	2010	3010	1820	2740		16	2500	3760	2210	3320	1980	2980
2150	3230	1920	2880	1740	2610		17	2470	3710	2170	3270	1950	2930
2050	3080	1820	2740	1650	2490		18	2430	3660	2140	3220	1920	2880
1950	2930	1730	2600	1570	2360		19	2400	3610	2110	3170	1890	2840
1850	2780	1640	2470	1490	2230		20	2370	3560	2080	3120	1850	2790
1650	2480	1460	2200	1320	1980		22	2300	3460	2010	3020	1790	2690
1450	2190	1290	1930	1160	1740		24	2230	3360	1940	2920	1730	2590
1270	1910	1120	1680	1000	1510		26	2170	3260	1880	2820	1660	2500
1100	1650	965	1450	865	1300		28	2100	3160	1810	2720	1600	2400
955	1430	840	1260	754	1130		30	2030	3060	1750	2630	1530	2300
839	1260	739	1110	663	996		32	1970	2960	1680	2530	1470	2210
743	1120	654	983	587	882		34	1900	2860	1620	2430	1400	2110
663	996	584	877	523	787		36	1830	2760	1550	2330	1340	2010
595	894	524	787	470	706		38	1770	2650	1480	2230	1260	1890
537	807	473	711	424	637		40	1700	2550	1400	2110	1180	1780
487	732	429	645	385	578		42	1630	2450	1330	2000	1120	1680
444	667	391	587	350	527		44	1550	2330	1260	1890	1060	1590
406	610	357	537	321	482		46	1480	2220	1200	1800	1010	1510
373	560	328	493	294	443		48	1410	2120	1140	1720	958	1440
344	516	303	455	271	408		50	1350	2020	1090	1640	915	1380

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3190	4790	2860	4300	2620	3930
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2460	3690	2210	3310	2020	3020
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
805	1210	711	1070	649	973
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
626	941	555	834	500	751

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.82	42.1	9.73	38.7	9.63	36.1
Area, in.^{2}					
81.9		73.5		67.2	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
9600	823	8490	724	7650	651
r_y, in.					
3.17		3.14		3.11	
r_x/r_y					
3.41		3.41		3.44	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W24

F_y = 65 ksi
F_u = 80 ksi

W24×						Shape		W24×					
207		192		176		lb/ft		207		192		176	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
2360	3550	2200	3310	2010	3020	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	1970	2950	1810	2730	1660	2490
2240	3370	2090	3140	1910	2870		6	1970	2950	1810	2730	1660	2490
2200	3310	2050	3080	1870	2810		7	1970	2950	1810	2730	1660	2490
2150	3240	2000	3010	1830	2750		8	1970	2950	1810	2730	1660	2490
2100	3160	1960	2940	1780	2680		9	1970	2950	1810	2730	1660	2490
2050	3070	1900	2860	1740	2610		10	1950	2930	1800	2700	1640	2460
1980	2980	1840	2770	1680	2530		11	1920	2890	1770	2660	1610	2420
1920	2880	1780	2680	1630	2440		12	1890	2840	1740	2610	1580	2370
1850	2780	1720	2590	1570	2350		13	1860	2790	1710	2560	1550	2330
1780	2680	1650	2490	1510	2260		14	1820	2740	1670	2520	1520	2280
1710	2570	1590	2380	1440	2170		15	1790	2700	1640	2470	1490	2240
1630	2450	1520	2280	1380	2070		16	1760	2650	1610	2420	1460	2190
1560	2340	1450	2170	1310	1970		17	1730	2600	1580	2380	1430	2150
1480	2220	1370	2060	1250	1870		18	1700	2550	1550	2330	1400	2100
1400	2110	1300	1960	1180	1770		19	1670	2510	1520	2290	1370	2060
1330	1990	1230	1850	1110	1670		20	1640	2460	1490	2240	1340	2010
1180	1770	1090	1640	983	1480		22	1570	2360	1430	2150	1280	1920
1030	1550	953	1430	857	1290		24	1510	2270	1370	2050	1220	1830
889	1340	822	1240	738	1110		26	1450	2170	1310	1960	1160	1740
767	1150	709	1070	636	956		28	1380	2080	1240	1870	1100	1650
668	1000	618	928	554	833		30	1320	1980	1180	1780	1040	1560
587	882	543	816	487	732		32	1260	1890	1120	1680	963	1450
520	781	481	723	431	648		34	1190	1790	1040	1560	889	1340
464	697	429	645	385	578		36	1110	1660	966	1450	826	1240
416	626	385	579	345	519		38	1040	1560	903	1360	771	1160
376	565	347	522	312	468		40	975	1460	848	1270	723	1090
341	512	315	474	283	425		42	920	1380	800	1200	681	1020
310	467	287	432	258	387		44	871	1310	757	1140	643	967
284	427	263	395	236	354		46	827	1240	718	1080	610	916
261	392	241	363	216	325		48	787	1180	683	1030	580	871
240	361	222	334	199	300		50	752	1130	652	979	552	830

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2360	3550	2200	3310	2010	3020
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1820	2730	1700	2540	1550	2330
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
581	872	537	806	491	737
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
444	668	409	614	373	561

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.54	33.6	9.51	32.2	9.42	30.6
Area, in.^{2}					
60.7		56.5		51.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
6820	578	6260	530	5680	479
r_y, in.					
3.08		3.07		3.04	
r_x/r_y					
3.44		3.42		3.45	

Note: Confirm ASTM A913 material availability before specifying.

W24

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 65 ksi

F_u = 80 ksi

W24×						Shape		W24×					
162		146^c		131^c		lb/ft		162		146		131	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
1860	2800	1650	2490	1460	2190	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	1520	2280	1360	2040	1200	1800
1760	2650	1580	2370	1390	2090		6	1520	2280	1360	2040	1200	1800
1730	2600	1550	2330	1360	2050		7	1520	2280	1360	2040	1200	1800
1690	2540	1520	2280	1340	2010		8	1520	2280	1360	2040	1200	1800
1650	2480	1480	2230	1310	1960		9	1520	2280	1360	2040	1200	1800
1610	2410	1440	2160	1270	1910		10	1500	2260	1340	2010	1180	1770
1560	2340	1390	2100	1240	1860		11	1470	2210	1310	1970	1150	1730
1510	2260	1350	2020	1200	1800		12	1440	2170	1280	1930	1130	1700
1450	2180	1300	1950	1160	1740		13	1420	2130	1260	1890	1100	1660
1390	2100	1240	1870	1110	1670		14	1390	2080	1230	1850	1080	1620
1340	2010	1190	1790	1060	1590		15	1360	2040	1200	1800	1050	1580
1280	1920	1140	1710	1010	1520		16	1330	2000	1170	1760	1020	1540
1220	1830	1080	1630	959	1440		17	1300	1950	1150	1720	999	1500
1160	1740	1030	1540	909	1370		18	1270	1910	1120	1680	973	1460
1090	1640	970	1460	858	1290		19	1240	1870	1090	1640	948	1420
1030	1550	915	1370	808	1210		20	1210	1820	1060	1600	922	1390
913	1370	806	1210	709	1070		22	1160	1740	1010	1520	870	1310
797	1200	701	1050	615	924		24	1100	1650	953	1430	819	1230
687	1030	602	904	526	790		26	1040	1560	899	1350	767	1150
592	890	519	780	453	681		28	983	1480	844	1270	697	1050
516	775	452	679	395	594		30	917	1380	763	1150	628	943
453	681	397	597	347	522		32	839	1260	696	1050	570	857
402	603	352	529	307	462		34	773	1160	639	960	523	785
358	538	314	472	274	412		36	716	1080	591	888	482	724
321	483	282	423	246	370		38	667	1000	549	826	447	672
290	436	254	382	222	334		40	625	939	513	771	417	626
263	395	231	346	201	303		42	587	883	482	724	390	586
240	360	210	316	184	276		44	554	833	454	682	367	551
219	330	192	289	168	252		46	525	789	429	645	346	520
201	303	176	265	154	232		48	498	749	407	611	328	493
186	279	163	244				50	474	713	387	581	311	468

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1860	2800	1670	2520	1500	2260
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1430	2150	1290	1940	1160	1740
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
458	687	417	626	385	578
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
341	512	302	454	264	397

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.45	29.5	9.32	28.1	9.20	26.8
Area, in.^{2}					
47.8		43.0		38.6	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
5170	443	4580	391	4020	340
r_y, in.					
3.05		3.01		2.97	
r_x/r_y					
3.41		3.42		3.43	

^{c} Shape is slender for compression with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W24

F_y = 65 ksi
F_u = 80 ksi

W24×						Shape	W24×					
117^c		104^c		103^c		lb/ft	117		104^f		103	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
1270	1910	1110	1660	1110	1670	0	1060	1590	925	1390	908	1370
1210	1820	1050	1580	998	1500	6	1060	1590	925	1390	908	1370
1190	1780	1030	1550	960	1440	7	1060	1590	925	1390	884	1330
1160	1750	1010	1520	918	1380	8	1060	1590	925	1390	855	1290
1130	1710	987	1480	872	1310	9	1060	1590	925	1390	827	1240
1110	1660	960	1440	824	1240	10	1040	1560	916	1380	798	1200
1070	1610	932	1400	774	1160	11	1020	1530	893	1340	769	1160
1040	1560	902	1360	717	1080	12	991	1490	871	1310	741	1110
1000	1510	871	1310	658	988	13	967	1450	849	1280	712	1070
967	1450	838	1260	599	900	14	943	1420	827	1240	683	1030
928	1400	804	1210	542	814	15	919	1380	805	1210	654	984
889	1340	770	1160	487	732	16	895	1350	783	1180	626	940
847	1270	734	1100	433	651	17	871	1310	760	1140	597	897
802	1200	699	1050	387	581	18	847	1270	738	1110	568	854
756	1140	663	997	347	521	19	823	1240	716	1080	529	796
711	1070	626	941	313	471	20	799	1200	694	1040	490	736
622	935	546	821	259	389	22	752	1130	650	976	425	639
538	808	471	708	217	327	24	704	1060	605	910	375	563
459	690	401	603	185	278	26	651	979	544	817	335	504
396	595	346	520	160	240	28	578	869	481	723	303	455
345	518	302	453	139	209	30	519	781	430	647	276	415
303	456	265	398	122	184	32	470	707	389	584	254	382
268	404	235	353			34	430	646	354	532	235	353
239	360	209	315			36	395	594	324	488	219	329
215	323	188	282			38	365	549	299	450	205	308
194	292	170	255			40	340	511	278	417	192	289
176	264	154	231			42	317	477	259	389	181	273
160	241	140	211			44	298	448	242	364	172	258
147	220	128	193			46	281	422	228	342	163	245
135	202	118	177			48	265	398	215	323	155	233
						50	251	378	203	305	148	222

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1340	2010	1190	1800	1180	1770
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1030	1550	921	1380	909	1360
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
347	521	313	470	350	526
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
232	348	198	298	135	202

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.11	25.8	9.59	24.9	6.16	18.4
Area, in.^{2}					
34.4		30.7		30.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
3540	297	3100	259	3000	119
r_y, in.					
2.94		2.91		1.99	
r_x/r_y					
3.44		3.47		5.03	

[c] Shape is slender for compression with F_y = 65 ksi.

[f] Shape exceeds compact limit for flexure with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W24

F_y = 65 ksi
F_u = 80 ksi

W24×						Shape	W24×					
94^c		84^c		76^c		lb/ft	94		84		76^v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
995	1500	864	1300	766	1150	0	824	1240	727	1090	649	975
893	1340	772	1160	683	1030	6	824	1240	727	1090	647	973
859	1290	742	1120	655	985	7	800	1200	703	1060	624	938
821	1230	708	1060	625	939	8	773	1160	678	1020	601	904
780	1170	672	1010	592	890	9	746	1120	653	981	578	869
737	1110	634	953	557	838	10	719	1080	628	943	555	834
692	1040	594	893	522	784	11	692	1040	603	906	532	799
646	971	553	832	485	729	12	665	1000	578	868	508	764
598	898	512	770	448	674	13	638	959	553	831	485	729
544	817	471	709	411	618	14	611	918	528	793	462	695
491	739	428	643	375	564	15	584	878	503	755	439	660
441	663	383	575	337	506	16	557	837	478	718	416	625
392	590	339	510	298	448	17	530	796	453	680	387	582
350	526	303	455	266	400	18	501	753	416	626	351	528
314	472	272	408	239	359	19	460	691	381	572	320	482
283	426	245	368	215	324	20	424	638	350	527	294	442
234	352	203	304	178	268	22	366	551	301	453	252	379
197	296	170	256	150	225	24	322	484	264	396	220	330
168	252	145	218	128	192	26	287	431	234	351	194	292
145	217	125	188	110	165	28	258	388	210	315	174	261
126	189	109	164	95.8	144	30	235	353	190	286	157	236
111	166	95.7	144	84.2	127	32	215	324	174	261	143	215
						34	199	299	160	241	131	198
						36	185	277	148	223	121	183
						38	172	259	138	208	113	170
						40	162	243	129	194	106	159
						42	152	229	122	183	99.0	149
						44	144	216	115	172	93.2	140
						46	136	205	109	163	88.1	132
						48	130	195	103	155	83.6	126
						50	124	186	98.2	148	79.5	119

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1080	1620	961	1440	872	1310
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
831	1250	741	1110	672	1010
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
325	488	295	442	246	369
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
122	183	106	159	92.8	139

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
6.13	18.0	6.04	17.3	5.95	16.7
Area, in.^{2}					
27.7		24.7		22.4	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
2700	109	2370	94.4	2100	82.5
r_y, in.					
1.98		1.95		1.92	
r_x/r_y					
4.98		5.02		5.05	

^{c} Shape is slender for compression with F_y = 65 ksi.

^{v} Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

W24

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

Shape	W24×						W24×					
lb/ft	68^c		62^c		55^c		68^v		62^v		55^v	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	670	1010	600	902	515	774	574	863	496	746	435	653
6	595	894	485	729	411	618	570	856	454	682	393	591
7	569	856	449	675	379	570	548	824	430	646	371	558
8	542	814	411	618	345	519	527	792	405	609	349	524
9	512	770	372	559	311	467	505	759	381	572	326	490
10	481	723	333	500	276	415	484	727	356	535	304	457
11	449	674	294	441	243	365	462	694	332	499	282	423
12	416	625	251	378	211	317	441	662	307	462	259	390
13	383	575	214	322	180	270	419	630	274	411	225	338
14	350	526	185	277	155	233	398	597	241	362	197	296
15	318	478	161	242	135	203	376	565	214	322	175	263
16	287	431	141	212	119	178	354	533	192	289	157	235
17	254	382	125	188	105	158	322	483	174	262	141	212
18	226	340	112	168	93.7	141	291	437	159	239	129	193
19	203	305	100	151	84.1	126	265	398	146	219	118	177
20	183	276	90.4	136	75.9	114	243	365	135	202	108	163
22	152	228	74.7	112	62.7	94.3	207	311	116	175	93.3	140
24	127	191					180	270	102	154	81.7	123
26	109	163					158	238	91.2	137	72.6	109
28	93.6	141					141	212	82.2	124	65.2	98.0
30	81.5	123					127	191	74.8	112	59.1	88.9
32							116	174	68.7	103	54.1	81.3
34							106	159	63.4	95.3	49.8	74.9
36							97.5	147	58.9	88.6	46.2	69.4
38							90.4	136	55.1	82.8	43.1	64.7
40							84.3	127	51.7	77.7	40.3	60.6
42							78.9	119	48.7	73.2	37.9	57.0
44							74.2	112	46.0	69.2	35.8	53.8
46							70.0	105	43.6	65.6	33.9	51.0
48							66.3	99.6	41.5	62.4	32.2	48.4
50							62.9	94.6	39.6	59.5	30.7	46.1

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
782	1180	708	1060	631	948
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
603	905	546	819	486	729
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
230	345	238	358	214	322
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
79.5	119	50.9	76.4	43.1	64.7

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.79	16.2	4.28	12.4	4.15	12.0
Area, in.^{2}					
20.1		18.2		16.2	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1830	70.4	1550	34.5	1350	29.1
r_y, in.					
1.87		1.38		1.34	
r_x/r_y					
5.11		6.69		6.80	

[c] Shape is slender for compression with F_y = 65 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W21

F_y = 65 ksi
F_u = 80 ksi

W21×						Shape	W21×					
275^h		248		223		lb/ft	275^h		248		223	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
3180	4790	2870	4320	2590	3890	0	2430	3650	2180	3270	1950	2930
3020	4550	2730	4100	2450	3690	6	2430	3650	2180	3270	1950	2930
2970	4460	2680	4020	2410	3620	7	2430	3650	2180	3270	1950	2930
2910	4370	2620	3940	2350	3540	8	2430	3650	2180	3270	1950	2930
2840	4260	2560	3840	2300	3450	9	2430	3650	2180	3270	1950	2930
2760	4150	2490	3740	2230	3350	10	2420	3640	2170	3250	1930	2910
2680	4030	2410	3630	2160	3250	11	2390	3600	2140	3220	1910	2870
2590	3900	2330	3510	2090	3140	12	2370	3560	2120	3180	1890	2830
2500	3760	2250	3380	2020	3030	13	2340	3520	2090	3140	1860	2800
2410	3620	2160	3250	1940	2910	14	2320	3490	2070	3110	1840	2760
2310	3470	2080	3120	1850	2790	15	2290	3450	2040	3070	1810	2720
2210	3320	1990	2980	1770	2660	16	2270	3410	2020	3030	1790	2690
2110	3170	1890	2850	1690	2540	17	2240	3370	1990	2990	1760	2650
2010	3020	1800	2710	1600	2410	18	2220	3330	1970	2960	1740	2610
1900	2860	1710	2560	1520	2280	19	2190	3300	1940	2920	1710	2570
1800	2710	1610	2420	1430	2150	20	2170	3260	1920	2880	1690	2540
1600	2400	1430	2150	1260	1900	22	2120	3180	1870	2810	1640	2460
1400	2110	1250	1880	1100	1660	24	2070	3110	1820	2730	1590	2390
1210	1820	1080	1620	949	1430	26	2020	3030	1770	2660	1540	2310
1050	1570	932	1400	818	1230	28	1970	2960	1720	2580	1490	2240
912	1370	812	1220	713	1070	30	1920	2880	1670	2510	1440	2160
801	1200	714	1070	626	942	32	1870	2810	1620	2440	1390	2090
710	1070	632	950	555	834	34	1820	2730	1570	2360	1340	2020
633	952	564	847	495	744	36	1770	2660	1520	2290	1290	1940
568	854	506	761	444	668	38	1720	2580	1470	2210	1240	1870
513	771	457	686	401	603	40	1670	2510	1420	2140	1190	1790
465	699	414	623	364	547	42	1620	2430	1370	2060	1130	1700
424	637	377	567	331	498	44	1570	2350	1320	1990	1080	1620
388	583	345	519	303	456	46	1520	2280	1270	1900	1020	1540
356	535	317	477	278	418	48	1470	2200	1210	1820	978	1470
328	493	292	439	257	386	50	1410	2120	1160	1740	936	1410

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3180	4790	2870	4320	2590	3890
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2450	3680	2210	3320	2000	2990
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
764	1150	678	1020	608	913
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
620	931	551	829	487	731

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.60	48.7	9.54	44.6	9.42	40.5
Area, in.^{2}					
81.8		73.8		66.5	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
7690	787	6830	699	6080	614
r_y, in.					
3.10		3.08		3.04	
r_x/r_y					
3.13		3.12		3.14	

^{h} Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

W21

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W21×						Shape	W21×					
201		182		166		lb/ft	201		182		166	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Design	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2310	3470	2090	3140	1900	2850	0	1720	2580	1540	2320	1400	2110
2190	3290	1980	2970	1800	2700	6	1720	2580	1540	2320	1400	2110
2140	3220	1940	2910	1760	2650	7	1720	2580	1540	2320	1400	2110
2100	3150	1890	2840	1720	2590	8	1720	2580	1540	2320	1400	2110
2040	3070	1840	2770	1680	2520	9	1720	2580	1540	2320	1400	2110
1990	2990	1790	2690	1630	2450	10	1700	2560	1530	2300	1380	2080
1920	2890	1740	2610	1580	2370	11	1680	2520	1500	2260	1360	2040
1860	2790	1680	2520	1520	2290	12	1650	2490	1480	2220	1340	2010
1790	2690	1610	2420	1470	2200	13	1630	2450	1450	2190	1310	1970
1720	2590	1550	2330	1410	2110	14	1600	2410	1430	2150	1290	1940
1650	2470	1480	2230	1350	2020	15	1580	2380	1410	2110	1270	1900
1570	2360	1410	2120	1280	1930	16	1560	2340	1380	2080	1240	1870
1500	2250	1340	2020	1220	1830	17	1530	2300	1360	2040	1220	1830
1420	2130	1270	1920	1160	1740	18	1510	2260	1330	2010	1200	1800
1340	2020	1200	1810	1090	1640	19	1480	2230	1310	1970	1170	1760
1270	1900	1140	1710	1030	1550	20	1460	2190	1290	1930	1150	1730
1120	1680	999	1500	905	1360	22	1410	2120	1240	1860	1100	1660
972	1460	869	1310	786	1180	24	1360	2040	1190	1790	1060	1590
835	1260	745	1120	674	1010	26	1310	1970	1140	1720	1010	1520
720	1080	642	965	581	873	28	1260	1890	1090	1650	961	1440
627	943	559	841	506	760	30	1210	1820	1050	1570	914	1370
551	829	492	739	445	668	32	1160	1750	999	1500	867	1300
488	734	436	655	394	592	34	1110	1670	951	1430	805	1210
436	655	389	584	351	528	36	1060	1600	888	1330	750	1130
391	588	349	524	315	474	38	1000	1510	833	1250	703	1060
353	530	315	473	285	428	40	947	1420	784	1180	661	993
320	481	285	429	258	388	42	896	1350	741	1110	624	938
292	438	260	391	235	354	44	850	1280	703	1060	591	888
267	401	238	358	215	323	46	809	1220	668	1000	562	844
245	368	219	328	198	297	48	771	1160	637	957	535	804
226	339	201	303			50	737	1110	609	915	511	768

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2310	3470	2090	3140	1900	2850
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1780	2670	1610	2410	1460	2200
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
544	816	490	735	439	658
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
431	648	386	580	350	527

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.36	36.7	9.29	34.2	9.26	32.2
Area, in.^{2}					
59.3		53.6		48.8	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
5310	542	4730	483	4280	435
r_y, in.					
3.02		3.00		2.99	
r_x/r_y					
3.14		3.13		3.13	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W21

F_y = 65 ksi
F_u = 80 ksi

W21×						Shape		W21×					
147		132		122		lb/ft		147		132		122	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
1680	2530	1510	2270	1400	2100	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	1210	1820	1080	1620	996	1500
1590	2390	1430	2140	1320	1980		6	1210	1820	1080	1620	996	1500
1560	2340	1400	2100	1290	1940		7	1210	1820	1080	1620	996	1500
1520	2290	1360	2050	1260	1900		8	1210	1820	1080	1620	996	1500
1480	2220	1330	1990	1230	1840		9	1210	1820	1080	1620	996	1500
1440	2160	1290	1940	1190	1790		10	1190	1790	1060	1590	976	1470
1390	2090	1250	1870	1150	1730		11	1170	1760	1040	1560	955	1440
1340	2020	1200	1800	1110	1670		12	1150	1720	1020	1530	935	1400
1290	1940	1150	1730	1070	1600		13	1120	1690	996	1500	914	1370
1240	1860	1100	1660	1020	1530		14	1100	1650	975	1460	893	1340
1180	1770	1050	1590	974	1460		15	1080	1620	953	1430	872	1310
1120	1690	1000	1510	926	1390		16	1060	1590	932	1400	852	1280
1070	1600	953	1430	879	1320		17	1030	1550	910	1370	831	1250
1010	1520	901	1350	831	1250		18	1010	1520	889	1340	810	1220
953	1430	849	1280	783	1180		19	988	1490	868	1300	790	1190
896	1350	798	1200	735	1110		20	966	1450	846	1270	769	1160
785	1180	698	1050	642	966		22	921	1380	803	1210	728	1090
680	1020	603	906	554	833		24	876	1320	760	1140	686	1030
580	872	514	773	473	710		26	831	1250	718	1080	645	969
501	752	443	667	408	613		28	786	1180	675	1010	594	892
436	655	386	581	355	534		30	737	1110	615	924	538	808
383	576	340	510	312	469		32	676	1020	563	846	491	738
339	510	301	452	276	415		34	625	939	519	779	452	679
303	455	268	403	247	371		36	581	873	481	723	418	629
272	408	241	362	221	333		38	542	815	448	674	390	585
245	369	217	327	200	300		40	509	765	420	631	364	548
222	334	197	296	181	272		42	480	721	395	594	342	515
203	305	180	270	165	248		44	453	681	373	561	323	485
185	279	164	247	151	227		46	430	646	353	531	306	459
170	256	151	227	139	208		48	409	615	336	505	290	436
							50	390	586	320	481	276	415

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1680	2530	1510	2270	1400	2100
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1300	1940	1160	1750	1080	1620
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
414	621	368	553	339	508
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
300	451	267	401	245	369

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.14	29.7	9.08	28.2	9.05	27.2
Area, in.^{2}					
43.2		38.8		35.9	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
3630	376	3220	333	2960	305
r_y, in.					
2.95		2.93		2.92	
r_x/r_y					
3.11		3.11		3.11	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

W21

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W21×						Shape	W21×					
111^c		101^c		93^c		lb/ft	111		101		93	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1250	1870	1120	1680	1050	1590	0	905	1360	821	1230	717	1080
1180	1780	1060	1590	919	1380	6	905	1360	821	1230	710	1070
1160	1750	1040	1560	872	1310	7	905	1360	821	1230	687	1030
1140	1710	1020	1530	820	1230	8	905	1360	821	1230	663	997
1110	1670	992	1490	766	1150	9	905	1360	820	1230	640	962
1080	1620	965	1450	709	1070	10	885	1330	801	1200	617	927
1040	1570	936	1410	651	979	11	865	1300	783	1180	594	892
1000	1510	905	1360	594	892	12	845	1270	764	1150	570	857
964	1450	872	1310	537	806	13	826	1240	745	1120	547	822
922	1390	839	1260	481	723	14	806	1210	727	1090	524	787
880	1320	802	1210	428	643	15	786	1180	708	1060	500	752
837	1260	762	1150	377	566	16	766	1150	690	1040	477	717
793	1190	722	1090	334	502	17	747	1120	671	1010	454	682
749	1130	682	1030	298	448	18	727	1090	652	980	427	642
705	1060	642	965	267	402	19	707	1060	634	952	395	594
662	995	602	905	241	363	20	687	1030	615	925	367	551
577	867	525	789	199	300	22	648	974	578	869	321	482
497	747	451	678	167	252	24	608	914	541	813	285	429
423	636	384	578	143	215	26	569	855	495	744	257	386
365	549	331	498	123	185	28	510	767	441	663	233	351
318	478	289	434	107	161	30	461	692	397	597	214	321
279	420	254	381			32	420	631	361	543	197	297
248	372	225	338			34	385	579	331	497	183	276
221	332	200	301			36	356	535	305	458	171	258
198	298	180	270			38	331	497	283	425	161	242
179	269	162	244			40	309	464	263	396	151	228
162	244	147	221			42	290	436	247	371	143	215
148	222	134	202			44	273	410	232	349	136	204
135	203	123	185			46	258	388	219	329	129	194
124	187	113	169			48	245	367	207	311	123	185
						50	232	349	197	296	118	177

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1270	1910	1160	1740	1060	1600
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
978	1470	894	1340	819	1230
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
307	461	278	417	326	489
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
221	332	200	301	113	169

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.98	26.2	8.95	25.4	5.70	17.8
Area, in.^{2}					
32.6		29.8		27.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
2670	274	2420	248	2070	92.9
r_y, in.					
2.9		2.89		1.84	
r_x/r_y					
3.12		3.12		4.73	

[c] Shape is slender for compression with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W21

F_y = 65 ksi
F_u = 80 ksi

W21×						Shape	W21×					
83^c		73^c		68^c		lb/ft	83		73		68	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
912	1370	777	1170	711	1070	0	636	956	558	839	519	780
804	1210	682	1030	623	937	6	628	945	550	827	511	768
768	1150	651	979	595	894	7	607	912	530	796	491	738
728	1090	617	927	563	846	8	585	879	510	766	472	709
682	1030	580	872	529	796	9	563	846	490	736	452	680
631	949	542	815	494	743	10	541	813	470	706	433	651
579	870	502	755	458	688	11	519	780	450	676	414	622
527	792	459	689	421	633	12	497	747	429	645	394	593
476	715	413	621	381	573	13	475	714	409	615	375	563
426	641	369	555	340	511	14	453	681	389	585	355	534
379	569	327	491	301	452	15	431	648	369	555	336	505
333	501	287	432	264	397	16	409	615	349	525	315	474
295	444	254	382	234	352	17	387	581	320	481	285	429
263	396	227	341	209	314	18	354	532	292	440	260	391
236	355	204	306	187	282	19	326	491	269	404	239	359
213	320	184	276	169	254	20	302	455	248	373	220	331
176	265	152	228	140	210	22	264	396	215	324	191	286
148	223	128	192	117	176	24	233	351	190	285	168	252
126	190	109	163	100	150	26	209	314	169	255	149	224
109	164	93.8	141	86.3	130	28	190	285	153	230	135	202
94.8	142	81.7	123	75.2	113	30	173	261	140	210	122	184
						32	160	240	128	193	112	169
						34	148	223	119	178	104	156
						36	138	208	110	166	96.4	145
						38	129	195	103	155	90.0	135
						40	122	183	96.9	146	84.5	127
						42	115	173	91.3	137	79.6	120
						44	109	164	86.4	130	75.2	113
						46	104	156	82.0	123	71.3	107
						48	98.6	148	78.0	117	67.8	102
						50	94.2	142	74.4	112	64.7	97.2

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
950	1430	837	1260	778	1170
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
732	1100	645	968	600	900
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
287	430	251	376	236	354
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
98.9	149	86.3	130	79.1	119

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.67	17.0	5.61	16.3	5.58	15.9
Area, in.^{2}					
24.4		21.5		20.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1830	81.4	1600	70.6	1480	64.7
r_y, in.					
1.83		1.81		1.80	
r_x/r_y					
4.74		4.77		4.78	

[c] Shape is slender for compression with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W21

F_y = 65 ksi
F_u = 80 ksi

W21×						Shape		W21×					
62^c		55^c		48^c		lb/ft		62		55^v		48$^{f, v}$	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
636	956	549	825	462	695	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	467	702	409	614	332	499
556	835	477	717	398	598		6	458	688	398	598	332	499
529	795	453	681	377	566		7	440	661	381	573	319	480
500	752	428	643	354	532		8	422	635	365	548	305	458
469	705	400	602	329	495		9	404	608	348	523	290	435
437	657	372	559	304	458		10	386	581	332	499	275	413
404	607	343	515	279	419		11	369	554	315	474	260	391
371	557	313	471	254	381		12	351	527	299	449	245	368
338	507	284	428	229	343		13	333	500	282	424	230	346
303	455	256	385	204	307		14	315	474	265	399	215	324
266	400	225	338	180	271		15	297	447	248	372	193	290
234	351	198	297	158	238		16	273	410	221	332	172	259
207	311	175	263	140	211		17	246	370	199	299	155	233
185	278	156	235	125	188		18	224	337	181	272	140	210
166	249	140	211	112	169		19	205	309	165	248	128	192
150	225	127	190	101	152		20	189	284	152	228	117	176
124	186	105	157	83.8	126		22	163	245	130	195	99.8	150
104	156	87.9	132	70.4	106		24	143	214	113	170	86.7	130
88.5	133	74.9	113	60.0	90.2		26	127	190	100	151	76.3	115
76.3	115	64.6	97.0				28	114	171	89.8	135	68.1	102
							30	103	155	81.2	122	61.3	92.2
							32	94.4	142	74.0	111	55.8	83.8
							34	87.0	131	68.0	102	51.1	76.8
							36	80.7	121	62.9	94.6	47.1	70.8
							38	75.2	113	58.5	87.9	43.7	65.7
							40	70.4	106	54.7	82.2	40.7	61.2
							42	66.2	99.6	51.3	77.1	38.2	57.4
							44	62.5	94.0	48.4	72.7	35.9	53.9
							46	59.2	89.0	45.7	68.7	33.9	50.9
							48	56.3	84.5	43.4	65.2	32.1	48.2
							50	53.6	80.5	41.2	62.0	30.4	45.8

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
712	1070	631	948	549	825
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
549	824	486	729	423	635
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
218	328	182	274	168	253
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
70.4	106	59.7	89.7	45.4	68.2

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.48	15.5	5.36	14.9	6.15	14.3
Area, in.^{2}					
18.3		16.2		14.1	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1330	57.5	1140	48.4	959	38.7
r_y, in.					
1.77		1.73		1.66	
r_x/r_y					
4.82		4.86		4.96	

[c] Shape is slender for compression with F_y = 65 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 65 ksi.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

W21

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W21×						Shape		W21×					
57^c		50^c		44^c		lb/ft		57		50^v		44^v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending		ASD	LRFD	ASD	LRFD	ASD	LRFD
575	864	491	738	420	631		0	418	629	357	536	309	465
458	688	385	579	325	488		6	381	572	320	481	274	411
422	634	353	531	296	445		7	360	541	301	453	257	386
384	577	319	480	266	400		8	339	510	283	425	240	360
345	518	285	428	236	354		9	319	479	264	397	223	335
306	459	251	377	206	310		10	298	448	246	369	206	309
262	394	214	322	177	267		11	277	417	227	341	189	284
221	332	180	271	150	225		12	257	386	204	307	164	246
188	283	153	231	127	192		13	227	341	178	267	142	214
162	244	132	199	110	165		14	201	302	157	236	125	188
141	212	115	173	95.7	144		15	180	270	140	210	111	167
124	187	101	152	84.2	126		16	163	244	126	189	99.9	150
110	165	89.7	135	74.5	112		17	148	222	114	172	90.4	136
98.1	147	80.0	120	66.5	99.9		18	136	204	105	157	82.4	124
88.0	132	71.8	108	59.7	89.7		19	125	188	96.2	145	75.6	114
79.4	119	64.8	97.4	53.9	80.9		20	116	175	89.0	134	69.8	105
65.6	98.7						22	101	153	77.3	116	60.3	90.7
							24	90.0	135	68.2	103	53.0	79.7
							26	80.8	121	61.0	91.7	47.3	71.0
							28	73.4	110	55.2	82.9	42.6	64.0
							30	67.2	101	50.4	75.7	38.8	58.3
							32	62.0	93.1	46.3	69.6	35.6	53.4
							34	57.5	86.4	42.9	64.5	32.8	49.4
							36	53.7	80.7	39.9	60.0	30.5	45.9
							38	50.3	75.6	37.4	56.2	28.5	42.8
							40	47.4	71.2	35.1	52.8	26.7	40.2
							42	44.8	67.3	33.1	49.8	25.2	37.9
							44	42.4	63.8	31.4	47.2	23.8	35.8
							46	40.3	60.6	29.8	44.8	22.6	33.9
							48	38.5	57.8	28.4	42.6	21.5	32.3
							50	36.7	55.2	27.1	40.7	20.5	30.8

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
650	977	572	860	506	761
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
501	752	441	662	390	585
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
222	333	185	277	169	254
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
48.0	72.2	39.6	59.5	33.1	49.7

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
4.18	12.2	4.03	11.7	3.90	11.2
Area, in.^{2}					
16.7		14.7		13.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1170	30.6	984	24.9	843	20.7
r_y, in.					
1.35		1.30		1.26	
r_x/r_y					
6.19		6.29		6.40	

[c] Shape is slender for compression with F_y = 65 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W18

F_y = 65 ksi
F_u = 80 ksi

W18×						Shape	W18×					
311^h		283^h		258^h		lb/ft	311^h		283^h		258^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
3570	5360	3240	4870	2960	4450	0	2450	3680	2190	3300	1980	2980
3370	5060	3060	4600	2790	4190	6	2450	3680	2190	3300	1980	2980
3300	4960	3000	4500	2730	4100	7	2450	3680	2190	3300	1980	2980
3220	4850	2920	4390	2660	4000	8	2450	3680	2190	3300	1980	2980
3140	4720	2840	4280	2590	3890	9	2450	3680	2190	3300	1980	2980
3050	4580	2760	4150	2510	3770	10	2430	3650	2170	3270	1960	2950
2950	4430	2670	4010	2420	3640	11	2410	3620	2150	3240	1940	2920
2840	4270	2570	3860	2330	3510	12	2390	3590	2140	3210	1920	2890
2730	4110	2470	3710	2240	3360	13	2370	3560	2120	3180	1900	2860
2620	3940	2360	3550	2140	3220	14	2350	3540	2100	3150	1890	2830
2500	3760	2250	3390	2040	3070	15	2330	3510	2080	3120	1870	2810
2380	3580	2140	3220	1940	2910	16	2310	3480	2060	3100	1850	2780
2260	3400	2030	3050	1840	2760	17	2290	3450	2040	3070	1830	2750
2140	3220	1920	2890	1730	2600	18	2280	3420	2020	3040	1810	2720
2020	3040	1810	2720	1630	2450	19	2260	3390	2000	3010	1790	2690
1900	2860	1700	2550	1530	2300	20	2240	3360	1980	2980	1770	2660
1670	2500	1480	2230	1330	2000	22	2200	3300	1950	2930	1740	2610
1440	2170	1280	1920	1140	1720	24	2160	3250	1910	2870	1700	2550
1230	1850	1090	1640	973	1460	26	2120	3190	1870	2810	1660	2490
1060	1600	939	1410	839	1260	28	2080	3130	1830	2750	1620	2440
925	1390	818	1230	731	1100	30	2040	3070	1790	2700	1580	2380
813	1220	719	1080	643	966	32	2010	3010	1760	2640	1550	2320
720	1080	637	957	569	855	34	1970	2960	1720	2580	1510	2270
642	965	568	854	508	763	36	1930	2900	1680	2530	1470	2210
576	866	510	766	456	685	38	1890	2840	1640	2470	1430	2150
520	782	460	692	411	618	40	1850	2780	1600	2410	1400	2100
472	709	417	627	373	561	42	1810	2730	1570	2350	1360	2040
430	646	380	572	340	511	44	1780	2670	1530	2300	1320	1980
393	591	348	523	311	467	46	1740	2610	1490	2240	1280	1930
361	543	320	480	286	429	48	1700	2550	1450	2180	1240	1870
						50	1660	2490	1410	2130	1210	1810

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3570	5360	3240	4870	2960	4450
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2750	4120	2500	3750	2280	3420
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
881	1320	797	1200	716	1070
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
671	1010	600	902	538	809

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.14	62.6	9.02	57.0	8.92	52.1
Area, in.^{2}					
91.6		83.3		76.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
6970	795	6170	704	5510	628
r_y, in.					
2.95		2.91		2.88	
r_x/r_y					
2.96		2.96		2.96	

^{h} Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W18

F_y = 65 ksi
F_u = 80 ksi

W18×						Shape	W18×					
234^h		211		192		lb/ft	234^h		211		192	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
2670	4010	2420	3640	2190	3290	0	1780	2680	1590	2390	1430	2150
2510	3780	2280	3430	2050	3090	6	1780	2680	1590	2390	1430	2150
2460	3700	2230	3350	2010	3020	7	1780	2680	1590	2390	1430	2150
2400	3600	2170	3260	1950	2940	8	1780	2680	1590	2390	1430	2150
2330	3500	2110	3170	1900	2850	9	1780	2670	1580	2380	1430	2140
2260	3390	2040	3070	1830	2760	10	1760	2640	1570	2350	1410	2120
2180	3270	1970	2960	1770	2660	11	1740	2620	1550	2330	1390	2090
2090	3150	1890	2840	1700	2550	12	1720	2590	1530	2300	1370	2060
2010	3020	1810	2720	1630	2440	13	1700	2560	1510	2270	1350	2040
1920	2880	1730	2600	1550	2330	14	1680	2530	1490	2240	1340	2010
1830	2750	1650	2470	1470	2210	15	1670	2500	1470	2220	1320	1980
1730	2610	1560	2350	1390	2100	16	1650	2480	1460	2190	1300	1950
1640	2470	1470	2220	1320	1980	17	1630	2450	1440	2160	1280	1930
1550	2320	1390	2090	1240	1860	18	1610	2420	1420	2130	1260	1900
1450	2180	1300	1960	1160	1740	19	1590	2390	1400	2100	1240	1870
1360	2050	1220	1830	1080	1630	20	1570	2360	1380	2080	1230	1840
1180	1780	1050	1580	934	1400	22	1540	2310	1350	2020	1190	1790
1010	1520	898	1350	793	1190	24	1500	2250	1310	1970	1150	1730
860	1290	765	1150	675	1020	26	1460	2200	1270	1910	1120	1680
742	1120	660	991	582	875	28	1420	2140	1230	1860	1080	1620
646	971	575	864	507	763	30	1390	2080	1200	1800	1040	1570
568	854	505	759	446	670	32	1350	2030	1160	1750	1010	1510
503	756	447	672	395	594	34	1310	1970	1120	1690	970	1460
449	675	399	600	352	530	36	1280	1920	1090	1630	934	1400
403	605	358	538	316	475	38	1240	1860	1050	1580	897	1350
364	546	323	486	285	429	40	1200	1810	1010	1520	860	1290
330	496	293	441	259	389	42	1160	1750	977	1470	815	1230
300	452	267	401	236	355	44	1130	1690	937	1410	776	1170
275	413	244	367	216	324	46	1090	1640	894	1340	740	1110
						48	1050	1580	854	1280	707	1060
						50	1010	1510	818	1230	677	1020

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2670	4010	2420	3640	2190	3290
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2060	3090	1870	2800	1690	2530
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
636	955	570	856	509	764
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
483	726	428	644	386	580

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.83	47.7	8.74	43.4	8.64	39.9
Area, in.^{2}					
68.6		62.3		56.2	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
4900	558	4330	493	3870	440
r_y, in.					
2.85		2.82		2.79	
r_x/r_y					
2.96		2.96		2.97	

^{h} Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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W18

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W18×						Shape	W18×					
175		158		143		lb/ft	175		158		143	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2000	3010	1800	2710	1630	2460	0	1290	1940	1150	1740	1040	1570
1880	2820	1690	2540	1530	2300	6	1290	1940	1150	1740	1040	1570
1830	2750	1650	2480	1490	2240	7	1290	1940	1150	1740	1040	1570
1780	2680	1600	2410	1450	2180	8	1290	1940	1150	1740	1040	1570
1730	2600	1550	2340	1410	2120	9	1280	1930	1150	1720	1030	1550
1670	2510	1500	2260	1360	2040	10	1260	1900	1130	1700	1020	1530
1610	2420	1450	2170	1310	1960	11	1250	1870	1110	1670	1000	1500
1540	2320	1390	2080	1250	1880	12	1230	1850	1090	1640	982	1480
1480	2220	1320	1990	1200	1800	13	1210	1820	1070	1620	965	1450
1410	2110	1260	1890	1140	1710	14	1190	1790	1060	1590	947	1420
1340	2010	1200	1800	1080	1620	15	1170	1770	1040	1560	930	1400
1260	1900	1130	1700	1020	1530	16	1160	1740	1020	1540	912	1370
1190	1790	1060	1600	958	1440	17	1140	1710	1000	1510	895	1350
1120	1680	998	1500	898	1350	18	1120	1680	986	1480	878	1320
1050	1570	933	1400	838	1260	19	1100	1660	968	1460	860	1290
975	1470	869	1310	780	1170	20	1080	1630	951	1430	843	1270
838	1260	746	1120	668	1000	22	1050	1580	915	1380	808	1210
710	1070	630	947	563	846	24	1010	1520	880	1320	773	1160
605	909	537	807	480	721	26	977	1470	844	1270	738	1110
521	784	463	696	414	622	28	941	1410	809	1220	703	1060
454	683	403	606	360	542	30	904	1360	773	1160	669	1000
399	600	354	533	317	476	32	868	1310	738	1110	631	948
354	531	314	472	281	422	34	832	1250	701	1050	587	883
315	474	280	421	250	376	36	796	1200	657	988	549	826
283	425	251	378	225	338	38	754	1130	618	929	516	776
255	384	227	341	203	305	40	713	1070	584	877	487	732
232	348	206	309	184	276	42	676	1020	553	831	461	693
211	317	187	282	168	252	44	643	966	525	790	438	658
193	290					46	612	920	500	752	417	626
						48	585	879	478	718	398	598
						50	560	842	457	687	380	572

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2000	3010	1800	2710	1630	2460
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1540	2310	1390	2080	1260	1890
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
463	694	415	622	370	555
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
344	517	307	462	277	416

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.55	36.9	8.49	33.9	8.43	31.6
Area, in.^{2}					
51.4		46.3		42.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
3450	391	3060	347	2750	311
r_y, in.					
2.76		2.74		2.72	
r_x/r_y					
2.97		2.96		2.97	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W18

F_y = 65 ksi
F_u = 80 ksi

W18×						Shape	W18×					
130		119		106		lb/ft	130		119		106	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
1490	2240	1370	2050	1210	1820	0	941	1410	850	1280	746	1120
1390	2090	1280	1920	1130	1700	6	941	1410	850	1280	746	1120
1360	2040	1250	1870	1100	1650	7	941	1410	850	1280	746	1120
1320	1990	1210	1820	1070	1610	8	941	1410	850	1280	746	1120
1280	1920	1170	1760	1030	1560	9	930	1400	839	1260	734	1100
1240	1860	1130	1700	998	1500	10	913	1370	822	1240	718	1080
1190	1790	1090	1630	958	1440	11	896	1350	805	1210	702	1060
1140	1710	1040	1560	916	1380	12	879	1320	789	1190	687	1030
1090	1630	992	1490	873	1310	13	862	1300	772	1160	671	1010
1030	1550	943	1420	828	1250	14	845	1270	755	1140	655	984
977	1470	893	1340	783	1180	15	828	1240	739	1110	639	961
922	1390	842	1270	738	1110	16	811	1220	722	1090	623	937
866	1300	791	1190	692	1040	17	794	1190	705	1060	608	913
811	1220	740	1110	647	972	18	777	1170	689	1040	592	890
757	1140	690	1040	602	905	19	760	1140	672	1010	576	866
703	1060	641	964	558	839	20	743	1120	655	985	560	842
601	903	547	822	475	713	22	709	1060	622	935	529	795
506	760	460	692	399	599	24	675	1010	589	885	497	747
431	648	392	589	340	511	26	640	963	555	835	465	700
372	559	338	508	293	440	28	606	912	520	782	421	633
324	487	295	443	255	384	30	568	854	476	716	384	578
285	428	259	389	224	337	32	525	789	439	660	353	531
252	379	229	345	199	299	34	488	734	407	612	327	492
225	338	205	307	177	266	36	456	686	380	571	305	458
202	303	184	276	159	239	38	428	644	356	535	285	428
182	274	166	249	144	216	40	404	607	335	504	268	402
165	248	150	226	130	196	42	382	574	316	476	252	379
151	226	137	206	119	178	44	362	544	300	451	239	359
						46	345	518	285	428	227	341
						48	329	494	272	408	216	325
						50	314	472	259	390	206	310

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1490	2240	1370	2050	1210	1820
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1150	1720	1050	1580	933	1400
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
336	504	324	485	287	430
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
249	374	224	337	196	295

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.36	29.5	8.33	27.9	8.24	26.1
Area, in.^{2}					
38.3		35.1		31.1	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
2460	278	2190	253	1910	220
r_y, in.					
2.70		2.69		2.66	
r_x/r_y					
2.97		2.94		2.95	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

W18

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W18×						Shape	W18×					
97		86^c		76^c		lb/ft	97		86		76^f	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1110	1670	972	1460	835	1250	0	684	1030	603	907	527	793
1030	1550	912	1370	783	1180	6	684	1030	603	907	527	793
1010	1520	892	1340	765	1150	7	684	1030	603	907	527	793
979	1470	868	1300	744	1120	8	684	1030	603	907	527	793
947	1420	839	1260	722	1080	9	672	1010	591	888	516	776
913	1370	808	1210	698	1050	10	657	988	577	867	503	756
876	1320	775	1160	672	1010	11	642	965	562	845	490	736
838	1260	741	1110	645	969	12	627	942	548	823	476	716
798	1200	705	1060	616	926	13	611	919	533	802	463	696
757	1140	668	1000	585	880	14	596	896	519	780	450	676
715	1080	631	948	552	830	15	581	873	505	759	436	656
674	1010	593	892	519	780	16	566	850	490	737	423	636
632	949	556	835	486	730	17	551	827	476	715	410	616
590	887	519	780	453	680	18	535	805	462	694	396	596
549	825	482	724	420	632	19	520	782	447	672	383	575
509	765	446	671	389	584	20	505	759	433	650	370	555
432	649	377	567	328	492	22	474	713	404	607	343	515
363	545	317	477	275	414	24	444	667	374	562	306	461
309	464	270	406	235	353	26	406	610	332	499	271	407
266	400	233	350	202	304	28	366	550	298	448	242	364
232	349	203	305	176	265	30	333	501	270	406	219	329
204	307	178	268	155	233	32	306	460	247	372	200	300
181	272	158	237	137	206	34	283	425	228	343	183	276
161	242	141	212	122	184	36	263	395	211	318	170	255
145	217	126	190	110	165	38	245	369	197	296	158	237
131	196	114	172	99.1	149	40	230	346	184	277	147	221
118	178	104	156	89.9	135	42	217	326	173	261	138	208
108	162					44	205	308	164	246	130	196
						46	194	292	155	233	123	185
						48	185	278	147	221	117	175
						50	176	265	140	211	111	167

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1110	1670	985	1480	868	1300
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
855	1280	759	1140	669	1000
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
259	388	230	344	201	302
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
179	270	157	236	136	205

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.21	25.1	8.15	23.9	8.18	22.8
Area, in.^{2}					
28.5		25.3		22.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1750	201	1530	175	1330	152
r_y, in.					
2.65		2.63		2.61	
r_x/r_y					
2.95		2.95		2.96	

[c] Shape is slender for compression with F_y = 65 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 65 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W18

F_y = 65 ksi
F_u = 80 ksi

W18×						Shape	W18×					
71^c		65^c		60^c		lb/ft	71		65		60	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Design	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
807	1210	719	1080	649	975	0	474	712	431	648	399	600
686	1030	619	931	557	838	6	461	693	419	630	387	582
645	969	586	882	528	793	7	445	668	404	607	372	559
601	903	547	822	495	745	8	428	643	388	583	357	536
554	833	504	758	461	693	9	411	618	372	559	341	513
507	761	460	692	422	634	10	394	593	356	535	326	490
459	689	416	626	381	573	11	378	568	340	512	311	468
411	618	373	560	341	512	12	361	542	325	488	296	445
365	549	331	497	302	454	13	344	517	309	464	281	422
322	483	291	437	265	398	14	327	492	293	441	266	399
280	421	253	380	230	346	15	311	467	277	417	251	377
246	370	222	334	203	304	16	294	442	259	389	230	345
218	328	197	296	179	270	17	272	408	236	355	209	314
195	292	176	264	160	241	18	250	376	217	326	192	288
175	262	158	237	144	216	19	232	348	201	302	177	266
158	237	142	214	130	195	20	216	324	187	281	164	247
130	196	118	177	107	161	22	190	285	164	246	144	216
109	165	98.9	149	90.0	135	24	169	254	145	219	127	192
93.3	140	84.2	127	76.7	115	26	153	230	131	197	115	172
80.4	121	72.6	109	66.1	99.4	28	139	209	119	179	104	156
						30	128	192	109	164	95.2	143
						32	118	178	101	152	87.9	132
						34	110	166	93.9	141	81.6	123
						36	103	155	87.8	132	76.1	114
						38	96.9	146	82.4	124	71.4	107
						40	91.4	137	77.6	117	67.2	101
						42	86.6	130	73.4	110	63.5	95.5
						44	82.2	124	69.7	105	60.2	90.5
						46	78.2	118	66.3	99.6	57.3	86.1
						48	74.7	112	63.2	95.0	54.6	82.1
						50	71.4	107	60.4	90.8	52.2	78.4

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
813	1220	743	1120	685	1030
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
627	941	573	860	528	792
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
238	357	215	323	196	295
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
80.1	120	73.0	110	66.8	100

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.27	16.3	5.24	15.7	5.20	15.4
Area, in.^{2}					
20.9		19.1		17.6	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1170	60.3	1070	54.8	984	50.1
r_y, in.					
1.70		1.69		1.68	
r_x/r_y					
4.41		4.43		4.45	

^{c} Shape is slender for compression with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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W18

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W18×						Shape	W18×					
55^c		50^c		46^c		lb/ft	55		50		46	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
587	882	519	780	474	712	0	363	546	328	492	294	442
504	757	444	667	368	553	6	351	528	316	474	264	397
477	716	419	630	336	505	7	337	506	302	454	249	374
447	672	393	590	303	455	8	322	485	289	434	234	351
416	625	365	548	269	404	9	308	463	275	414	219	329
384	577	336	505	231	347	10	294	441	262	393	204	306
348	523	307	461	194	291	11	279	420	248	373	189	283
311	467	277	417	163	245	12	265	398	235	353	170	255
275	413	245	368	139	209	13	250	376	221	333	149	224
241	362	213	320	120	180	14	236	354	208	312	133	200
210	315	186	279	104	157	15	220	331	189	284	119	179
184	277	163	245	91.6	138	16	198	298	170	256	108	163
163	245	145	217	81.1	122	17	180	271	154	232	99.0	149
146	219	129	194	72.4	109	18	165	248	141	212	91.1	137
131	196	116	174	65.0	97.6	19	152	228	130	195	84.4	127
118	177	104	157	58.6	88.1	20	141	212	120	180	78.5	118
97.4	146	86.3	130			22	123	184	104	156	69.0	104
81.9	123	72.5	109			24	108	163	91.5	137	61.5	92.4
69.8	105	61.8	92.9			26	97.1	146	81.7	123	55.4	83.3
						28	87.9	132	73.8	111	50.5	75.9
						30	80.4	121	67.3	101	46.4	69.7
						32	74.0	111	61.8	92.9	42.9	64.5
						34	68.6	103	57.2	85.9	39.9	60.0
						36	63.9	96.1	53.2	79.9	37.4	56.2
						38	59.9	90.0	49.7	74.8	35.1	52.8
						40	56.3	84.6	46.7	70.2	33.1	49.8
						42	53.2	79.9	44.0	66.2	31.3	47.1
						44	50.3	75.7	41.7	62.6	29.7	44.7
						46	47.8	71.9	39.5	59.4	28.3	42.5
						48	45.6	68.5	37.6	56.6	27.0	40.6
						50	43.5	65.4	35.9	54.0	25.8	38.8

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
631	948	572	860	525	790
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
486	729	441	662	405	608
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
184	275	166	249	169	254
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
60.0	90.2	53.8	80.9	37.9	57.0

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.17	14.9	5.11	14.4	4.00	11.6
Area, in.^{2}					
16.2		14.7		13.5	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
890	44.9	800	40.1	712	22.5
r_y, in.					
1.67		1.65		1.29	
r_x/r_y					
4.44		4.47		5.62	

^{c} Shape is slender for compression with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W18–W16

F_y = 65 ksi
F_u = 80 ksi

W-Shapes

W18×				W16×		Shape	W18×				W16×	
40^c		35^c		100		lb/ft	40^v		35^v		100	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
399	599	338	508	1140	1720	0	254	382	216	324	642	965
307	461	256	384	1060	1590	6	226	340	188	283	642	965
279	420	231	347	1030	1550	7	213	320	176	264	642	965
251	377	206	310	996	1500	8	199	299	163	246	639	961
222	333	181	272	960	1440	9	185	279	151	227	626	941
193	291	156	235	921	1380	10	172	258	139	208	613	922
164	247	132	199	880	1320	11	158	238	123	185	600	902
138	207	111	167	837	1260	12	138	208	106	159	587	882
118	177	94.7	142	793	1190	13	121	182	91.9	138	574	863
101	152	81.6	123	747	1120	14	107	161	81.1	122	561	843
88.3	133	71.1	107	702	1050	15	95.9	144	72.4	109	548	823
77.6	117	62.5	93.9	656	986	16	86.6	130	65.2	97.9	535	804
68.7	103	55.4	83.2	611	918	17	78.9	119	59.2	88.9	522	784
61.3	92.2	49.4	74.2	566	851	18	72.4	109	54.1	81.3	509	764
55.0	82.7	44.3	66.6	522	785	19	66.8	100	49.8	74.8	496	745
49.7	74.6	40.0	60.1	480	721	20	62.0	93.2	46.1	69.2	482	725
				399	600	22	54.2	81.4	40.0	60.1	456	686
				336	504	24	48.0	72.2	35.3	53.1	430	647
				286	430	26	43.2	64.9	31.6	47.5	404	607
				247	371	28	39.2	58.9	28.6	43.0	370	557
				215	323	30	35.9	53.9	26.1	39.2	340	511
				189	284	32	33.1	49.8	24.0	36.1	314	472
				167	251	34	30.7	46.2	22.2	33.4	292	439
				149	224	36	28.7	43.1	20.7	31.1	273	410
				134	201	38	26.9	40.4	19.4	29.1	256	385
				121	182	40	25.3	38.1	18.2	27.4	242	363
						42	23.9	36.0	17.2	25.8	228	343
						44	22.7	34.1	16.3	24.4	217	326
						46	21.6	32.4	15.4	23.2	206	310
						48	20.6	30.9	14.7	22.1	197	296
						50	19.6	29.5	14.0	21.1	188	283

Properties

W18×40		W18×35		W16×100	
Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
459	690	401	603	1140	1720
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
354	531	309	464	882	1320
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
132	198	124	186	259	388
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
32.4	48.8	26.1	39.3	178	268

W18×40		W18×35		W16×100	
Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
3.93	11.2	3.78	10.6	7.78	26.5
Area, in.^{2}					
11.8		10.3		29.4	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
612	19.1	510	15.3	1490	186
r_y, in.					
1.27		1.22		2.51	
r_x/r_y					
5.68		5.77		2.83	

[c] Shape is slender for compression with F_y = 65 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W16

F_y = 65 ksi
F_u = 80 ksi

W16×						Shape		W16×					
89		77		67^c		lb/ft		89		77		67	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
1020	1530	880	1320	743	1120	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	568	853	487	731	422	634
942	1420	811	1220	691	1040		6	568	853	487	731	422	634
915	1380	788	1180	673	1010		7	568	853	487	731	422	634
885	1330	762	1150	652	980		8	564	848	482	725	417	628
853	1280	733	1100	630	947		9	551	829	471	707	407	611
818	1230	703	1060	606	911		10	539	809	459	690	396	595
781	1170	671	1010	580	872		11	526	790	447	672	385	578
742	1120	637	957	551	828		12	513	771	435	654	374	562
702	1060	602	905	521	782		13	500	752	423	636	363	545
662	994	567	852	490	736		14	488	733	412	619	352	529
621	933	531	798	459	689		15	475	714	400	601	341	512
580	871	495	744	428	643		16	462	695	388	583	330	496
539	810	460	691	397	596		17	450	676	376	565	319	479
499	750	425	639	367	551		18	437	657	364	548	308	463
460	691	391	588	337	507		19	424	638	352	530	297	446
422	634	359	539	309	464		20	412	619	341	512	286	430
350	527	297	447	256	384		22	386	580	317	476	262	395
294	442	250	376	215	323		24	361	542	287	432	230	346
251	377	213	320	183	275		26	328	493	257	386	205	308
216	325	184	276	158	237		28	298	447	232	349	184	277
188	283	160	240	138	207		30	272	409	212	318	167	252
166	249	141	211	121	182		32	251	377	194	292	153	230
147	220	124	187	107	161		34	233	350	180	270	141	213
131	197	111	167	95.5	144		36	217	327	167	252	131	197
117	176	99.7	150	85.7	129		38	204	306	157	235	122	184
106	159	89.9	135	77.4	116		40	192	288	147	221	115	172
							42	181	272	139	209	108	162
							44	172	258	131	197	102	153
							46	163	245	125	187	96.7	145
							48	156	234	119	178	91.9	138
							50	149	223	113	170	87.5	132

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1020	1530	880	1320	763	1150
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
786	1180	678	1020	588	882
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
229	344	195	293	167	251
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
156	234	133	200	115	173

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.71	24.7	7.65	23.1	7.62	21.8
Area, in.^{2}					
26.2		22.6		19.6	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1300	163	1110	138	954	119
r_y, in.					
2.49		2.47		2.46	
r_x/r_y					
2.83		2.83		2.83	

^{c} Shape is slender for compression with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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W16

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W16×						Shape	W16×					
57^c		50^c		45^c		lb/ft	57		50		45	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
646	971	547	823	484	727	0	341	512	298	449	267	401
539	811	462	694	406	610	6	327	492	285	429	254	382
503	756	434	653	381	573	7	315	473	273	411	243	365
464	698	405	608	355	533	8	302	454	261	393	232	348
424	637	369	555	327	491	9	289	435	249	375	221	331
383	576	333	500	297	447	10	276	415	237	357	209	315
342	515	297	447	264	397	11	264	396	225	339	198	298
303	455	262	394	233	350	12	251	377	213	321	187	281
265	398	229	344	203	304	13	238	358	201	303	176	264
229	344	198	297	175	262	14	225	339	189	284	164	246
200	300	172	259	152	229	15	213	320	173	260	147	220
175	264	152	228	134	201	16	195	293	157	236	133	199
155	233	134	202	118	178	17	179	269	143	216	121	181
139	208	120	180	106	159	18	165	248	132	198	111	166
124	187	107	162	94.8	142	19	153	230	122	183	102	154
112	169	97.0	146	85.5	129	20	143	215	114	171	94.9	143
92.8	139	80.1	120	70.7	106	22	126	189	99.6	150	82.9	125
77.9	117	67.3	101	59.4	89.3	24	112	169	88.6	133	73.5	111
66.4	99.8	57.4	86.2	50.6	76.1	26	102	153	79.9	120	66.1	99.3
						28	92.9	140	72.7	109	60.0	90.2
						30	85.5	128	66.8	100	55.0	82.6
						32	79.2	119	61.7	92.7	50.7	76.2
						34	73.8	111	57.4	86.3	47.1	70.8
						36	69.1	104	53.7	80.6	43.9	66.0
						38	65.0	97.7	50.4	75.7	41.2	61.9
						40	61.3	92.2	47.5	71.4	38.8	58.3
						42	58.1	87.3	44.9	67.5	36.7	55.1
						44	55.2	83.0	42.6	64.1	34.8	52.3
						46	52.6	79.0	40.6	61.0	33.1	49.7
						48	50.2	75.4	38.7	58.2	31.5	47.4
						50	48.0	72.2	37.0	55.6	30.1	45.2

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
654	983	572	860	518	778
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
504	756	441	662	399	599
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
183	275	161	242	144	217
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
61.3	92.1	52.9	79.5	47.0	70.7

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
4.96	15.3	4.93	14.4	4.86	13.9
Area, in.^{2}					
16.8		14.7		13.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
758	43.1	659	37.2	586	32.8
r_y, in.					
1.60		1.59		1.57	
r_x/r_y					
4.20		4.20		4.24	

[c] Shape is slender for compression with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W16

F_y = 65 ksi
F_u = 80 ksi

W16×						Shape		W16×					
40^c		36^c		31^c		lb/ft		40		$36^{f,v}$		31^v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
416	626	368	553	307	462	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	237	356	207	311	175	263
349	525	306	459	226	339		6	225	338	195	294	150	226
328	493	286	430	202	304		7	215	323	186	279	140	210
305	458	265	398	178	267		8	204	307	176	265	130	195
281	422	243	365	154	231		9	194	292	167	251	119	179
256	385	220	331	130	196		10	184	276	157	236	109	163
231	348	198	297	108	162		11	173	260	148	222	93.1	140
206	310	176	264	90.6	136		12	163	245	138	208	80.4	121
180	270	151	227	77.2	116		13	153	229	129	194	70.5	106
155	233	130	196	66.6	100		14	139	209	114	171	62.6	94.0
135	203	114	171	58.0	87.1		15	124	186	101	152	56.1	84.4
119	178	99.9	150	51.0	76.6		16	112	168	90.8	136	50.8	76.4
105	158	88.5	133	45.1	67.8		17	101	152	82.2	124	46.4	69.7
93.7	141	78.9	119	40.3	60.5		18	92.8	139	75.0	113	42.6	64.0
84.1	126	70.8	106	36.1	54.3		19	85.4	128	68.8	103	39.4	59.2
75.9	114	63.9	96.1				20	79.0	119	63.6	95.5	36.6	55.0
62.7	94.3	52.8	79.4				22	68.7	103	55.0	82.6	32.0	48.2
52.7	79.2	44.4	66.7				24	60.7	91.2	48.4	72.7	28.5	42.8
44.9	67.5						26	54.3	81.7	43.1	64.8	25.6	38.5
							28	49.2	73.9	38.9	58.4	23.3	35.1
							30	44.9	67.5	35.4	53.2	21.4	32.1
							32	41.3	62.1	32.5	48.8	19.8	29.7
							34	38.3	57.5	30.0	45.1	18.4	27.6
							36	35.7	53.6	27.9	41.9	17.2	25.8
							38	33.4	50.2	26.0	39.1	16.1	24.2
							40	31.4	47.2	24.4	36.7	15.2	22.8
							42	29.6	44.5	23.0	34.6	14.4	21.6
							44	28.0	42.1	21.8	32.7	13.6	20.5
							46	26.6	40.0	20.6	31.0	13.0	19.5
							48	25.4	38.1	19.6	29.5	12.4	18.6
							50	24.2	36.4	18.7	28.1	11.8	17.7

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
459	690	413	620	355	534
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
354	531	318	477	274	411
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
127	190	110	165	102	153
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
41.2	61.9	34.9	52.4	22.8	34.3

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
4.86	13.5	4.77	13.1	3.62	10.1
Area, in.^{2}					
11.8		10.6		9.13	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
518	28.9	448	24.5	375	12.4
r_y, in.					
1.57		1.52		1.17	
r_x/r_y					
4.22		4.28		5.48	

[c] Shape is slender for compression with F_y = 65 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 65 ksi.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W16–W14

F_y = 65 ksi
F_u = 80 ksi

W16×		W14×				Shape		W16×		W14×			
26^c		873^h		808^h		lb/ft		26^v		873^h		808^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
248	372	10000	15000	9260	13900	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	143	215	6580	9900	5940	8920
178	267	9800	14700	9070	13600		6	120	181	6580	9900	5940	8920
158	237	9730	14600	9000	13500		7	111	167	6580	9900	5940	8920
138	207	9640	14500	8920	13400		8	102	153	6580	9900	5940	8920
118	177	9550	14400	8830	13300		9	93.0	140	6580	9900	5940	8920
99.1	149	9450	14200	8740	13100		10	81.5	122	6580	9900	5940	8920
83.1	125	9340	14000	8630	13000		11	68.8	103	6580	9900	5940	8920
69.8	105	9210	13800	8510	12800		12	59.1	88.8	6580	9900	5940	8920
59.5	89.4	9080	13700	8390	12600		13	51.5	77.5	6580	9900	5940	8920
51.3	77.1	8950	13400	8260	12400		14	45.5	68.4	6580	9900	5940	8920
44.7	67.2	8800	13200	8120	12200		15	40.6	61.1	6580	9900	5940	8920
39.3	59.0	8640	13000	7970	12000		16	36.6	55.0	6570	9880	5920	8900
34.8	52.3	8480	12800	7820	11800		17	33.2	50.0	6560	9860	5910	8880
31.0	46.6	8320	12500	7660	11500		18	30.4	45.7	6550	9840	5900	8860
		8140	12200	7500	11300		19	28.0	42.1	6530	9820	5890	8850
		7960	12000	7330	11000		20	25.9	39.0	6520	9800	5870	8830
		7590	11400	6970	10500		22	22.6	33.9	6500	9760	5850	8790
		7200	10800	6610	9930		24	19.9	30.0	6470	9720	5820	8750
		6800	10200	6230	9360		26	17.8	26.8	6440	9680	5800	8710
		6400	9620	5850	8790		28	16.2	24.3	6420	9640	5770	8680
		5990	9000	5460	8210		30	14.8	22.2	6390	9610	5750	8640
		5580	8390	5080	7630		32	13.6	20.4	6360	9570	5720	8600
		5180	7780	4700	7070		34	12.6	18.9	6340	9530	5700	8560
		4780	7180	4330	6510		36	11.7	17.6	6310	9490	5670	8530
		4390	6600	3970	5970		38	11.0	16.5	6290	9450	5650	8490
		4020	6040	3620	5450		40	10.3	15.5	6260	9410	5620	8450
		3650	5490	3290	4940		42	9.74	14.6	6230	9370	5600	8410
		3330	5000	2990	4500		44	9.22	13.9	6210	9330	5570	8380
		3040	4570	2740	4120		46	8.76	13.2	6180	9290	5550	8340
		2800	4200	2520	3780		48	8.34	12.5	6160	9250	5520	8300
		2580	3870	2320	3480		50	7.96	12.0	6130	9210	5500	8270

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
299	449	10000	15000	9260	13900
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
230	346	7710	11600	7140	10700
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
86.6	130	2420	3630	2220	3330
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
17.8	26.7	3310	4970	3020	4530

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
3.47	9.64	15.2	253	15.0	238
Area, in.^{2}					
7.68		257		238	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
301	9.59	18100	6170	15900	5550
r_y, in.					
1.12		4.90		4.83	
r_x/r_y					
5.59		1.71		1.69	

[c] Shape is slender for compression with F_y = 65 ksi.

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W14

F_y = 65 ksi
F_u = 80 ksi

W14×						Shape		W14×					
730^h		665^h		605^h		lb/ft		730^h		665^h		605^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
8370	12600	7630	11500	6930	10400	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	5380	8090	4800	7220	4280	6440
8180	12300	7450	11200	6770	10200		6	5380	8090	4800	7220	4280	6440
8120	12200	7390	11100	6710	10100		7	5380	8090	4800	7220	4280	6440
8040	12100	7320	11000	6640	9980		8	5380	8090	4800	7220	4280	6440
7960	12000	7240	10900	6570	9870		9	5380	8090	4800	7220	4280	6440
7860	11800	7150	10800	6480	9750		10	5380	8090	4800	7220	4280	6440
7760	11700	7060	10600	6400	9610		11	5380	8090	4800	7220	4280	6440
7650	11500	6960	10500	6300	9470		12	5380	8090	4800	7220	4280	6440
7530	11300	6850	10300	6200	9310		13	5380	8090	4800	7220	4280	6440
7410	11100	6730	10100	6090	9150		14	5380	8090	4800	7220	4280	6440
7270	10900	6600	9930	5970	8970		15	5380	8080	4790	7200	4270	6420
7140	10700	6470	9730	5850	8790		16	5370	8060	4780	7180	4260	6400
6990	10500	6340	9530	5720	8600		17	5350	8050	4770	7170	4250	6380
6840	10300	6200	9310	5590	8410		18	5340	8030	4760	7150	4240	6370
6680	10000	6050	9100	5460	8200		19	5330	8010	4740	7130	4220	6350
6520	9810	5900	8870	5320	7990		20	5320	7990	4730	7110	4210	6330
6190	9310	5590	8410	5030	7560		22	5290	7950	4710	7070	4190	6300
5850	8790	5270	7920	4730	7120		24	5270	7910	4680	7040	4170	6260
5490	8260	4950	7430	4430	6660		26	5240	7880	4660	7000	4140	6230
5140	7720	4610	6940	4130	6200		28	5210	7840	4630	6970	4120	6190
4780	7180	4280	6440	3820	5740		30	5190	7800	4610	6930	4100	6160
4420	6650	3960	5950	3520	5290		32	5160	7760	4590	6890	4070	6120
4080	6130	3640	5460	3230	4850		34	5140	7720	4560	6860	4050	6090
3740	5620	3320	4990	2940	4420		36	5110	7690	4540	6820	4030	6050
3410	5120	3020	4540	2660	4000		38	5090	7650	4510	6780	4000	6020
3090	4640	2730	4100	2400	3610		40	5060	7610	4490	6750	3980	5980
2800	4210	2480	3720	2180	3280		42	5040	7570	4460	6710	3960	5940
2550	3830	2260	3390	1990	2990		44	5010	7540	4440	6670	3930	5910
2330	3510	2060	3100	1820	2730		46	4990	7500	4420	6640	3910	5870
2140	3220	1900	2850	1670	2510		48	4960	7460	4390	6600	3890	5840
1970	2970	1750	2630	1540	2310		50	4940	7420	4370	6560	3860	5800

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
8370	12600	7630	11500	6930	10400
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
6450	9680	5880	8820	5340	8010
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1790	2680	1590	2380	1410	2120
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
2650	3980	2370	3560	2110	3180

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
14.5	212	14.3	195	14.1	178
Area, in.^{2}					
215		196		178	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
14300	4720	12400	4170	10800	3680
r_y, in.					
4.69		4.62		4.55	
r_x/r_y					
1.74		1.73		1.71	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

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W14

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W14×						Shape		W14×					
550^h		500^h		455^h		lb/ft		550^h		500^h		455^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
6310	9480	5720	8600	5220	7840	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	3830	5750	3410	5120	3040	4560
6150	9250	5580	8390	5080	7640		6	3830	5750	3410	5120	3040	4560
6100	9170	5530	8310	5040	7570		7	3830	5750	3410	5120	3040	4560
6040	9070	5470	8220	4980	7490		8	3830	5750	3410	5120	3040	4560
5970	8970	5410	8130	4920	7400		9	3830	5750	3410	5120	3040	4560
5890	8850	5340	8020	4860	7300		10	3830	5750	3410	5120	3040	4560
5810	8730	5260	7900	4780	7190		11	3830	5750	3410	5120	3040	4560
5720	8590	5170	7780	4710	7070		12	3830	5750	3410	5120	3040	4560
5620	8450	5090	7640	4620	6950		13	3830	5750	3410	5120	3040	4560
5520	8300	4990	7500	4530	6820		14	3830	5750	3400	5110	3030	4560
5410	8130	4890	7350	4440	6680		15	3810	5730	3390	5100	3020	4540
5300	7970	4790	7190	4340	6530		16	3800	5720	3380	5080	3010	4520
5180	7790	4680	7030	4240	6380		17	3790	5700	3370	5060	3000	4510
5060	7610	4560	6860	4140	6220		18	3780	5680	3360	5050	2990	4490
4930	7420	4450	6690	4030	6060		19	3770	5670	3350	5030	2980	4480
4810	7220	4330	6510	3920	5890		20	3760	5650	3340	5020	2970	4460
4540	6820	4080	6140	3690	5550		22	3740	5610	3310	4980	2950	4430
4260	6410	3830	5750	3460	5200		24	3710	5580	3290	4950	2930	4400
3980	5990	3570	5370	3220	4840		26	3690	5550	3270	4920	2900	4360
3700	5570	3310	4980	2980	4480		28	3670	5510	3250	4880	2880	4330
3420	5140	3050	4590	2740	4120		30	3640	5480	3230	4850	2860	4300
3150	4730	2800	4210	2510	3780		32	3620	5440	3200	4820	2840	4270
2880	4320	2550	3840	2290	3440		34	3600	5410	3180	4780	2820	4240
2620	3930	2320	3480	2070	3110		36	3580	5370	3160	4750	2800	4210
2360	3550	2090	3130	1860	2790		38	3550	5340	3140	4720	2780	4170
2130	3200	1880	2830	1680	2520		40	3530	5310	3120	4690	2760	4140
1930	2900	1710	2570	1520	2290		42	3510	5270	3100	4650	2730	4110
1760	2650	1560	2340	1390	2080		44	3480	5240	3070	4620	2710	4080
1610	2420	1420	2140	1270	1910		46	3460	5200	3050	4590	2690	4050
1480	2220	1310	1960	1160	1750		48	3440	5170	3030	4550	2670	4010
1360	2050	1200	1810	1070	1610		50	3420	5130	3010	4520	2650	3980

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
6310	9480	5720	8600	5220	7840
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4860	7290	4410	6620	4020	6030
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1250	1870	1120	1670	998	1500
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
1890	2840	1690	2540	1520	2280

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
13.9	164	13.7	151	13.6	138
Area, in.^{2}					
162		147		134	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
9430	3250	8210	2880	7190	2560
r_y, in.					
4.49		4.43		4.38	
r_x/r_y					
1.70		1.69		1.67	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W14

F_y = 65 ksi
F_u = 80 ksi

W14×						Shape		W14×					
426^h		398^h		370^h		lb/ft		426^h		398^h		370^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
4870	7310	4550	6840	4240	6380	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	2820	4240	2600	3900	2390	3590
4740	7120	4430	6670	4130	6210		6	2820	4240	2600	3900	2390	3590
4700	7060	4390	6600	4090	6150		7	2820	4240	2600	3900	2390	3590
4640	6980	4340	6530	4040	6080		8	2820	4240	2600	3900	2390	3590
4590	6890	4290	6450	3990	6000		9	2820	4240	2600	3900	2390	3590
4520	6800	4230	6360	3940	5920		10	2820	4240	2600	3900	2390	3590
4460	6700	4170	6260	3870	5820		11	2820	4240	2600	3900	2390	3590
4380	6590	4100	6160	3810	5720		12	2820	4240	2600	3900	2390	3590
4300	6470	4020	6040	3740	5620		13	2820	4240	2600	3900	2390	3590
4220	6340	3940	5920	3660	5500		14	2810	4230	2590	3890	2380	3580
4130	6210	3860	5800	3580	5390		15	2800	4210	2580	3880	2370	3560
4040	6070	3770	5670	3500	5260		16	2790	4200	2570	3860	2360	3550
3940	5930	3680	5530	3420	5130		17	2780	4180	2560	3850	2350	3530
3840	5780	3590	5390	3330	5000		18	2770	4160	2550	3830	2340	3520
3740	5630	3490	5250	3240	4860		19	2760	4150	2540	3820	2330	3500
3640	5470	3390	5100	3140	4720		20	2750	4130	2530	3800	2320	3490
3420	5140	3190	4790	2950	4430		22	2730	4100	2510	3770	2300	3460
3200	4810	2980	4480	2750	4140		24	2710	4070	2490	3740	2280	3430
2980	4470	2770	4160	2550	3840		26	2690	4040	2470	3710	2260	3400
2750	4140	2560	3840	2360	3540		28	2670	4010	2450	3680	2240	3370
2530	3800	2350	3530	2160	3240		30	2650	3980	2430	3650	2220	3340
2310	3470	2140	3220	1970	2960		32	2620	3940	2410	3620	2200	3310
2100	3160	1940	2920	1780	2680		34	2600	3910	2390	3590	2180	3280
1900	2850	1750	2630	1600	2410		36	2580	3880	2370	3560	2160	3250
1700	2560	1570	2360	1440	2160		38	2560	3850	2350	3530	2140	3220
1540	2310	1420	2130	1300	1950		40	2540	3820	2330	3490	2120	3190
1390	2090	1290	1930	1180	1770		42	2520	3790	2300	3460	2100	3160
1270	1910	1170	1760	1070	1610		44	2500	3760	2280	3430	2080	3120
1160	1750	1070	1610	980	1470		46	2480	3720	2260	3400	2060	3090
1070	1600	985	1480	900	1350		48	2460	3690	2240	3370	2040	3060
983	1480	907	1360	830	1250		50	2440	3660	2220	3340	2020	3030

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4870	7310	4550	6840	4240	6380
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3750	5630	3510	5270	3270	4910
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
914	1370	842	1260	773	1160
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
1410	2120	1300	1960	1200	1800

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
13.4	130	13.4	122	13.2	114
Area, in.^{2}					
125		117		109	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
6600	2360	6000	2170	5440	1990
r_y, in.					
4.34		4.31		4.27	
r_x/r_y					
1.67		1.66		1.66	

^{h} Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

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W14

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W14×						Shape	W14×					
342^h		311^h		283^h		lb/ft	342^h		311^h		283^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
3930	5910	3560	5350	3240	4870	0	2180	3280	1960	2940	1760	2640
3820	5750	3460	5200	3150	4740	6	2180	3280	1960	2940	1760	2640
3790	5690	3420	5150	3120	4690	7	2180	3280	1960	2940	1760	2640
3740	5630	3390	5090	3080	4630	8	2180	3280	1960	2940	1760	2640
3700	5560	3340	5020	3040	4570	9	2180	3280	1960	2940	1760	2640
3640	5480	3290	4950	3000	4500	10	2180	3280	1960	2940	1760	2640
3590	5390	3240	4870	2950	4430	11	2180	3280	1960	2940	1760	2640
3520	5290	3180	4780	2890	4350	12	2180	3280	1960	2940	1760	2640
3460	5200	3120	4690	2840	4270	13	2180	3280	1960	2940	1760	2640
3390	5090	3060	4590	2780	4180	14	2170	3260	1950	2930	1750	2630
3310	4980	2990	4490	2720	4080	15	2160	3250	1940	2910	1740	2610
3230	4860	2920	4380	2650	3980	16	2150	3230	1930	2900	1730	2600
3150	4740	2840	4270	2580	3880	17	2140	3220	1920	2880	1720	2580
3070	4620	2770	4160	2510	3780	18	2130	3200	1910	2870	1710	2570
2990	4490	2690	4040	2440	3670	19	2120	3190	1900	2850	1700	2560
2900	4360	2610	3920	2370	3560	20	2110	3170	1890	2840	1690	2540
2720	4090	2440	3670	2220	3330	22	2090	3150	1870	2810	1670	2510
2540	3810	2280	3420	2060	3100	24	2070	3120	1850	2780	1650	2480
2350	3530	2110	3160	1900	2860	26	2050	3090	1830	2750	1630	2460
2160	3250	1940	2910	1750	2630	28	2030	3060	1810	2720	1610	2430
1980	2980	1770	2660	1600	2400	30	2010	3030	1790	2690	1600	2400
1800	2710	1610	2420	1450	2180	32	1990	3000	1770	2660	1580	2370
1630	2450	1450	2180	1310	1960	34	1980	2970	1750	2640	1560	2340
1460	2200	1300	1950	1170	1750	36	1960	2940	1730	2610	1540	2310
1310	1970	1170	1750	1050	1570	38	1940	2910	1710	2580	1520	2280
1180	1780	1050	1580	945	1420	40	1920	2880	1700	2550	1500	2260
1070	1610	954	1430	857	1290	42	1900	2850	1680	2520	1480	2230
979	1470	869	1310	781	1170	44	1880	2820	1660	2490	1460	2200
896	1350	795	1200	715	1070	46	1860	2790	1640	2460	1440	2170
823	1240	730	1100	656	986	48	1840	2760	1620	2430	1420	2140
758	1140	673	1010	605	909	50	1820	2730	1600	2400	1410	2110
						Properties						
Available Strength in Tensile Yielding, kips							Limiting Unbraced Lengths, ft					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		L_p	L_r	L_p	L_r	L_p	L_r
3930	5910	3560	5350	3240	4870		13.1	106	13.0	96.7	12.9	88.3
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips							Area, in.^{2}					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		101		91.4		83.3	
3030	4550	2740	4110	2500	3750		Moment of Inertia, in.^{4}					
Available Strength in Shear, kips							I_x	I_y	I_x	I_y	I_x	I_y
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$		4900	1810	4330	1610	3840	1440
701	1050	627	940	560	840		r_y, in.					
Available Strength in Flexure about Y-Y Axis, kip-ft							4.24		4.20		4.17	
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$		r_x/r_y					
1100	1650	986	1480	889	1340		1.65		1.64		1.63	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W14

F_y = 65 ksi
F_u = 80 ksi

W14×						Shape	W14×					
257		233		211		lb/ft	257		233		211	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Design	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2940	4420	2670	4010	2410	3630	0	1580	2370	1410	2130	1260	1900
2860	4300	2590	3890	2340	3520	6	1580	2370	1410	2130	1260	1900
2830	4250	2560	3850	2320	3480	7	1580	2370	1410	2130	1260	1900
2800	4200	2530	3800	2290	3440	8	1580	2370	1410	2130	1260	1900
2760	4140	2500	3750	2260	3390	9	1580	2370	1410	2130	1260	1900
2720	4080	2460	3690	2220	3340	10	1580	2370	1410	2130	1260	1900
2670	4010	2420	3630	2180	3280	11	1580	2370	1410	2130	1260	1900
2620	3940	2370	3560	2140	3220	12	1580	2370	1410	2130	1260	1900
2570	3860	2320	3490	2100	3150	13	1580	2370	1410	2120	1260	1900
2510	3780	2270	3420	2050	3080	14	1570	2360	1400	2110	1250	1880
2460	3690	2220	3340	2000	3010	15	1560	2340	1390	2090	1240	1870
2400	3600	2160	3250	1950	2940	16	1550	2330	1380	2080	1230	1850
2330	3510	2110	3170	1900	2860	17	1540	2310	1370	2070	1220	1840
2270	3410	2050	3080	1850	2780	18	1530	2300	1370	2050	1220	1830
2200	3310	1990	2990	1790	2690	19	1520	2290	1360	2040	1210	1810
2130	3210	1930	2890	1730	2610	20	1510	2270	1350	2020	1200	1800
2000	3000	1800	2700	1620	2430	22	1490	2240	1330	2000	1180	1770
1850	2790	1670	2510	1500	2250	24	1470	2220	1310	1970	1160	1750
1710	2570	1540	2310	1380	2070	26	1460	2190	1290	1940	1140	1720
1570	2360	1410	2120	1260	1900	28	1440	2160	1270	1910	1120	1690
1430	2150	1280	1930	1150	1720	30	1420	2130	1250	1880	1110	1660
1290	1940	1160	1740	1040	1560	32	1400	2100	1240	1860	1090	1640
1160	1750	1040	1560	927	1390	34	1380	2070	1220	1830	1070	1610
1040	1560	927	1390	827	1240	36	1360	2050	1200	1800	1050	1580
932	1400	832	1250	742	1120	38	1340	2020	1180	1770	1030	1550
841	1260	751	1130	670	1010	40	1320	1990	1160	1750	1020	1530
763	1150	681	1020	608	913	42	1310	1960	1140	1720	997	1500
695	1040	621	933	554	832	44	1290	1930	1120	1690	979	1470
636	956	568	854	507	761	46	1270	1910	1110	1660	961	1440
584	878	522	784	465	699	48	1250	1880	1090	1630	943	1420
538	809	481	723	429	644	50	1230	1850	1070	1610	924	1390

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2940	4420	2670	4010	2410	3630
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2270	3400	2060	3080	1860	2790
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
503	755	445	668	400	600
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
798	1200	717	1080	642	965

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.8	80.7	12.7	73.5	12.6	67.2
Area, in.^{2}					
75.6		68.5		62.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
3400	1290	3010	1150	2660	1030
r_y, in.					
4.13		4.10		4.07	
r_x/r_y					
1.62		1.62		1.61	

Note: Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W14

F_y = 65 ksi
F_u = 80 ksi

W14×						Shape	W14×					
193		176		159		lb/ft	193		176		159	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2210	3320	2020	3030	1820	2730	0	1150	1730	1040	1560	931	1400
2150	3220	1960	2940	1760	2650	6	1150	1730	1040	1560	931	1400
2120	3190	1930	2910	1740	2620	7	1150	1730	1040	1560	931	1400
2100	3150	1910	2870	1720	2590	8	1150	1730	1040	1560	931	1400
2070	3110	1880	2830	1700	2550	9	1150	1730	1040	1560	931	1400
2030	3060	1850	2780	1670	2510	10	1150	1730	1040	1560	931	1400
2000	3000	1820	2740	1640	2460	11	1150	1730	1040	1560	931	1400
1960	2950	1780	2680	1610	2420	12	1150	1730	1040	1560	931	1400
1920	2890	1750	2630	1570	2360	13	1150	1720	1030	1550	925	1390
1880	2820	1710	2570	1540	2310	14	1140	1710	1020	1540	917	1380
1830	2750	1670	2500	1500	2250	15	1130	1700	1020	1530	908	1360
1790	2680	1620	2440	1460	2190	16	1120	1680	1010	1510	899	1350
1740	2610	1580	2370	1420	2130	17	1110	1670	997	1500	890	1340
1690	2540	1530	2300	1380	2070	18	1100	1660	988	1490	881	1320
1640	2460	1490	2230	1330	2010	19	1090	1640	979	1470	872	1310
1580	2380	1440	2160	1290	1940	20	1080	1630	970	1460	863	1300
1480	2220	1340	2010	1200	1810	22	1070	1600	952	1430	846	1270
1370	2050	1240	1860	1110	1670	24	1050	1570	935	1400	828	1240
1260	1890	1140	1710	1020	1530	26	1030	1550	917	1380	810	1220
1150	1730	1040	1560	929	1400	28	1010	1520	899	1350	793	1190
1040	1570	941	1410	842	1270	30	993	1490	881	1320	775	1160
941	1410	847	1270	757	1140	32	975	1460	863	1300	757	1140
841	1260	756	1140	675	1010	34	957	1440	845	1270	739	1110
750	1130	674	1010	602	905	36	938	1410	827	1240	722	1080
673	1010	605	909	540	812	38	920	1380	809	1220	704	1060
608	914	546	821	487	733	40	902	1360	791	1190	686	1030
551	829	495	744	442	665	42	884	1330	773	1160	669	1000
502	755	451	678	403	605	44	866	1300	756	1140	651	978
460	691	413	621	369	554	46	848	1270	738	1110	633	951
422	634	379	570	339	509	48	829	1250	720	1080	615	925
389	585	350	525	312	469	50	811	1220	702	1050	598	898

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2210	3320	2020	3030	1820	2730
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1700	2560	1550	2330	1400	2100
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
359	538	328	492	291	436
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
584	878	529	795	474	712

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.5	61.8	12.5	57.1	12.4	52.4
Area, in.^{2}					
56.8		51.8		46.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
2400	931	2140	838	1900	748
r_y, in.					
4.05		4.02		4.00	
r_x/r_y					
1.60		1.60		1.60	

Note: Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W14

F_y = 65 ksi
F_u = 80 ksi

W-Shapes

W14×						Shape		W14×					
145		132		120		lb/ft		145		132		120	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending		ASD	LRFD	ASD	LRFD	ASD	LRFD
1660	2500	1510	2270	1370	2070		0	843	1270	759	1140	688	1030
1610	2420	1460	2190	1330	1990		6	843	1270	759	1140	688	1030
1590	2390	1440	2160	1310	1970		7	843	1270	759	1140	688	1030
1570	2360	1420	2130	1290	1940		8	843	1270	759	1140	688	1030
1550	2330	1400	2100	1270	1910		9	843	1270	759	1140	688	1030
1520	2290	1370	2060	1250	1870		10	843	1270	759	1140	688	1030
1500	2250	1340	2020	1220	1830		11	843	1270	759	1140	688	1030
1470	2210	1310	1970	1190	1790		12	843	1270	756	1140	684	1030
1440	2160	1280	1930	1160	1750		13	837	1260	747	1120	675	1020
1400	2110	1250	1880	1130	1700		14	829	1250	738	1110	667	1000
1370	2060	1210	1830	1100	1660		15	820	1230	730	1100	658	989
1330	2000	1180	1770	1070	1610		16	811	1220	721	1080	650	977
1290	1950	1140	1720	1040	1560		17	803	1210	712	1070	641	964
1260	1890	1100	1660	1000	1500		18	794	1190	704	1060	633	951
1220	1830	1060	1600	965	1450		19	785	1180	695	1040	624	938
1180	1770	1030	1540	929	1400		20	777	1170	686	1030	615	925
1090	1640	945	1420	856	1290		22	759	1140	669	1010	598	899
1010	1520	865	1300	782	1180		24	742	1110	651	979	581	873
927	1390	785	1180	709	1070		26	724	1090	634	953	564	848
844	1270	707	1060	638	959		28	707	1060	616	927	547	822
764	1150	632	950	569	856		30	690	1040	599	900	530	796
686	1030	559	840	503	756		32	672	1010	582	874	513	770
611	918	495	744	446	670		34	655	984	564	848	495	745
545	819	442	664	398	598		36	637	958	547	822	478	719
489	735	397	596	357	536		38	620	932	529	796	461	693
441	663	358	538	322	484		40	603	906	512	769	444	667
400	602	325	488	292	439		42	585	879	494	743	424	638
365	548	296	445	266	400		44	568	853	477	717	402	604
334	501	271	407	244	366		46	550	827	454	682	381	573
306	461	249	374	224	336		48	533	801	432	650	363	545
282	424	229	344	206	310		50	511	768	413	620	346	520

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1660	2500	1510	2270	1370	2070
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1280	1920	1160	1750	1060	1590
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
262	392	247	370	222	334
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
431	648	367	551	331	497

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.3	48.7	11.6	44.3	11.6	41.5
Area, in.^{2}					
42.7		38.8		35.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1710	677	1530	548	1380	495
r_y, in.					
3.98		3.76		3.74	
r_x/r_y					
1.59		1.67		1.67	

Note: Confirm ASTM A913 material availability before specifying.

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W14

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W14×						Shape		W14×					
109		99		90		lb/ft		109[f]		99[f]		90[f]	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
1250	1870	1130	1700	1030	1550	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	615	924	541	813	479	719
1200	1810	1090	1640	995	1500		6	615	924	541	813	479	719
1190	1780	1080	1620	982	1480		7	615	924	541	813	479	719
1170	1760	1060	1600	968	1450		8	615	924	541	813	479	719
1150	1730	1040	1570	951	1430		9	615	924	541	813	479	719
1130	1700	1030	1540	933	1400		10	615	924	541	813	479	719
1110	1660	1000	1510	914	1370		11	615	924	541	813	479	719
1080	1620	982	1480	893	1340		12	615	924	541	813	479	719
1050	1590	957	1440	871	1310		13	611	918	541	813	479	719
1030	1540	932	1400	848	1270		14	602	905	541	813	479	719
998	1500	906	1360	824	1240		15	594	893	533	801	479	719
968	1460	878	1320	799	1200		16	586	880	525	789	473	712
937	1410	850	1280	773	1160		17	577	868	517	776	466	700
906	1360	821	1230	746	1120		18	569	855	509	764	458	688
873	1310	791	1190	719	1080		19	561	842	500	752	450	676
840	1260	761	1140	691	1040		20	552	830	492	740	442	664
774	1160	700	1050	636	956		22	535	805	476	716	426	640
707	1060	639	960	580	872		24	519	780	460	691	410	617
640	963	578	869	525	789		26	502	754	444	667	395	593
576	866	519	781	471	708		28	485	729	428	643	379	569
514	772	463	696	419	630		30	469	704	411	618	363	546
454	682	408	614	370	556		32	452	679	395	594	347	522
402	604	362	544	328	492		34	435	654	379	570	331	498
359	539	323	485	292	439		36	418	629	363	545	310	467
322	484	290	435	262	394		38	402	604	342	513	289	434
290	437	261	393	237	356		40	381	573	320	481	270	406
263	396	237	356	215	323		42	359	539	301	452	253	381
240	361	216	325	196	294		44	339	510	284	427	239	359
220	330	198	297	179	269		46	321	483	269	404	226	339
202	303	181	273	164	247		48	306	459	255	384	214	322
186	279	167	251	151	228		50	291	438	243	365	204	306

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1250	1870	1130	1700	1030	1550
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
960	1440	873	1310	795	1190
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
195	293	179	269	160	240
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
295	443	257	386	223	336

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.5	39.1	14.0	36.8	15.3	34.9
Area, in.^{2}					
32.0		29.1		26.5	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1240	447	1110	402	999	362
r_y, in.					
3.73		3.71		3.70	
r_x/r_y					
1.67		1.66		1.66	

[f] Shape exceeds compact limit for flexure with F_y = 65 ksi.

Note: Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W14

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W14×						Shape		W14×					
82		74		68		lb/ft		82		74		68	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
934	1400	849	1280	778	1170	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	451	678	409	614	373	561
862	1300	783	1180	718	1080		6	451	678	409	614	373	561
838	1260	761	1140	697	1050		7	451	678	409	614	373	561
810	1220	736	1110	674	1010		8	448	673	406	610	370	556
780	1170	709	1060	648	974		9	439	660	397	597	361	543
748	1120	679	1020	621	933		10	430	646	388	584	353	530
714	1070	648	974	592	890		11	421	633	379	570	344	517
678	1020	616	926	562	845		12	412	619	371	557	336	505
641	964	583	876	531	798		13	403	606	362	544	327	492
604	908	549	824	500	751		14	394	592	353	531	319	479
566	851	514	773	468	703		15	385	578	344	517	310	466
528	794	480	721	436	656		16	376	565	335	504	302	453
491	738	446	670	405	609		17	367	551	327	491	293	441
454	683	413	620	374	562		18	358	538	318	478	285	428
418	629	380	571	344	517		19	349	524	309	465	276	415
384	576	348	524	315	473		20	340	511	300	451	268	402
318	478	289	435	261	392		22	322	484	283	425	251	377
267	402	243	365	219	330		24	304	456	265	398	233	351
228	343	207	311	187	281		26	286	429	244	367	210	315
197	295	179	268	161	242		28	263	396	222	334	190	286
171	257	156	234	140	211		30	242	364	204	306	174	262
150	226	137	205	123	185		32	224	337	188	283	161	242
133	200	121	182	109	164		34	209	313	175	263	149	224
119	179	108	162	97.5	147		36	195	293	164	246	139	209
107	160	96.9	146	87.5	131		38	183	276	154	231	131	196
96.3	145	87.5	131	79.0	119		40	173	260	145	218	123	185
							42	164	246	137	206	116	175
							44	156	234	130	195	110	166
							46	148	223	124	186	105	157
							48	141	212	118	177	99.9	150
							50	135	203	113	169	95.4	143

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
934	1400	849	1280	778	1170
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
720	1080	654	981	600	900
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
190	284	166	249	151	227
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
145	218	131	197	120	180

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.68	26.7	7.68	25.2	7.62	24.0
Area, in.^{2}					
24.0		21.8		20.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
881	148	795	134	722	121
r_y, in.					
2.48		2.48		2.46	
r_x/r_y					
2.44		2.44		2.44	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

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W14

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W14×						Shape		W14×					
61		53		48^c		lb/ft		61		53		48	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending		ASD	LRFD	ASD	LRFD	ASD	LRFD
697	1050	607	913	542	815		0	331	497	283	425	254	382
642	965	531	798	479	721		6	331	497	282	424	254	381
623	936	506	761	457	686		7	331	497	274	411	245	369
602	905	479	720	432	649		8	328	492	265	398	237	357
579	871	449	676	405	609		9	320	480	257	386	229	344
555	834	419	630	377	567		10	311	468	248	373	221	332
529	795	387	582	349	524		11	303	456	239	360	213	320
502	754	356	535	320	481		12	295	444	231	347	205	308
474	712	324	487	291	438		13	287	432	222	334	197	295
446	670	293	441	263	395		14	279	420	214	321	188	283
417	627	263	396	236	355		15	271	408	205	309	180	271
389	584	234	352	210	315		16	263	396	197	296	172	259
360	542	208	312	186	279		17	255	384	188	283	164	246
333	500	185	278	166	249		18	247	372	180	270	153	231
306	460	166	250	149	224		19	239	360	168	253	142	213
280	421	150	226	134	202		20	231	347	157	236	132	198
232	348	124	186	111	167		22	215	323	138	207	116	174
195	293	104	157	93.2	140		24	194	291	123	185	103	155
166	249	88.8	133	79.4	119		26	173	261	111	167	92.7	139
143	215	76.6	115	68.5	103		28	157	236	101	153	84.3	127
125	187	66.7	100	59.7	89.7		30	143	215	93.3	140	77.4	116
110	165	58.6	88.1				32	132	198	86.4	130	71.5	107
97.0	146						34	122	184	80.4	121	66.5	99.9
86.5	130						36	114	171	75.3	113	62.1	93.4
77.7	117						38	107	160	70.8	106	58.3	87.6
70.1	105						40	100	151	66.8	100	55.0	82.6
							42	94.6	142	63.2	95.0	52.0	78.1
							44	89.5	135	60.0	90.2	49.3	74.1
							46	85.0	128	57.1	85.9	46.9	70.5
							48	81.0	122	54.5	82.0	44.8	67.3
							50	77.3	116	52.2	78.4	42.8	64.3

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
697	1050	607	913	549	825
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
537	806	468	702	423	635
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
136	203	134	201	122	183
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
106	160	71.4	107	63.6	95.6

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.59	22.7	5.95	18.4	5.92	17.6
Area, in.^{2}					
17.9		15.6		14.1	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
640	107	541	57.7	484	51.4
r_y, in.					
2.45		1.92		1.91	
r_x/r_y					
2.44		3.07		3.06	

^{c} Shape is slender for compression with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W14

F_y = 65 ksi
F_u = 80 ksi

W14× 43^c		W14× 38^c		W14× 34^c		Shape lb/ft	W14× 43		W14× 38		W14× 34	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
475	713	413	621	361	542	0	226	339	199	300	177	266
419	630	345	518	300	450	6	225	338	189	285	167	251
400	602	323	485	280	421	7	217	326	181	272	159	239
380	571	299	450	260	390	8	209	315	172	259	151	228
358	539	275	413	238	357	9	202	303	164	246	144	216
334	502	247	371	216	324	10	194	292	155	234	136	204
308	464	219	329	192	288	11	186	280	147	221	128	192
282	425	192	288	168	252	12	179	269	138	208	120	180
257	386	166	250	145	217	13	171	257	130	195	112	169
231	348	143	215	125	187	14	164	246	120	180	100	151
207	311	125	188	109	163	15	156	234	108	162	89.9	135
184	276	110	165	95.4	143	16	148	223	97.4	146	81.2	122
163	244	97.2	146	84.5	127	17	140	210	88.9	134	73.9	111
145	218	86.7	130	75.4	113	18	128	192	81.8	123	67.7	102
130	196	77.8	117	67.7	102	19	118	177	75.6	114	62.5	93.9
117	177	70.2	106	61.1	91.8	20	109	164	70.3	106	57.9	87.1
97.1	146	58.0	87.2	50.5	75.9	22	95.5	144	61.6	92.6	50.6	76.0
81.6	123	48.8	73.3	42.4	63.8	24	84.6	127	54.8	82.4	44.8	67.4
69.5	104					26	76.0	114	49.4	74.2	40.2	60.5
59.9	90.1					28	68.9	104	44.9	67.5	36.5	54.9
52.2	78.5					30	63.1	94.8	41.2	62.0	33.4	50.2
						32	58.2	87.4	38.1	57.3	30.8	46.3
						34	54.0	81.1	35.4	53.2	28.6	43.0
						36	50.4	75.7	33.1	49.8	26.7	40.1
						38	47.2	70.9	31.1	46.7	25.0	37.6
						40	44.4	66.8	29.3	44.0	23.5	35.4
						42	42.0	63.1	27.7	41.7	22.2	33.4
						44	39.8	59.8	26.3	39.5	21.1	31.7
						46	37.8	56.8	25.0	37.6	20.0	30.1
						48	36.0	54.2	23.9	35.9	19.1	28.7
						50	34.4	51.7	22.8	34.3	18.2	27.4

Properties

Available Strength in Tensile Yielding, kips							Limiting Unbraced Lengths, ft					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		L_p	L_r	L_p	L_r	L_p	L_r
490	737	436	655	389	585		5.86	16.8	4.80	13.7	4.74	13.2
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips							Area, in.^{2}					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		12.6		11.2		10.0	
378	567	336	504	300	450		Moment of Inertia, in.^{4}					
Available Strength in Shear, kips							I_x	I_y	I_x	I_y	I_x	I_y
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$		428	45.2	385	26.7	340	23.3
109	163	114	170	104	156		r_y, in.					
Available Strength in Flexure about Y-Y Axis, kip-ft							1.89		1.55		1.53	
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$		r_x/r_y					
56.1	84.3	39.2	59.0	34.4	51.7		3.08		3.79		3.81	

[c] Shape is slender for compression with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W14

F_y = 65 ksi
F_u = 80 ksi

W14×						Shape	W14×					
30^c		26^c		22^c		lb/ft	30^f		26^v		22^v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
313	471	266	399	216	324	0	150	226	130	196	108	162
258	388	185	278	146	220	6	143	216	108	163	87.5	131
241	362	162	244	127	191	7	136	205	100	151	80.2	121
222	334	139	209	108	163	8	129	194	91.9	138	72.9	110
203	305	116	174	90.1	135	9	122	183	83.7	126	65.4	98.2
183	275	93.6	141	73.3	110	10	115	173	72.5	109	54.4	81.7
163	246	77.4	116	60.6	91.0	11	108	162	61.9	93.1	46.2	69.4
142	213	65.0	97.7	50.9	76.5	12	100	151	53.9	81.0	39.9	60.0
121	182	55.4	83.3	43.4	65.2	13	91.7	138	47.5	71.5	35.0	52.7
105	157	47.8	71.8	37.4	56.2	14	80.9	122	42.5	63.8	31.1	46.8
91.1	137	41.6	62.5	32.6	48.9	15	72.1	108	38.3	57.6	28.0	42.0
80.1	120	36.6	55.0	28.6	43.0	16	64.9	97.5	34.9	52.4	25.3	38.1
71.0	107	32.4	48.7	25.4	38.1	17	58.9	88.5	32.0	48.1	23.1	34.8
63.3	95.1	28.9	43.4			18	53.8	80.9	29.5	44.4	21.3	32.0
56.8	85.4					19	49.5	74.4	27.4	41.2	19.7	29.6
51.3	77.1					20	45.8	68.8	25.6	38.5	18.3	27.5
42.4	63.7					22	39.7	59.7	22.6	34.0	16.1	24.1
35.6	53.5					24	35.1	52.7	20.2	30.4	14.3	21.5
						26	31.3	47.1	18.3	27.5	12.9	19.4
						28	28.3	42.6	16.7	25.1	11.7	17.6
						30	25.9	38.9	15.4	23.2	10.8	16.2
						32	23.8	35.7	14.3	21.5	9.94	14.9
						34	22.0	33.1	13.3	20.0	9.25	13.9
						36	20.5	30.8	12.5	18.8	8.64	13.0
						38	19.2	28.8	11.7	17.6	8.12	12.2
						40	18.0	27.1	11.1	16.7	7.65	11.5
						42	17.0	25.5	10.5	15.8	7.24	10.9
						44	16.1	24.2	9.98	15.0	6.87	10.3
						46	15.3	22.9	9.51	14.3	6.54	9.82
						48	14.5	21.8	9.08	13.6	6.23	9.37
						50	13.9	20.8	8.69	13.1	5.96	8.96

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
344	518	299	450	253	380
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
266	398	231	346	195	292
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
96.9	145	82.8	124	73.6	111
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
28.3	42.5	18.0	27.0	14.2	21.4

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.06	12.7	3.35	9.42	3.22	8.97
Area, in.^{2}					
8.85		7.69		6.49	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
291	19.6	245	8.91	199	7.00
r_y, in.					
1.49		1.08		1.04	
r_x/r_y					
3.85		5.23		5.33	

[c] Shape is slender for compression with F_y = 65 ksi.

[f] Shape exceeds compact limit for flexure with F_y = 65 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W12

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W12×						Shape	W12×					
336^h		305^h		279^h		lb/ft	336^h		305^h		279^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
3850	5790	3480	5240	3190	4790	0	1960	2940	1740	2620	1560	2340
3700	5550	3340	5020	3050	4590	6	1960	2940	1740	2620	1560	2340
3640	5470	3290	4940	3010	4520	7	1960	2940	1740	2620	1560	2340
3580	5380	3230	4860	2950	4440	8	1960	2940	1740	2620	1560	2340
3510	5280	3170	4760	2890	4350	9	1960	2940	1740	2620	1560	2340
3440	5160	3100	4660	2830	4250	10	1960	2940	1740	2620	1560	2340
3350	5040	3020	4540	2760	4140	11	1950	2940	1740	2610	1560	2340
3270	4910	2940	4420	2680	4030	12	1950	2920	1730	2600	1550	2330
3180	4770	2860	4300	2600	3910	13	1940	2910	1720	2590	1540	2320
3080	4630	2770	4160	2520	3790	14	1930	2900	1710	2580	1530	2300
2980	4480	2680	4020	2430	3660	15	1920	2890	1710	2570	1530	2290
2880	4320	2580	3880	2350	3530	16	1910	2880	1700	2550	1520	2280
2770	4170	2480	3730	2250	3390	17	1900	2860	1690	2540	1510	2270
2660	4000	2380	3580	2160	3250	18	1900	2850	1680	2530	1500	2260
2550	3840	2280	3430	2070	3110	19	1890	2840	1670	2520	1490	2250
2440	3670	2180	3280	1970	2970	20	1880	2830	1670	2510	1490	2230
2220	3340	1980	2970	1790	2680	22	1860	2800	1650	2480	1470	2210
2000	3010	1780	2670	1600	2400	24	1850	2780	1630	2460	1460	2190
1790	2680	1580	2370	1420	2130	26	1830	2750	1620	2430	1440	2160
1580	2370	1390	2090	1250	1870	28	1810	2730	1600	2410	1420	2140
1380	2080	1210	1820	1090	1630	30	1800	2700	1590	2390	1410	2120
1210	1820	1070	1600	954	1430	32	1780	2680	1570	2360	1390	2090
1080	1620	945	1420	845	1270	34	1770	2650	1560	2340	1380	2070
959	1440	843	1270	754	1130	36	1750	2630	1540	2310	1360	2050
861	1290	757	1140	676	1020	38	1730	2600	1520	2290	1350	2020
777	1170	683	1030	610	917	40	1720	2580	1510	2270	1330	2000
705	1060	619	931	554	832	42	1700	2560	1490	2240	1320	1980
642	965	564	848	504	758	44	1680	2530	1480	2220	1300	1960
587	883	516	776	462	694	46	1670	2510	1460	2190	1290	1930
539	811	474	713	424	637	48	1650	2480	1440	2170	1270	1910
497	747	437	657	391	587	50	1630	2460	1430	2150	1250	1890

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3850	5790	3480	5240	3190	4790
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2970	4450	2690	4030	2460	3690
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
778	1170	691	1040	633	949
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
889	1340	791	1190	714	1070

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.7	116	10.6	105	10.5	96.8
Area, in.^{2}					
98.9		89.5		81.9	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
4060	1190	3550	1050	3110	937
r_y, in.					
3.47		3.42		3.38	
r_x/r_y					
1.85		1.84		1.82	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W12

F_y = 65 ksi
F_u = 80 ksi

W12×						Shape		W12×					
252^h		230^h		210		lb/ft		252^h		230^h		210	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
2880	4330	2640	3960	2410	3620	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	1390	2090	1250	1880	1130	1700
2760	4150	2520	3790	2300	3450		6	1390	2090	1250	1880	1130	1700
2720	4080	2480	3730	2260	3400		7	1390	2090	1250	1880	1130	1700
2670	4010	2430	3660	2220	3330		8	1390	2090	1250	1880	1130	1700
2610	3920	2380	3580	2170	3260		9	1390	2090	1250	1880	1130	1700
2550	3830	2330	3500	2120	3180		10	1390	2090	1250	1880	1130	1700
2490	3740	2270	3400	2060	3100		11	1380	2080	1250	1870	1120	1690
2420	3630	2200	3310	2000	3010		12	1380	2070	1240	1860	1120	1680
2340	3520	2130	3210	1940	2920		13	1370	2060	1230	1850	1110	1670
2270	3410	2060	3100	1870	2820		14	1360	2050	1220	1840	1100	1650
2190	3290	1990	2990	1810	2720		15	1350	2030	1220	1830	1090	1640
2110	3170	1910	2880	1740	2610		16	1350	2020	1210	1820	1090	1630
2020	3040	1840	2760	1670	2500		17	1340	2010	1200	1810	1080	1620
1940	2910	1760	2640	1590	2390		18	1330	2000	1190	1800	1070	1610
1850	2780	1680	2520	1520	2280		19	1320	1990	1190	1780	1060	1600
1770	2650	1600	2400	1450	2170		20	1320	1980	1180	1770	1060	1590
1590	2390	1440	2160	1300	1950		22	1300	1950	1160	1750	1040	1570
1420	2140	1280	1930	1160	1740		24	1290	1930	1150	1730	1030	1540
1260	1890	1130	1700	1020	1530		26	1270	1910	1140	1710	1010	1520
1100	1650	988	1480	885	1330		28	1250	1890	1120	1680	998	1500
959	1440	860	1290	771	1160		30	1240	1860	1110	1660	984	1480
843	1270	756	1140	678	1020		32	1220	1840	1090	1640	969	1460
746	1120	670	1010	600	902		34	1210	1820	1080	1620	955	1430
666	1000	597	898	535	805		36	1190	1800	1060	1590	940	1410
598	898	536	806	481	722		38	1180	1770	1050	1570	925	1390
539	811	484	727	434	652		40	1160	1750	1030	1550	911	1370
489	735	439	660	393	591		42	1150	1730	1020	1530	896	1350
446	670	400	601	358	539		44	1130	1700	1000	1510	882	1320
408	613	366	550	328	493		46	1120	1680	987	1480	867	1300
374	563	336	505	301	453		48	1100	1660	972	1460	852	1280
345	519	310	465	278	417		50	1090	1640	957	1440	838	1260

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2880	4330	2640	3960	2410	3620
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2220	3330	2030	3050	1850	2780
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
561	841	506	760	451	676
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
636	956	574	863	516	775

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.3	88.0	10.3	80.7	10.2	73.9
Area, in.^{2}					
74.1		67.7		61.8	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
2720	828	2420	742	2140	664
r_y, in.					
3.34		3.31		3.28	
r_x/r_y					
1.81		1.80		1.80	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W12

F_y = 65 ksi
F_u = 80 ksi

W12×						Shape		W12×					
190		170		152		lb/ft		190		170		152	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
2180	3280	1950	2920	1740	2610	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	1010	1520	892	1340	788	1180
2080	3130	1860	2790	1660	2490		6	1010	1520	892	1340	788	1180
2050	3070	1820	2740	1630	2450		7	1010	1520	892	1340	788	1180
2010	3020	1790	2690	1600	2400		8	1010	1520	892	1340	788	1180
1960	2950	1750	2630	1560	2350		9	1010	1520	892	1340	788	1180
1910	2880	1710	2560	1520	2290		10	1010	1520	892	1340	787	1180
1860	2800	1660	2490	1480	2220		11	1000	1510	885	1330	780	1170
1810	2720	1610	2420	1430	2150		12	995	1500	878	1320	773	1160
1750	2630	1560	2340	1390	2080		13	988	1480	871	1310	766	1150
1690	2540	1500	2260	1340	2010		14	980	1470	864	1300	759	1140
1630	2450	1450	2170	1290	1930		15	973	1460	856	1290	752	1130
1560	2350	1390	2090	1230	1850		16	966	1450	849	1280	745	1120
1500	2250	1330	2000	1180	1770		17	959	1440	842	1270	738	1110
1430	2150	1270	1910	1130	1690		18	952	1430	835	1260	731	1100
1370	2050	1210	1820	1070	1610		19	945	1420	828	1240	724	1090
1300	1950	1150	1730	1020	1530		20	937	1410	821	1230	717	1080
1160	1750	1030	1540	907	1360		22	923	1390	807	1210	703	1060
1030	1550	910	1370	802	1210		24	909	1370	793	1190	690	1040
908	1360	797	1200	701	1050		26	894	1340	779	1170	676	1020
788	1180	690	1040	606	910		28	880	1320	765	1150	662	994
686	1030	601	904	528	793		30	866	1300	751	1130	648	973
603	906	528	794	464	697		32	851	1280	736	1110	634	952
534	803	468	704	411	617		34	837	1260	722	1090	620	931
476	716	418	628	366	551		36	823	1240	708	1060	606	910
428	643	375	563	329	494		38	808	1210	694	1040	592	889
386	580	338	508	297	446		40	794	1190	680	1020	578	868
350	526	307	461	269	405		42	779	1170	666	1000	564	847
319	479	280	420	245	369		44	765	1150	652	979	550	826
292	439	256	384	224	337		46	751	1130	637	958	536	805
268	403	235	353	206	310		48	736	1110	623	937	522	784
247	371	216	325	190	285		50	722	1090	609	916	508	763

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2180	3280	1950	2930	1740	2610
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1680	2520	1500	2250	1340	2010
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
397	595	349	524	310	465
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
464	697	409	614	360	541

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.1	67.4	9.98	60.7	9.88	54.8
Area, in.^{2}					
56.0		50.0		44.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1890	589	1650	517	1430	454
r_y, in.					
3.25		3.22		3.19	
r_x/r_y					
1.79		1.78		1.77	

Note: Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W12

F_y = 65 ksi
F_u = 80 ksi

W-Shapes

W12×						Shape		W12×					
136		120		106		lb/ft		136		120		106	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
1550	2330	1370	2060	1210	1830	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	694	1040	603	907	532	800
1480	2220	1300	1960	1150	1730		6	694	1040	603	907	532	800
1450	2180	1280	1920	1130	1700		7	694	1040	603	907	532	800
1420	2140	1250	1880	1110	1670		8	694	1040	603	907	532	800
1390	2090	1220	1840	1080	1630		9	694	1040	603	907	532	800
1350	2040	1190	1790	1050	1580		10	693	1040	601	904	529	796
1320	1980	1160	1740	1020	1540		11	686	1030	594	893	523	786
1270	1920	1120	1680	990	1490		12	679	1020	588	883	516	776
1230	1850	1080	1630	956	1440		13	672	1010	581	873	509	766
1190	1780	1040	1570	920	1380		14	665	1000	574	863	503	756
1140	1710	1000	1500	883	1330		15	658	989	567	853	496	745
1090	1640	958	1440	845	1270		16	651	979	561	843	489	735
1050	1570	915	1380	807	1210		17	644	968	554	832	483	725
996	1500	871	1310	768	1150		18	637	958	547	822	476	715
947	1420	827	1240	729	1100		19	630	948	540	812	469	705
898	1350	783	1180	689	1040		20	624	937	534	802	462	695
800	1200	697	1050	612	920		22	610	916	520	782	449	675
705	1060	613	921	537	808		24	596	896	507	761	436	655
615	924	532	800	466	700		26	582	875	493	741	422	635
530	797	459	690	402	604		28	568	854	479	721	409	615
462	695	400	601	350	526		30	554	833	466	700	395	594
406	610	352	528	308	462		32	541	813	452	680	382	574
360	541	311	468	272	410		34	527	792	439	660	369	554
321	482	278	417	243	365		36	513	771	425	639	355	534
288	433	249	375	218	328		38	499	750	412	619	342	514
260	391	225	338	197	296		40	485	730	398	599	328	493
236	354	204	307	179	268		42	472	709	385	578	311	467
215	323	186	279	163	245		44	458	688	371	558	295	444
197	295	170	256	149	224		46	444	667	354	532	281	423
181	271	156	235	137	205		48	430	646	338	508	268	404
166	250	144	216	126	189		50	414	623	324	487	257	386

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1550	2330	1370	2060	1210	1830
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1200	1800	1060	1580	936	1400
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
275	413	242	363	205	307
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
318	478	277	416	244	366

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.79	49.1	9.70	44.2	9.63	39.9
Area, in.^{2}					
39.9		35.2		31.2	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1240	398	1070	345	933	301
r_y, in.					
3.16		3.13		3.11	
r_x/r_y					
1.77		1.76		1.76	

Note: Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W12

F_y = 65 ksi
F_u = 80 ksi

W12×						Shape	W12×					
96		87		79		lb/ft	96		87		79^f	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1100	1650	996	1500	903	1360	0	477	717	428	644	384	577
1040	1570	946	1420	856	1290	6	477	717	428	644	384	577
1020	1540	928	1390	840	1260	7	477	717	428	644	384	577
1000	1510	908	1360	822	1240	8	477	717	428	644	384	577
977	1470	886	1330	802	1200	9	477	717	428	644	384	577
951	1430	862	1300	779	1170	10	474	712	425	639	382	575
923	1390	836	1260	756	1140	11	467	703	419	629	376	565
893	1340	808	1210	731	1100	12	461	693	412	619	370	556
861	1290	780	1170	704	1060	13	454	683	406	610	364	546
829	1250	750	1130	677	1020	14	448	673	399	600	357	537
795	1190	719	1080	648	975	15	441	663	393	590	351	527
760	1140	687	1030	620	931	16	435	653	386	581	345	518
725	1090	655	984	590	887	17	428	644	380	571	338	508
690	1040	622	936	561	843	18	422	634	373	561	332	499
654	983	590	887	531	798	19	415	624	367	552	326	490
619	930	557	838	501	753	20	409	614	361	542	319	480
548	824	493	742	443	666	22	395	594	348	522	307	461
481	722	432	649	387	582	24	382	575	335	503	294	442
416	625	373	560	333	501	26	369	555	322	484	282	423
358	539	321	483	287	432	28	356	535	309	464	269	404
312	469	280	421	250	376	30	343	516	296	445	256	385
274	413	246	370	220	331	32	330	496	283	426	244	366
243	365	218	327	195	293	34	317	476	270	406	226	340
217	326	194	292	174	261	36	304	456	253	381	211	317
195	293	174	262	156	234	38	288	433	238	357	198	297
176	264	157	237	141	212	40	272	408	224	337	186	280
159	239	143	215	128	192	42	257	386	212	318	176	264
145	218	130	196	116	175	44	244	367	201	302	167	250
133	200	119	179	106	160	46	232	349	191	287	158	238
122	183	109	164	97.8	147	48	222	333	182	274	151	227
112	169	101	151	90.1	135	50	212	319	174	262	144	217

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1100	1650	996	1500	903	1360
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
846	1270	768	1150	696	1040
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
182	272	167	251	152	227
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
219	329	196	294	175	263

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.57	37.0	9.51	34.4	9.78	32.1
Area, in.^{2}					
28.2		25.6		23.2	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
833	270	740	241	662	216
r_y, in.					
3.09		3.07		3.05	
r_x/r_y					
1.76		1.75		1.75	

[f] Shape exceeds compact limit for flexure with F_y = 65 ksi.

Note: Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W12

F_y = 65 ksi
F_u = 80 ksi

W12×						Shape	W12×					
72		65		58		lb/ft	72^f		65^f		58	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
821	1230	743	1120	662	994	0	341	512	297	447	280	421
779	1170	704	1060	612	920	6	341	512	297	447	280	421
764	1150	691	1040	595	894	7	341	512	297	447	280	421
747	1120	675	1020	576	865	8	341	512	297	447	279	419
728	1090	658	989	555	834	9	341	512	297	447	273	410
708	1060	640	962	532	800	10	341	512	297	447	266	400
687	1030	620	932	509	765	11	341	512	297	447	260	391
664	997	599	900	484	727	12	334	503	297	447	254	382
639	961	577	867	458	689	13	328	493	293	440	248	372
614	923	554	833	432	650	14	322	484	287	431	242	363
589	885	530	797	406	610	15	316	475	281	422	235	354
562	845	506	761	379	570	16	310	466	275	413	229	344
535	805	482	724	353	531	17	304	456	269	404	223	335
508	764	457	687	327	492	18	298	447	263	395	217	326
481	723	432	650	302	454	19	291	438	257	387	210	316
454	683	408	613	277	417	20	285	429	251	378	204	307
401	603	360	540	231	347	22	273	410	240	360	192	288
350	526	313	471	194	292	24	261	392	228	342	179	270
301	453	269	404	165	249	26	248	373	216	325	162	244
260	390	232	349	143	214	28	236	355	204	307	148	222
226	340	202	304	124	187	30	224	336	189	284	135	203
199	299	178	267	109	164	32	207	311	173	260	125	188
176	265	157	236	96.7	145	34	192	288	160	241	116	174
157	236	140	211	86.3	130	36	179	268	149	224	108	163
141	212	126	189	77.4	116	38	167	251	139	209	102	153
127	191	114	171	69.9	105	40	157	236	130	196	95.7	144
115	173	103	155			42	148	223	123	185	90.5	136
105	158	93.9	141			44	140	211	116	175	85.8	129
96.2	145	85.9	129			46	133	200	110	166	81.6	123
88.3	133	78.9	119			48	127	191	105	158	77.8	117
81.4	122	72.7	109			50	121	182	100	150	74.3	112

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
821	1230	743	1120	662	995
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
633	950	573	860	510	765
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
138	206	123	184	114	171
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
153	230	132	198	105	158

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.0	30.4	12.2	28.8	7.78	24.4
Area, in.^{2}					
21.1		19.1		17.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
597	195	533	174	475	107
r_y, in.					
3.04		3.02		2.51	
r_x/r_y					
1.75		1.75		2.10	

^{f} Shape exceeds compact limit for flexure with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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W12

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W12× 53 P_n/Ω_c ASD	W12× 53 $\phi_c P_n$ LRFD	W12× 50 P_n/Ω_c ASD	W12× 50 $\phi_c P_n$ LRFD	W12× 45 P_n/Ω_c ASD	W12× 45 $\phi_c P_n$ LRFD	Design: Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	W12× 53f M_{nx}/Ω_b ASD	W12× 53f $\phi_b M_{nx}$ LRFD	W12× 50 M_{nx}/Ω_b ASD	W12× 50 $\phi_b M_{nx}$ LRFD	W12× 45 M_{nx}/Ω_b ASD	W12× 45 $\phi_b M_{nx}$ LRFD
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
607	913	568	854	510	766	0	248	373	233	351	208	313
560	842	500	751	448	673	6	248	373	233	351	208	313
544	818	477	717	427	642	7	248	373	227	341	202	304
527	791	452	680	405	609	8	248	373	221	332	196	295
507	762	426	640	381	573	9	245	368	214	322	190	285
486	731	398	598	356	535	10	239	359	208	312	184	276
464	697	369	555	330	496	11	233	350	201	302	177	267
441	662	340	511	304	456	12	227	341	195	292	171	257
417	627	311	468	278	417	13	221	332	188	283	165	248
393	590	283	425	252	378	14	215	323	181	273	159	239
368	553	255	383	227	341	15	209	314	175	263	153	229
343	516	228	343	203	305	16	203	306	168	253	146	220
319	480	203	304	180	270	17	197	297	162	243	140	211
295	444	181	272	160	241	18	191	288	155	233	134	201
272	409	162	244	144	216	19	185	279	149	224	126	189
249	375	146	220	130	195	20	180	270	140	211	117	176
207	311	121	182	107	161	22	168	252	124	186	103	155
174	261	102	153	90.3	136	24	153	230	111	167	92.0	138
148	223	86.6	130	76.9	116	26	137	206	101	151	83.1	125
128	192	74.7	112	66.3	99.7	28	124	187	91.9	138	75.8	114
111	167	65.0	97.8	57.8	86.8	30	114	171	84.6	127	69.7	105
97.8	147	57.2	85.9	50.8	76.3	32	105	157	78.5	118	64.5	97.0
86.6	130					34	97.2	146	73.2	110	60.1	90.3
77.3	116					36	90.6	136	68.6	103	56.2	84.5
69.4	104					38	84.9	128	64.5	97.0	52.9	79.4
62.6	94.1					40	79.8	120	60.9	91.6	49.9	75.0
						42	75.4	113	57.7	86.8	47.2	71.0
						44	71.4	107	54.9	82.5	44.8	67.4
						46	67.9	102	52.3	78.6	42.7	64.2
						48	64.7	97.2	49.9	75.1	40.7	61.2
						50	61.7	92.8	47.8	71.8	39.0	58.6

Properties

	W12× 53		W12× 50		W12× 45	
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	607	913	568	854	510	766
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	468	702	438	657	393	590
Available Strength in Shear, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	109	163	117	176	105	158
Available Strength in Flexure about Y-Y Axis, kip-ft	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	91.8	138	69.1	104	61.6	92.6

	W12× 53f		W12× 50		W12× 45	
Limiting Unbraced Lengths, ft	L_p	L_r	L_p	L_r	L_p	L_r
	8.47	23.2	6.07	19.5	6.04	18.5
Area, in.^{2}	15.6		14.6		13.1	
Moment of Inertia, in.^{4}	I_x	I_y	I_x	I_y	I_x	I_y
	425	95.8	391	56.3	348	50.0
r_y, in.	2.48		1.96		1.95	
r_x/r_y	2.11		2.64		2.64	

[f] Shape exceeds compact limit for flexure with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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W12

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W12× 40^c Available Compressive Strength, kips P_n/Ω_c ASD	40^c $\phi_c P_n$ LRFD	35^c P_n/Ω_c ASD	35^c $\phi_c P_n$ LRFD	30^c P_n/Ω_c ASD	30^c $\phi_c P_n$ LRFD	Shape lb/ft Design: Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	W12× 40 Available Flexural Strength, kip-ft M_{nx}/Ω_b ASD	40 $\phi_b M_{nx}$ LRFD	35 M_{nx}/Ω_b ASD	35 $\phi_b M_{nx}$ LRFD	30 M_{nx}/Ω_b ASD	30 $\phi_b M_{nx}$ LRFD
450	677	389	584	321	483	0	185	278	166	250	140	210
400	600	323	486	266	399	6	185	278	158	237	132	198
381	573	302	454	248	373	7	179	269	151	227	126	189
361	542	277	416	229	345	8	173	260	144	216	120	180
339	510	251	378	210	315	9	167	252	137	206	113	171
317	476	225	338	189	284	10	161	243	130	196	107	161
293	441	199	300	167	251	11	156	234	124	186	101	152
270	405	175	262	146	219	12	150	225	117	175	95.1	143
246	370	151	227	125	189	13	144	216	110	165	88.9	134
223	336	130	196	108	163	14	138	207	103	154	79.8	120
201	302	113	170	94.2	142	15	132	199	92.6	139	71.6	108
179	270	99.6	150	82.8	124	16	126	190	84.3	127	64.8	97.4
159	239	88.2	133	73.3	110	17	120	181	77.2	116	59.1	88.9
142	213	78.7	118	65.4	98.3	18	113	170	71.3	107	54.4	81.7
127	191	70.6	106	58.7	88.3	19	104	157	66.1	99.4	50.3	75.5
115	173	63.7	95.8	53.0	79.7	20	97.0	146	61.7	92.7	46.7	70.2
95.0	143	52.7	79.2	43.8	65.8	22	85.0	128	54.4	81.7	40.9	61.5
79.8	120	44.3	66.5	36.8	55.3	24	75.6	114	48.6	73.1	36.4	54.7
68.0	102					26	68.0	102	44.0	66.1	32.8	49.3
58.6	88.1					28	61.9	93.0	40.2	60.4	29.8	44.8
51.1	76.8					30	56.8	85.3	37.0	55.6	27.3	41.1
44.9	67.5					32	52.4	78.8	34.3	51.5	25.3	38.0
						34	48.7	73.3	31.9	48.0	23.5	35.3
						36	45.6	68.5	29.9	44.9	21.9	33.0
						38	42.8	64.3	28.1	42.3	20.6	31.0
						40	40.3	60.6	26.6	39.9	19.4	29.2
						42	38.1	57.3	25.2	37.8	18.4	27.6
						44	36.2	54.3	23.9	35.9	17.4	26.2
						46	34.4	51.7	22.8	34.2	16.6	24.9
						48	32.8	49.3	21.7	32.7	15.8	23.8
						50	31.4	47.1	20.8	31.3	15.1	22.7

Properties

Available Strength in Tensile Yielding, kips

40 P_n/Ω_t	40 $\phi_t P_n$	35 P_n/Ω_t	35 $\phi_t P_n$	30 P_n/Ω_t	30 $\phi_t P_n$
455	684	401	603	342	514

Available Strength in Tensile Rupture (A_e = 0.75A_g), kips

40 P_n/Ω_t	40 $\phi_t P_n$	35 P_n/Ω_t	35 $\phi_t P_n$	30 P_n/Ω_t	30 $\phi_t P_n$
351	527	309	464	264	396

Available Strength in Shear, kips

40 V_n/Ω_v	40 $\phi_v V_n$	35 V_n/Ω_v	35 $\phi_v V_n$	30 V_n/Ω_v	30 $\phi_v V_n$
91.3	137	97.5	146	83.1	125

Available Strength in Flexure about Y-Y Axis, kip-ft

40 M_{ny}/Ω_b	40 $\phi_b M_{ny}$	35 M_{ny}/Ω_b	35 $\phi_b M_{ny}$	30 M_{ny}/Ω_b	30 $\phi_b M_{ny}$
54.5	81.9	37.3	56.1	31.0	46.6

Limiting Unbraced Lengths, ft

40 L_p	40 L_r	35 L_p	35 L_r	30 L_p	30 L_r
6.01	17.6	4.77	13.9	4.71	13.2

Property	40	35	30
Area, in.^{2}	11.7	10.3	8.79
r_y, in.	1.94	1.54	1.52
r_x/r_y	2.64	3.41	3.43

Moment of Inertia, in.^{4}

40 I_x	40 I_y	35 I_x	35 I_y	30 I_x	30 I_y
307	44.1	285	24.5	238	20.3

[c] Shape is slender for compression with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W12

F_y = 65 ksi
F_u = 80 ksi

Shape	W12× 26^c		W12× 22^c		W12× 19^c		W12× 26^f		W12× 22		W12× 19	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Design	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	272	409	231	348	192	288	119	179	95.0	143	80.1	120
6	224	337	127	191	104	156	113	170	70.4	106	57.3	86.2
7	209	315	99.2	149	80.2	120	108	162	63.1	94.8	50.7	76.2
8	193	290	76.0	114	61.4	92.3	102	153	54.6	82.1	41.9	63.0
9	176	265	60.0	90.3	48.5	72.9	96.5	145	45.3	68.1	34.5	51.9
10	160	240	48.6	73.1	39.3	59.0	90.9	137	38.6	58.0	29.2	43.9
11	143	214	40.2	60.4	32.5	48.8	85.3	128	33.5	50.4	25.2	37.9
12	125	189	33.8	50.8	27.3	41.0	79.8	120	29.6	44.5	22.1	33.3
13	108	162	28.8	43.3	23.2	34.9	72.8	109	26.5	39.8	19.7	29.6
14	92.9	140	24.8	37.3			64.4	96.8	24.0	36.0	17.7	26.7
15	80.9	122					57.5	86.5	21.9	32.9	16.1	24.2
16	71.1	107					51.9	78.0	20.1	30.3	14.8	22.2
17	63.0	94.7					47.2	70.9	18.6	28.0	13.6	20.5
18	56.2	84.5					43.2	64.9	17.4	26.1	12.7	19.0
19	50.4	75.8					39.8	59.9	16.2	24.4	11.8	17.8
20	45.5	68.4					36.9	55.5	15.3	23.0	11.1	16.7
22	37.6	56.5					32.1	48.3	13.6	20.5	9.86	14.8
24	31.6	47.5					28.4	42.8	12.3	18.5	8.88	13.3
26							25.5	38.3	11.3	16.9	8.08	12.2
28							23.1	34.7	10.4	15.6	7.42	11.2
30							21.1	31.8	9.60	14.4	6.86	10.3
32							19.5	29.3	8.95	13.4	6.39	9.60
34							18.0	27.1	8.38	12.6	5.97	8.97
36							16.8	25.3	7.88	11.8	5.61	8.43
38							15.8	23.7	7.44	11.2	5.29	7.95
40							14.8	22.3	7.04	10.6	5.00	7.52
42							14.0	21.0	6.69	10.1	4.75	7.14
44							13.3	19.9	6.37	9.58	4.52	6.79
46							12.6	18.9	6.08	9.14	4.31	6.48
48							12.0	18.0	5.82	8.74	4.12	6.19
50							11.5	17.2	5.58	8.38	3.95	5.93

Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending

Properties

	W12× 26		W12× 22		W12× 19	
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	298	448	252	379	217	326
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	230	344	194	292	167	251
Available Strength in Shear, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	73.0	109	83.1	125	74.5	112
Available Strength in Flexure about Y-Y Axis, kip-ft	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	25.9	39.0	11.9	17.8	9.67	14.5

	W12× 26		W12× 22		W12× 19	
Limiting Unbraced Lengths, ft	L_p	L_r	L_p	L_r	L_p	L_r
	4.99	12.7	2.63	7.74	2.55	7.36
Area, in.^{2}	7.65		6.48		5.57	
Moment of Inertia, in.^{4}	I_x	I_y	I_x	I_y	I_x	I_y
	204	17.3	156	4.66	130	3.76
r_y, in.	1.51		0.848		0.822	
r_x/r_y	3.42		5.79		5.86	

[c] Shape is slender for compression with F_y = 65 ksi.

[f] Shape exceeds compact limit for flexure with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W12–W10

F_y = 65 ksi
F_u = 80 ksi

W12×				W10×		Shape	W12×				W10×	
16^{c}		14^{c}		112		lb/ft	16^{v}		$14^{f,v}$		112	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Design	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
156	235	133	199	1280	1920	0	65.2	98.0	55.1	82.8	477	717
79.3	119	65.2	98.0	1200	1800	6	44.2	66.4	37.4	56.2	477	717
60.0	90.1	50.2	75.5	1170	1750	7	38.0	57.2	31.0	46.6	477	717
45.9	69.0	38.5	57.8	1130	1700	8	30.1	45.3	24.4	36.7	477	717
36.3	54.5	30.4	45.7	1100	1650	9	24.7	37.1	19.9	29.9	474	712
29.4	44.2	24.6	37.0	1060	1590	10	20.7	31.1	16.7	25	469	705
24.3	36.5	20.3	30.6	1020	1530	11	17.8	26.7	14.2	21.4	464	698
20.4	30.7	17.1	25.7	973	1460	12	15.5	23.3	12.4	18.6	460	691
				928	1390	13	13.8	20.7	10.9	16.4	455	684
				881	1320	14	12.3	18.5	9.76	14.7	450	677
				834	1250	15	11.2	16.8	8.80	13.2	446	670
				786	1180	16	10.2	15.3	8.01	12.0	441	663
				738	1110	17	9.37	14.1	7.35	11.0	437	656
				691	1040	18	8.67	13.0	6.78	10.2	432	649
				644	967	19	8.07	12.1	6.30	9.46	427	642
				597	898	20	7.55	11.3	5.87	8.83	423	635
				509	765	22	6.68	10.0	5.18	7.79	414	622
				428	644	24	6.00	9.02	4.64	6.97	404	608
				365	548	26	5.45	8.18	4.20	6.31	395	594
				315	473	28	4.99	7.50	3.83	5.76	386	580
				274	412	30	4.60	6.92	3.53	5.31	377	566
				241	362	32	4.27	6.42	3.27	4.92	367	552
				213	321	34	3.99	6.00	3.05	4.59	358	538
				190	286	36	3.74	5.62	2.86	4.30	349	524
				171	257	38	3.52	5.30	2.69	4.04	340	511
				154	232	40	3.33	5.01	2.54	3.82	330	497
				140	210	42	3.16	4.75	2.41	3.61	321	483
				127	191	44	3.00	4.51	2.29	3.43	312	469
						46	2.86	4.30	2.18	3.27	303	455
						48	2.74	4.11	2.08	3.12	294	441
						50	2.62	3.94	1.99	2.99	284	426

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
183	276	162	243	1280	1920
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
141	212	125	187	987	1480
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
61.7	92.7	55	82.6	224	336
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
7.32	11.0	5.95	8.95	224	337

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
2.39	6.92	2.60	6.68	8.30	49.6
Area, in.^{2}					
4.71		4.16		32.9	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
103	2.82	88.6	2.36	716	236
r_y, in.					
0.773		0.753		2.68	
r_x/r_y					
6.04		6.14		1.74	

^{c} Shape is slender for compression with F_y = 65 ksi.
^{f} Shape exceeds compact limit for flexure with F_y = 65 ksi.
^{v} Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 65 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W10

F_y = 65 ksi
F_u = 80 ksi

W10×						Shape	W10×					
100		88		77		lb/ft	100		88		77	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1140	1710	1010	1520	884	1330	0	422	634	367	551	317	476
1060	1600	942	1420	821	1230	6	422	634	367	551	317	476
1040	1560	918	1380	800	1200	7	422	634	367	551	317	476
1010	1510	892	1340	776	1170	8	422	634	367	551	317	476
974	1460	862	1300	750	1130	9	418	628	363	545	312	469
938	1410	830	1250	722	1080	10	413	621	358	538	308	463
901	1350	796	1200	692	1040	11	409	615	354	532	304	456
861	1290	761	1140	660	992	12	404	608	349	525	299	449
820	1230	724	1090	627	943	13	400	601	345	518	295	443
778	1170	687	1030	594	893	14	395	594	340	511	290	436
736	1110	648	974	560	842	15	391	587	336	505	286	429
692	1040	610	916	526	791	16	386	580	331	498	281	423
649	976	571	859	492	740	17	381	573	327	491	277	416
606	912	533	801	458	689	18	377	567	322	484	272	409
564	848	495	745	425	639	19	372	560	318	477	268	403
523	786	459	689	393	591	20	368	553	313	471	264	396
444	667	388	583	331	497	22	359	539	304	457	255	383
373	560	326	490	278	418	24	350	525	295	444	246	369
318	478	278	417	237	356	26	340	512	286	430	237	356
274	412	239	360	204	307	28	331	498	277	417	228	343
239	359	209	313	178	267	30	322	484	268	403	219	329
210	315	183	276	156	235	32	313	470	259	389	210	316
186	279	162	244	139	208	34	304	457	250	376	201	303
166	249	145	218	124	186	36	295	443	241	362	192	288
149	224	130	195	111	167	38	286	429	232	349	181	272
134	202	117	176	100	150	40	276	415	223	335	171	257
122	183	106	160	90.8	136	42	267	402	212	318	162	244
111	167					44	258	388	202	303	155	232
						46	248	372	192	289	147	222
						48	237	356	184	277	141	212
						50	227	341	176	265	135	203

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1140	1710	1010	1520	884	1330
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
879	1320	780	1170	681	1020
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
196	294	170	255	146	219
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
198	297	172	259	149	224

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.21	44.8	8.15	39.9	8.05	35.5
Area, in.^{2}					
29.3		26.0		22.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
623	207	534	179	455	154
r_y, in.					
2.65		2.63		2.60	
r_x/r_y					
1.74		1.73		1.73	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W10

F_y = 65 ksi
F_u = 80 ksi

W10× 68		W10× 60		W10× 54		Shape lb/ft	W10× 68		W10× 60		W10× 54[f]	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
775	1160	689	1040	615	924	0	277	416	242	364	215	324
720	1080	639	961	570	857	6	277	416	242	364	215	324
701	1050	622	935	555	834	7	277	416	242	364	215	324
680	1020	603	907	538	809	8	277	416	242	363	215	324
657	987	582	875	519	780	9	272	409	238	357	212	318
632	949	560	842	499	750	10	268	403	233	351	207	312
605	909	536	806	478	718	11	264	396	229	344	203	305
577	868	511	768	455	684	12	259	390	225	338	199	299
549	825	485	730	432	649	13	255	383	220	331	195	293
519	780	459	690	408	614	14	251	377	216	325	191	287
489	736	432	650	384	578	15	246	370	212	318	186	280
459	690	405	609	360	542	16	242	364	208	312	182	274
429	646	379	569	336	505	17	238	357	203	306	178	268
400	601	352	529	313	470	18	233	350	199	299	174	261
371	557	326	490	289	435	19	229	344	195	293	170	255
342	515	301	452	267	401	20	224	337	191	286	166	249
288	433	252	379	223	336	22	216	324	182	274	157	236
242	364	212	318	188	282	24	207	311	173	261	149	224
206	310	181	271	160	240	26	198	298	165	248	140	211
178	267	156	234	138	207	28	190	285	156	235	130	196
155	233	136	204	120	180	30	181	272	146	220	120	180
136	205	119	179	106	159	32	172	259	136	204	111	167
121	181	106	159	93.5	141	34	161	242	127	190	103	155
108	162	94.2	142	83.4	125	36	151	227	119	178	96.7	145
96.5	145	84.5	127	74.8	112	38	142	214	112	168	91.0	137
87.1	131	76.3	115	67.6	102	40	134	202	105	159	85.8	129
79.0	119	69.2	104	61.3	92.1	42	128	192	100	150	81.3	122
						44	121	182	95.0	143	77.2	116
						46	116	174	90.5	136	73.5	110
						48	111	166	86.5	130	70.2	105
						50	106	159	82.8	124	67.1	101

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
775	1160	689	1040	615	924
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
597	896	531	797	474	711
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
127	191	111	167	97.2	146
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
130	195	114	171	101	152

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.02	32.1	7.96	29.2	8.11	27.0
Area, in.^{2}					
19.9		17.7		15.8	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
394	134	341	116	303	103
r_y, in.					
2.59		2.57		2.56	
r_x/r_y					
1.71		1.71		1.71	

[f] Shape exceeds compact limit for flexure with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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W10

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W10×						Shape	W10×					
49		45		39		lb/ft	49[f]		45		39	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
560	842	518	778	448	673	0	191	287	178	268	152	228
519	780	458	689	395	593	6	191	287	178	268	152	228
505	759	438	659	377	567	7	191	287	175	263	148	223
489	735	417	626	358	538	8	191	287	170	256	144	217
472	709	393	591	337	507	9	191	287	166	250	140	210
453	681	369	554	316	474	10	187	281	162	243	136	204
434	652	344	516	293	441	11	183	275	157	237	132	198
413	621	318	478	271	407	12	179	269	153	230	128	192
392	589	292	439	248	373	13	175	263	149	224	123	186
370	556	266	401	226	339	14	171	257	144	217	119	179
348	523	242	363	204	307	15	167	251	140	211	115	173
326	489	217	327	183	275	16	163	245	136	204	111	167
304	456	194	292	163	245	17	159	239	131	198	107	161
282	424	173	260	145	218	18	155	233	127	191	103	154
261	392	155	234	130	196	19	151	227	123	185	98.6	148
240	361	140	211	118	177	20	147	221	118	178	93.9	141
200	301	116	174	97.2	146	22	139	208	109	164	83.0	125
168	253	97.4	146	81.7	123	24	131	196	98.1	147	74.4	112
143	216	83.0	125	69.6	105	26	122	183	89.2	134	67.4	101
124	186	71.5	108	60.0	90.2	28	111	166	81.9	123	61.7	92.8
108	162	62.3	93.7	52.3	78.6	30	102	153	75.7	114	56.9	85.5
94.7	142	54.8	82.3	46.0	69.1	32	93.9	141	70.3	106	52.8	79.4
83.9	126					34	87.3	131	65.8	98.8	49.3	74.0
74.8	112					36	81.6	123	61.7	92.8	46.2	69.4
67.2	101					38	76.7	115	58.2	87.5	43.5	65.4
60.6	91.1					40	72.3	109	55.0	82.7	41.1	61.7
55.0	82.6					42	68.4	103	52.2	78.5	38.9	58.5
						44	64.9	97.5	49.7	74.7	37.0	55.6
						46	61.7	92.8	47.4	71.2	35.3	53.0
						48	58.9	88.5	45.3	68.1	33.7	50.7
						50	56.3	84.6	43.4	65.2	32.3	48.5

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
560	842	518	778	448	673
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
432	648	399	599	345	518
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
88.4	133	91.9	138	81.2	122
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
88.4	133	65.8	99.0	55.8	83.9

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.09	25.6	6.23	21.6	6.13	19.7
Area, in.^{2}					
14.4		13.3		11.5	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
272	93.4	248	53.4	209	45.0
r_y, in.					
2.54		2.01		1.98	
r_x/r_y					
1.71		2.15		2.16	

[f] Shape exceeds compact limit for flexure with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W10

F_y = 65 ksi
F_u = 80 ksi

W10×						Shape		W10×					
33		30		26^c		lb/ft		33^f		30		26	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
378	568	344	517	291	438	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	122	183	119	178	102	153
332	498	265	398	227	341		6	122	183	110	165	93.2	140
316	475	241	362	206	310		7	122	183	105	158	88.6	133
299	450	216	324	184	277		8	118	178	100	150	84.0	126
282	423	191	286	163	244		9	114	172	95.0	143	79.4	119
263	395	166	249	141	212		10	110	166	90.0	135	74.7	112
243	366	142	214	121	182		11	107	160	85.0	128	70.1	105
224	336	120	181	102	153		12	103	154	80.0	120	65.5	98.4
204	307	102	154	86.9	131		13	98.8	149	75.0	113	59.4	89.3
185	278	88.4	133	75.0	113		14	95.0	143	68.4	103	53.3	80.1
167	251	77.0	116	65.3	98.1		15	91.1	137	62.2	93.5	48.3	72.7
149	224	67.7	102	57.4	86.3		16	87.2	131	57.1	85.8	44.2	66.4
132	198	59.9	90.1	50.8	76.4		17	83.4	125	52.8	79.3	40.7	61.2
118	177	53.5	80.3	45.3	68.2		18	79.5	119	49.0	73.7	37.7	56.7
106	159	48.0	72.1	40.7	61.2		19	73.6	111	45.8	68.9	35.1	52.8
95.4	143	43.3	65.1	36.7	55.2		20	68.5	103	43.0	64.6	32.9	49.5
78.8	118	35.8	53.8	30.4	45.6		22	60.2	90.5	38.3	57.6	29.2	43.9
66.2	99.5						24	53.7	80.8	34.6	51.9	26.2	39.4
56.4	84.8						26	48.5	72.9	31.5	47.3	23.8	35.8
48.7	73.1						28	44.2	66.5	28.9	43.5	21.9	32.9
42.4	63.7						30	40.6	61.1	26.8	40.3	20.2	30.3
37.3	56.0						32	37.6	56.5	24.9	37.5	18.8	28.2
							34	35.0	52.6	23.3	35.1	17.5	26.3
							36	32.8	49.2	21.9	32.9	16.4	24.7
							38	30.8	46.3	20.7	31.1	15.5	23.3
							40	29.0	43.7	19.6	29.4	14.7	22.0
							42	27.5	41.3	18.6	27.9	13.9	20.9
							44	26.1	39.2	17.7	26.6	13.2	19.9
							46	24.9	37.4	16.9	25.4	12.6	18.9
							48	23.7	35.6	16.1	24.3	12.0	18.1
							50	22.7	34.1	15.5	23.2	11.5	17.3

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
378	568	344	517	296	445
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
291	437	265	398	228	342
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
73.4	110	81.9	123	69.6	104
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
43.3	65.1	28.7	43.1	24.3	36.6

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.04	18.0	4.24	13.3	4.21	12.5
Area, in.^{2}					
9.71		8.84		7.61	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
171	36.6	170	16.7	144	14.1
r_y, in.					
1.94		1.37		1.36	
r_x/r_y					
2.16		3.20		3.20	

[c] Shape is slender for compression with F_y = 65 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 65 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

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W10

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W10×						Shape		W10×					
22^c		19^c		17^c		lb/ft		22		19		17	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending		ASD	LRFD	ASD	LRFD	ASD	LRFD
244	366	212	318	185	278		0	84.3	127	70.1	105	60.7	91.2
190	286	115	172	97.4	146		6	76.4	115	53.6	80.5	44.9	67.5
173	260	90.9	137	75.9	114		7	72.2	109	48.5	72.9	40.3	60.6
154	231	70.0	105	58.1	87.3		8	68.0	102	43.5	65.4	34.9	52.5
135	203	55.3	83.1	45.9	69.0		9	63.8	95.9	36.8	55.3	29	43.6
117	175	44.8	67.3	37.2	55.9		10	59.6	89.6	31.5	47.4	24.7	37.2
99.0	149	37.0	55.7	30.7	46.2		11	55.4	83.2	27.5	41.4	21.5	32.3
83.2	125	31.1	46.8	25.8	38.8		12	50.1	75.3	24.4	36.7	19.0	28.5
70.9	107	26.5	39.9	22.0	33.1		13	44.2	66.5	21.9	33.0	17.0	25.5
61.1	91.9	22.9	34.4	19.0	28.5		14	39.5	59.3	19.9	30.0	15.4	23.1
53.3	80.0						15	35.6	53.5	18.3	27.4	14.0	21.1
46.8	70.4						16	32.4	48.7	16.8	25.3	12.9	19.4
41.5	62.3						17	29.7	44.7	15.6	23.5	12.0	18.0
37.0	55.6						18	27.4	41.2	14.6	21.9	11.1	16.7
33.2	49.9						19	25.5	38.3	13.7	20.6	10.4	15.7
30.0	45.0						20	23.8	35.7	12.9	19.4	9.81	14.7
24.8	37.2						22	21.0	31.5	11.5	17.4	8.76	13.2
							24	18.8	28.2	10.5	15.7	7.92	11.9
							26	17.0	25.5	9.57	14.4	7.23	10.9
							28	15.5	23.3	8.82	13.3	6.66	10.0
							30	14.3	21.5	8.19	12.3	6.17	9.27
							32	13.2	19.9	7.64	11.5	5.75	8.64
							34	12.3	18.5	7.16	10.8	5.39	8.09
							36	11.6	17.4	6.74	10.1	5.06	7.61
							38	10.9	16.3	6.36	9.56	4.78	7.19
							40	10.3	15.4	6.03	9.06	4.53	6.81
							42	9.73	14.6	5.73	8.61	4.30	6.46
							44	9.24	13.9	5.46	8.21	4.10	6.16
							46	8.80	13.2	5.21	7.84	3.91	5.88
							48	8.41	12.6	4.99	7.50	3.74	5.62
							50	8.05	12.1	4.78	7.19	3.58	5.39

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
253	380	219	329	194	292
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
195	292	169	253	150	225
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
63.6	95.5	66.3	99.5	63.0	94.5
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
19.8	29.7	10.9	16.3	9.08	13.7

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
4.12	11.6	2.71	8.17	2.62	7.75
Area, in.^{2}					
6.49		5.62		4.99	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
118	11.4	96.3	4.29	81.9	3.56
r_y, in.					
1.33		0.874		0.845	
r_x/r_y					
3.21		4.74		4.79	

[c] Shape is slender for compression with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W10–W8

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W10×				W8×		Shape		W10×				W8×	
15^c		12^c		67		lb/ft		15		12^f		67	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
161	242	121	182	767	1150	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	51.9	78.0	39.1	58.8	227	342
81.0	122	61.9	93.1	687	1030		6	37.0	55.7	28.1	42.2	227	342
61.6	92.6	46.5	69.8	660	993		7	32.8	49.3	24.3	36.5	226	340
47.2	70.9	35.6	53.5	631	948		8	27.1	40.8	19.2	28.9	223	335
37.3	56.0	28.1	42.3	599	901		9	22.4	33.7	15.7	23.7	220	331
30.2	45.4	22.8	34.2	565	850		10	19.0	28.6	13.2	19.9	217	326
25.0	37.5	18.8	28.3	530	797		11	16.5	24.7	11.4	17.1	214	322
21.0	31.5	15.8	23.8	495	743		12	14.5	21.8	9.92	14.9	211	317
17.9	26.9	13.5	20.3	458	689		13	12.9	19.4	8.79	13.2	208	313
				422	634		14	11.6	17.5	7.88	11.8	205	309
				386	581		15	10.6	15.9	7.13	10.7	202	304
				352	528		16	9.73	14.6	6.51	9.79	199	300
				318	478		17	9.00	13.5	5.99	9.00	196	295
				285	429		18	8.36	12.6	5.54	8.33	193	291
				256	385		19	7.81	11.7	5.16	7.76	190	286
				231	347		20	7.33	11.0	4.83	7.25	187	282
				191	287		22	6.54	9.82	4.27	6.43	181	273
				160	241		24	5.90	8.86	3.84	5.77	176	264
				137	205		26	5.37	8.08	3.48	5.24	170	255
				118	177		28	4.94	7.42	3.19	4.80	164	246
				103	154		30	4.57	6.87	2.94	4.43	158	237
				90.3	136		32	4.26	6.40	2.73	4.11	152	228
				79.9	120		34	3.98	5.98	2.55	3.84	146	219
							36	3.74	5.62	2.39	3.60	140	210
							38	3.53	5.31	2.26	3.39	133	200
							40	3.34	5.02	2.13	3.20	126	190
							42	3.17	4.77	2.02	3.04	120	180
							44	3.02	4.54	1.92	2.89	114	172
							46	2.88	4.33	1.83	2.75	109	164
							48	2.76	4.14	1.75	2.63	105	157
							50	2.64	3.97	1.68	2.52	100	151

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
172	258	138	207	767	1150
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
132	198	106	159	591	887
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
59.7	89.6	48.8	73.1	133	200
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
7.46	11.2	5.31	7.98	106	159

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
2.51	7.34	2.91	6.93	6.57	36.9
Area, in.^{2}					
4.41		3.54		19.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
68.9	2.89	53.8	2.18	272	88.6
r_y, in.					
0.810		0.785		2.12	
r_x/r_y					
4.88		4.97		1.75	

[c] Shape is slender for compression with F_y = 65 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 65 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

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W8

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W8×						Shape	W8×					
58		48		40		lb/ft	58		48		40	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
666	1000	549	825	455	684	0	194	292	159	239	129	194
595	895	490	736	405	608	6	194	292	159	239	129	194
572	859	470	706	388	583	7	193	289	157	236	127	191
546	820	448	674	369	555	8	190	285	154	232	124	187
518	778	425	638	349	524	9	187	281	152	228	122	183
488	733	400	601	328	493	10	184	276	149	224	119	179
457	687	374	562	306	460	11	181	272	146	219	116	175
426	640	348	523	284	426	12	178	267	143	215	113	170
394	592	322	483	261	393	13	175	263	140	211	111	166
362	544	295	444	239	359	14	172	258	137	206	108	162
331	498	269	405	217	327	15	169	254	134	202	105	158
301	452	244	367	196	295	16	166	250	131	198	102	154
271	408	220	331	176	264	17	163	245	129	193	99.4	149
243	365	197	295	157	236	18	160	241	126	189	96.7	145
218	328	176	265	141	212	19	157	236	123	185	93.9	141
197	296	159	239	127	191	20	154	232	120	180	91.1	137
163	244	132	198	105	158	22	148	223	114	172	85.6	129
137	205	111	166	88.2	133	24	143	214	109	163	79.7	120
116	175	94.2	142	75.2	113	26	137	206	103	154	72.7	109
100	151	81.2	122	64.8	97.4	28	131	197	96.6	145	66.8	100
87.5	131	70.7	106	56.5	84.9	30	125	188	89.6	135	61.8	92.9
76.9	116	62.2	93.5	49.6	74.6	32	119	179	83.6	126	57.6	86.5
68.1	102	55.1	82.8	44.0	66.1	34	112	168	78.3	118	53.9	81.0
						36	106	159	73.7	111	50.6	76.1
						38	99.7	150	69.6	105	47.8	71.8
						40	94.6	142	65.9	99.1	45.2	67.9
						42	89.9	135	62.7	94.2	42.9	64.5
						44	85.7	129	59.7	89.7	40.9	61.4
						46	81.8	123	57.0	85.7	39.0	58.6
						48	78.3	118	54.5	82.0	37.3	56.0
						50	75.1	113	52.3	78.6	35.7	53.7

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
666	1000	549	825	455	684
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
513	770	423	635	351	527
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
116	174	88.4	133	77.2	116
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
90.5	136	74.3	112	60.0	90.2

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
6.51	32.4	6.44	27.6	6.32	23.8
Area, in.^{2}					
17.1		14.1		11.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
228	75.1	184	60.9	146	49.1
r_y, in.					
2.10		2.08		2.04	
r_x/r_y					
1.74		1.74		1.73	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W8

F_y = 65 ksi
F_u = 80 ksi

W8×						Shape	W8×					
35		31		28		lb/ft	35[f]		31[f]		28	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
401	603	355	534	321	483	0	112	169	95.4	143	88.2	133
356	535	315	473	266	400	6	112	169	95.4	143	85.5	129
341	512	301	453	249	374	7	111	166	95.4	143	82.7	124
324	487	287	431	230	346	8	108	162	94.1	141	80.0	120
306	460	271	407	210	316	9	105	158	91.5	137	77.2	116
288	432	254	382	191	286	10	102	154	88.8	134	74.4	112
268	403	237	356	171	257	11	99.8	150	86.2	130	71.7	108
248	373	219	329	152	228	12	97.1	146	83.6	126	68.9	104
229	344	202	303	133	200	13	94.4	142	81.0	122	66.1	99.4
209	314	184	277	115	173	14	91.7	138	78.4	118	63.4	95.2
190	285	167	251	100	151	15	89.0	134	75.8	114	60.6	91.1
171	257	151	226	88.3	133	16	86.2	130	73.2	110	57.8	86.9
153	230	135	202	78.2	118	17	83.5	126	70.6	106	55.0	82.6
137	206	120	180	69.8	105	18	80.8	121	68.0	102	51.2	76.9
123	184	108	162	62.6	94.1	19	78.1	117	65.4	98.3	47.9	71.9
111	166	97.2	146	56.5	84.9	20	75.4	113	62.8	94.4	45.0	67.6
91.5	138	80.3	121	46.7	70.2	22	69.5	105	55.7	83.7	40.1	60.3
76.9	116	67.5	101	39.2	59.0	24	62.6	94.1	49.9	75.0	36.3	54.5
65.5	98.5	57.5	86.5	33.4	50.2	26	56.9	85.5	45.3	68.1	33.1	49.8
56.5	84.9	49.6	74.5			28	52.2	78.5	41.4	62.3	30.5	45.8
49.2	74.0	43.2	64.9			30	48.2	72.5	38.2	57.4	28.2	42.4
43.3	65.0	38.0	57.1			32	44.8	67.4	35.5	53.3	26.3	39.5
						34	41.9	63.0	33.1	49.8	24.6	37.0
						36	39.3	59.1	31.0	46.7	23.1	34.8
						38	37.1	55.7	29.2	43.9	21.8	32.8
						40	35.1	52.7	27.6	41.5	20.7	31.1
						42	33.3	50.0	26.2	39.3	19.6	29.5
						44	31.7	47.6	24.9	37.4	18.7	28.1
						46	30.2	45.4	23.7	35.7	17.8	26.8
						48	28.9	43.4	22.7	34.1	17.1	25.7
						50	27.6	41.5	21.7	32.6	16.4	24.6

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
401	603	355	534	321	483
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
309	464	274	411	248	371
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
65.4	98.2	59.3	88.9	59.7	89.6
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
52.1	78.2	43.5	65.4	32.8	49.2

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
6.38	21.7	7.49	20.1	5.02	17.0
Area, in.^{2}					
10.3		9.13		8.25	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
127	42.6	110	37.1	98.0	21.7
r_y, in.					
2.03		2.02		1.62	
r_x/r_y					
1.73		1.72		2.13	

[f] Shape exceeds compact limit for flexure with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W8

W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W8×						Shape		W8×					
24		21		18		lb/ft		24^f		21		18	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
276	414	240	360	205	308	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	74.7	112	66.2	99.5	55.1	82.9
228	342	176	264	148	222		6	72.3	109	59.9	90.0	49.1	73.8
213	320	157	236	131	198		7	69.7	105	56.9	85.5	46.3	69.6
197	295	138	208	115	172		8	67.0	101	53.9	81.0	43.5	65.4
180	270	119	179	98.4	148		9	64.4	96.8	50.9	76.5	40.8	61.3
163	244	101	152	82.8	125		10	61.8	92.9	47.9	72.0	38.0	57.1
145	219	84.4	127	68.6	103		11	59.2	89.0	44.9	67.5	35.2	52.9
129	194	70.9	107	57.7	86.7		12	56.6	85.1	41.9	62.9	31.4	47.1
113	170	60.4	90.8	49.2	73.9		13	54.0	81.1	37.8	56.8	27.9	42.0
97.7	147	52.1	78.3	42.4	63.7		14	51.4	77.2	34.2	51.4	25.2	37.8
85.1	128	45.4	68.2	36.9	55.5		15	48.7	73.3	31.2	46.9	22.9	34.4
74.8	112	39.9	59.9	32.4	48.8		16	45.4	68.2	28.7	43.2	21.0	31.5
66.3	99.6	35.3	53.1	28.7	43.2		17	41.9	63.0	26.6	40.0	19.4	29.1
59.1	88.9	31.5	47.4	25.6	38.5		18	38.9	58.4	24.8	37.3	18.0	27.1
53.1	79.8	28.3	42.5	23.0	34.6		19	36.3	54.5	23.2	34.9	16.8	25.3
47.9	72.0	25.5	38.4	20.8	31.2		20	34.0	51.1	21.8	32.8	15.8	23.7
39.6	59.5						22	30.3	45.5	19.5	29.3	14.0	21.1
33.3	50.0						24	27.2	41.0	17.6	26.5	12.6	19.0
28.3	42.6						26	24.8	37.3	16.1	24.2	11.5	17.3
							28	22.8	34.2	14.8	22.3	10.6	15.9
							30	21.1	31.6	13.7	20.6	9.79	14.7
							32	19.6	29.4	12.8	19.2	9.11	13.7
							34	18.3	27.5	12.0	18.0	8.52	12.8
							36	17.2	25.8	11.3	17.0	8.00	12.0
							38	16.2	24.4	10.6	16.0	7.55	11.3
							40	15.3	23.1	10.1	15.2	7.14	10.7
							42	14.6	21.9	9.58	14.4	6.78	10.2
							44	13.9	20.8	9.12	13.7	6.45	9.69
							46	13.2	19.9	8.71	13.1	6.15	9.25
							48	12.6	19.0	8.33	12.5	5.88	8.84
							50	12.1	18.2	7.98	12.0	5.64	8.47

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
276	414	240	360	205	308
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
212	319	185	277	158	237
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
50.5	75.8	53.8	80.7	48.7	73.0
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
27.7	41.6	18.5	27.7	15.1	22.7

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.06	15.5	3.90	12.2	3.81	11.3
Area, in.^{2}					
7.08		6.16		5.26	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
82.7	18.3	75.3	9.77	61.9	7.97
r_y, in.					
1.61		1.26		1.23	
r_x/r_y					
2.12		2.77		2.79	

[f] Shape exceeds compact limit for flexure with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W8

F_y = 65 ksi
F_u = 80 ksi

W8× 15		W8× 13		W8× 10^c		Shape lb/ft		W8× 15		W8× 13		W8× 10^f	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending		ASD	LRFD	ASD	LRFD	ASD	LRFD
173	260	149	225	107	162		0	44.1	66.3	37.0	55.6	27.4	41.2
90.9	137	74.7	112	57.4	86.3		6	34.1	51.2	27.6	41.4	20.8	31.2
72.1	108	58.1	87.4	44.6	67.0		7	31.0	46.6	24.8	37.2	18.4	27.7
55.6	83.5	44.5	66.9	34.1	51.3		8	28.0	42.0	21.7	32.6	15.1	22.7
43.9	66.0	35.2	52.9	27.0	40.5		9	24.1	36.2	18.1	27.2	12.5	18.7
35.6	53.5	28.5	42.8	21.9	32.8		10	20.7	31.2	15.5	23.3	10.5	15.8
29.4	44.2	23.5	35.4	18.1	27.1		11	18.2	27.3	13.5	20.3	9.11	13.7
24.7	37.1	19.8	29.7	15.2	22.8		12	16.2	24.3	12.0	18.0	8.00	12.0
21.0	31.6	16.9	25.3	12.9	19.4		13	14.6	21.9	10.7	16.1	7.12	10.7
18.1	27.3	14.5	21.8	11.1	16.8		14	13.3	20.0	9.75	14.6	6.41	9.64
							15	12.2	18.3	8.92	13.4	5.83	8.76
							16	11.3	16.9	8.23	12.4	5.35	8.03
							17	10.5	15.7	7.63	11.5	4.93	7.42
							18	9.79	14.7	7.12	10.7	4.58	6.89
							19	9.19	13.8	6.67	10.0	4.28	6.43
							20	8.67	13.0	6.28	9.44	4.01	6.03
							22	7.78	11.7	5.63	8.46	3.57	5.36
							24	7.06	10.6	5.10	7.66	3.22	4.83
							26	6.47	9.72	4.66	7.00	2.93	4.40
							28	5.97	8.97	4.29	6.45	2.69	4.04
							30	5.54	8.33	3.98	5.99	2.49	3.74
							32	5.17	7.78	3.72	5.58	2.31	3.48
							34	4.85	7.29	3.48	5.23	2.16	3.25
							36	4.57	6.87	3.28	4.92	2.03	3.06
							38	4.32	6.49	3.09	4.65	1.92	2.88
							40	4.09	6.15	2.93	4.41	1.81	2.73
							42	3.89	5.85	2.79	4.19	1.72	2.59
							44	3.71	5.57	2.65	3.99	1.64	2.46
							46	3.54	5.32	2.53	3.81	1.56	2.35
							48	3.39	5.10	2.42	3.64	1.49	2.25
							50	3.25	4.89	2.32	3.49	1.43	2.15

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
173	260	149	225	115	173
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
133	200	115	173	88.8	133
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
51.7	77.5	47.8	71.7	34.9	52.3
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
8.66	13.0	6.97	10.5	5.02	7.55

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
2.71	8.38	2.61	7.82	3.17	7.28
Area, in.^{2}					
4.44		3.84		2.96	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
48.0	3.41	39.6	2.73	30.8	2.09
r_y, in.					
0.876		0.843		0.841	
r_x/r_y					
3.76		3.81		3.83	

[c] Shape is slender for compression with F_y = 65 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 65 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W6

F_y = 65 ksi
F_u = 80 ksi

W6×						Shape	W6×					
25		20		15		lb/ft	25		20^f		15^f	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
286	429	228	343	172	259	0	61.3	92.1	48.3	72.7	31.6	47.5
231	347	184	276	136	205	6	59.2	88.9	46.5	69.8	31.6	47.5
214	321	170	255	125	188	7	57.5	86.4	44.8	67.4	31.5	47.3
196	294	155	233	114	171	8	55.8	83.9	43.2	65.0	30.1	45.2
177	266	140	210	102	153	9	54.2	81.4	41.6	62.5	28.6	43.0
158	237	124	187	89.9	135	10	52.5	78.9	40.0	60.1	27.2	40.9
139	210	109	164	78.4	118	11	50.8	76.4	38.4	57.7	25.8	38.8
122	183	95.1	143	67.5	101	12	49.2	73.9	36.8	55.2	24.4	36.6
105	157	81.6	123	57.5	86.5	13	47.5	71.4	35.1	52.8	23.0	34.5
90.3	136	70.3	106	49.6	74.6	14	45.8	68.9	33.5	50.4	21.2	31.9
78.7	118	61.3	92.1	43.2	64.9	15	44.2	66.4	31.9	48.0	19.3	29.0
69.1	104	53.9	80.9	38.0	57.1	16	42.5	63.9	30.2	45.4	17.6	26.5
61.2	92.1	47.7	71.7	33.6	50.6	17	40.9	61.4	28.0	42.1	16.3	24.5
54.6	82.1	42.5	64.0	30.0	45.1	18	39.2	58.9	26.1	39.3	15.1	22.7
49.0	73.7	38.2	57.4	26.9	40.5	19	37.4	56.2	24.5	36.8	14.1	21.2
44.3	66.5	34.5	51.8	24.3	36.5	20	35.3	53.0	23.1	34.7	13.2	19.9
36.6	55.0	28.5	42.8	20.1	30.2	22	31.7	47.6	20.6	31.0	11.7	17.7
30.7	46.2	23.9	36.0	16.9	25.4	24	28.8	43.3	18.7	28.1	10.6	15.9
						26	26.4	39.7	17.1	25.7	9.62	14.5
						28	24.4	36.7	15.8	23.7	8.83	13.3
						30	22.7	34.1	14.6	22.0	8.17	12.3
						32	21.2	31.8	13.6	20.5	7.59	11.4
						34	19.9	29.9	12.8	19.2	7.10	10.7
						36	18.7	28.1	12.0	18.1	6.67	10.0
						38	17.7	26.6	11.4	17.1	6.29	9.45
						40	16.8	25.2	10.8	16.2	5.95	8.94
						42	15.9	24.0	10.2	15.4	5.64	8.48
						44	15.2	22.8	9.73	14.6	5.37	8.07
						46	14.5	21.8	9.30	14.0	5.12	7.70
						48	13.9	20.9	8.89	13.4	4.90	7.36
						50	13.3	20.0	8.53	12.8	4.69	7.05

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
286	429	228	343	172	259
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
220	330	176	264	133	199
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
53.1	79.6	41.9	62.9	35.8	53.7
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
27.8	41.7	21.6	32.5	13.2	19.8

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
4.71	18.8	4.84	15.9	6.91	13.6
Area, in.^{2}					
7.34		5.87		4.43	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
53.4	17.1	41.4	13.3	29.1	9.32
r_y, in.					
1.52		1.50		1.45	
r_x/r_y					
1.78		1.77		1.77	

[f] Shape exceeds compact limit for flexure with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

W6

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 65 ksi
F_u = 80 ksi

W6×						Shape	W6×					
16		12		9		lb/ft	16		12		9^f	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
184	277	138	208	104	157	0	37.9	57.0	26.9	40.5	19.6	29.4
109	164	77.0	116	57.2	85.9	6	32.6	48.9	21.8	32.8	15.7	23.6
90	135	62.3	93.7	46.0	69.1	7	30.8	46.3	20.2	30.3	14.3	21.5
72.3	109	48.8	73.3	35.8	53.8	8	29.0	43.6	18.5	27.9	12.9	19.4
57.1	85.8	38.6	57.9	28.3	42.5	9	27.2	40.9	16.9	25.4	10.9	16.4
46.3	69.5	31.2	46.9	22.9	34.4	10	25.4	38.2	14.8	22.3	9.36	14.1
38.2	57.5	25.8	38.8	18.9	28.5	11	23.6	35.5	13.1	19.7	8.17	12.3
32.1	48.3	21.7	32.6	15.9	23.9	12	21.4	32.2	11.7	17.6	7.25	10.9
27.4	41.1	18.5	27.8	13.6	20.4	13	19.5	29.4	10.6	15.9	6.52	9.80
23.6	35.5	15.9	23.9	11.7	17.6	14	17.9	27.0	9.70	14.6	5.92	8.90
20.6	30.9	13.9	20.9	10.2	15.3	15	16.6	24.9	8.94	13.4	5.42	8.15
18.1	27.2					16	15.4	23.2	8.29	12.5	5.01	7.52
						17	14.5	21.7	7.73	11.6	4.65	6.98
						18	13.6	20.4	7.24	10.9	4.34	6.52
						19	12.8	19.2	6.82	10.2	4.07	6.12
						20	12.1	18.2	6.44	9.68	3.83	5.76
						22	10.9	16.5	5.80	8.71	3.43	5.16
						24	9.99	15.0	5.28	7.93	3.11	4.68
						26	9.19	13.8	4.84	7.28	2.85	4.28
						28	8.50	12.8	4.48	6.73	2.63	3.95
						30	7.92	11.9	4.16	6.26	2.44	3.66
						32	7.41	11.1	3.89	5.85	2.27	3.42
						34	6.96	10.5	3.65	5.49	2.13	3.20
						36	6.57	9.87	3.44	5.17	2.00	3.01
						38	6.21	9.34	3.25	4.89	1.89	2.85
						40	5.90	8.86	3.09	4.64	1.79	2.70
						42	5.61	8.43	2.94	4.41	1.71	2.56
						44	5.35	8.04	2.80	4.21	1.62	2.44
						46	5.12	7.69	2.67	4.02	1.55	2.33
						48	4.90	7.36	2.56	3.85	1.48	2.23
						50	4.70	7.07	2.46	3.69	1.42	2.14

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
184	277	138	208	104	157
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
142	213	107	160	80.4	121
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
42.5	63.7	36.1	54.1	26.1	39.1
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
11.0	16.5	7.52	11.3	5.31	7.99

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
3.00	11.2	2.84	9.20	3.27	8.18
Area, in.^{2}					
4.74		3.55		2.68	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
32.1	4.43	22.1	2.99	16.4	2.20
r_y, in.					
0.967		0.918		0.905	
r_x/r_y					
2.69		2.71		2.73	

^{f} Shape exceeds compact limit for flexure with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W6–W5

F_y = 65 ksi
F_u = 80 ksi

W6×		W5×				Shape	W6×		W5×			
8.5		19		16		lb/ft	8.5^f		19		16	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
98.1	147	216	325	183	276	0	17.5	26.3	37.6	56.6	31.2	46.9
52.7	79.1	160	241	134	202	6	14.2	21.4	35.5	53.4	29.1	43.8
42.1	63.2	144	216	120	181	7	12.9	19.4	34.5	51.8	28.1	42.3
32.6	48.9	127	191	106	159	8	11.5	17.3	33.5	50.3	27.1	40.7
25.7	38.7	110	165	91.2	137	9	9.63	14.5	32.4	48.7	26.1	39.2
20.8	31.3	93.9	141	77.4	116	10	8.23	12.4	31.4	47.2	25.1	37.7
17.2	25.9	78.6	118	64.5	97.0	11	7.18	10.8	30.3	45.6	24.1	36.2
14.5	21.7	66.0	99.2	54.2	81.5	12	6.36	9.56	29.3	44.1	23.1	34.7
12.3	18.5	56.3	84.6	46.2	69.4	13	5.71	8.58	28.3	42.5	22.1	33.2
10.6	16.0	48.5	72.9	39.8	59.9	14	5.18	7.78	27.2	41.0	21.1	31.7
		42.3	63.5	34.7	52.1	15	4.74	7.12	26.2	39.4	20.1	30.1
		37.1	55.8	30.5	45.8	16	4.37	6.56	25.2	37.8	18.9	28.4
		32.9	49.5	27.0	40.6	17	4.05	6.09	24.1	36.3	17.7	26.5
		29.3	44.1	24.1	36.2	18	3.78	5.68	23.1	34.7	16.6	24.9
		26.3	39.6	21.6	32.5	19	3.54	5.32	21.8	32.7	15.6	23.5
		23.8	35.7	19.5	29.3	20	3.33	5.01	20.6	31.0	14.8	22.2
						22	2.98	4.49	18.6	28.0	13.3	20.0
						24	2.70	4.06	17.0	25.6	12.1	18.3
						26	2.47	3.71	15.6	23.5	11.2	16.8
						28	2.28	3.42	14.5	21.8	10.3	15.5
						30	2.11	3.17	13.5	20.3	9.61	14.4
						32	1.97	2.96	12.6	19.0	8.99	13.5
						34	1.85	2.77	11.9	17.8	8.44	12.7
						36	1.74	2.61	11.2	16.8	7.96	12.0
						38	1.64	2.46	10.6	15.9	7.53	11.3
						40	1.55	2.34	10.0	15.1	7.14	10.7
						42	1.48	2.22	9.56	14.4	6.80	10.2
						44	1.41	2.11	9.12	13.7	6.48	9.74
						46	1.34	2.02	8.72	13.1	6.19	9.31
						48	1.28	1.93	8.35	12.5	5.93	8.92
						50	1.23	1.85	8.01	12.0	5.69	8.55

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
98.1	147	216	325	183	276
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
75.6	113	167	250	141	212
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
25.8	38.7	36.2	54.2	31.3	46.9
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
4.62	6.95	17.9	27.0	14.9	22.3

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
3.59	7.99	3.97	18.0	3.90	15.6
Area, in.^{2}					
2.52		5.56		4.71	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
14.9	1.99	26.3	9.13	21.4	7.51
r_y, in.					
0.890		1.28		1.26	
r_x/r_y					
2.73		1.70		1.69	

^{f} Shape exceeds compact limit for flexure with F_y = 65 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W4

F_y = 65 ksi
F_u = 80 ksi

W4×		Shape		W4×	
13		lb/ft		13	
P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips				Available Flexural Strength, kip-ft	
ASD	LRFD			ASD	LRFD
149	224	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	20.4	30.6
91.1	137		6	18.4	27.7
76.2	115		7	17.8	26.7
62.1	93.3		8	17.1	25.7
49.4	74.2		9	16.4	24.7
40	60.1		10	15.7	23.7
33	49.7		11	15.1	22.6
27.8	41.7		12	14.4	21.6
23.7	35.6		13	13.7	20.6
20.4	30.7		14	13.1	19.6
17.8	26.7		15	12.4	18.6
15.6	23.5		16	11.6	17.4
			17	10.8	16.3
			18	10.2	15.3
			19	9.64	14.5
			20	9.14	13.7
			22	8.28	12.4
			24	7.57	11.4
			26	6.97	10.5
			28	6.46	9.72
			30	6.03	9.06
			32	5.64	8.48
			34	5.31	7.98
			36	5.01	7.53
			38	4.74	7.13
			40	4.50	6.77
			42	4.29	6.44
			44	4.09	6.15
			46	3.91	5.88
			48	3.75	5.63
			50	3.59	5.40

Properties

Available Strength in Tensile Yielding, kips		Limiting Unbraced Lengths, ft	
P_n/Ω_t	$\phi_t P_n$	L_p	L_r
149	224	3.10	15.0
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		Area, in.^{2}	
P_n/Ω_t	$\phi_t P_n$	3.83	
115	172	Moment of Inertia, in.^{4}	
Available Strength in Shear, kips		I_x	I_y
V_n/Ω_v	$\phi_v V_n$	11.3	3.86
30.3	45.4	r_y, in.	
Available Strength in Flexure about Y-Y Axis, kip-ft		1.00	
M_{ny}/Ω_b	$\phi_b M_{ny}$	r_x/r_y	
9.47	14.2	1.72	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

W44

Table IV-6B
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W44×						Shape		W44×					
335^c		290^c		262^c		lb/ft		335		290		262^v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending		ASD	LRFD	ASD	LRFD	ASD	LRFD
3870	5820	3210	4830	2830	4260		0	5660	8510	4930	7400	4440	6670
3730	5610	3100	4650	2730	4100		6	5660	8510	4930	7400	4440	6670
3680	5530	3050	4590	2690	4040		7	5660	8510	4930	7400	4440	6670
3620	5440	3010	4520	2650	3980		8	5660	8510	4930	7400	4440	6670
3560	5350	2950	4440	2600	3900		9	5660	8510	4930	7400	4440	6670
3490	5250	2890	4350	2550	3830		10	5660	8510	4930	7400	4440	6670
3410	5130	2830	4260	2490	3740		11	5600	8410	4870	7320	4380	6580
3330	5010	2760	4150	2430	3650		12	5490	8250	4770	7170	4290	6440
3250	4880	2690	4050	2370	3560		13	5380	8090	4670	7030	4190	6300
3160	4750	2620	3930	2300	3460		14	5280	7930	4580	6880	4100	6160
3070	4610	2540	3820	2230	3350		15	5170	7770	4480	6730	4010	6030
2970	4460	2460	3700	2160	3240		16	5060	7610	4380	6590	3920	5890
2870	4310	2380	3570	2080	3130		17	4960	7450	4290	6440	3830	5750
2770	4160	2290	3440	2010	3020		18	4850	7290	4190	6290	3730	5610
2660	4000	2200	3310	1930	2900		19	4740	7130	4090	6150	3640	5470
2540	3820	2120	3180	1850	2780		20	4640	6970	3990	6000	3550	5340
2300	3450	1940	2910	1700	2550		22	4420	6650	3800	5710	3370	5060
2060	3090	1760	2650	1540	2310		24	4210	6320	3610	5420	3180	4780
1820	2740	1580	2370	1380	2080		26	3990	6000	3410	5130	3000	4510
1600	2400	1380	2080	1230	1850		28	3780	5680	3220	4830	2820	4230
1390	2090	1210	1810	1080	1620		30	3570	5360	3010	4530	2580	3870
1220	1840	1060	1590	948	1420		32	3300	4960	2710	4070	2310	3480
1080	1630	939	1410	839	1260		34	3000	4510	2460	3700	2090	3150
966	1450	838	1260	749	1130		36	2750	4140	2250	3380	1910	2870
867	1300	752	1130	672	1010		38	2540	3820	2070	3110	1750	2630
783	1180	679	1020	606	911		40	2360	3550	1920	2880	1620	2430
710	1070	616	925	550	827		42	2200	3310	1780	2680	1500	2260
647	972	561	843	501	753		44	2060	3100	1660	2500	1400	2100
592	890	513	771	459	689		46	1940	2910	1560	2350	1310	1970
544	817	471	708	421	633		48	1830	2750	1470	2210	1230	1850
501	753	434	653	388	583		50	1730	2600	1390	2090	1160	1740

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4130	6210	3580	5380	3240	4860
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3320	4990	2880	4320	2610	3910
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1270	1900	1060	1580	855	1280
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
824	1240	716	1080	636	956

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.4	31.1	10.4	29.9	10.4	29.1
Area, in.^{2}					
98.5		85.4		77.2	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
31100	1200	27000	1040	24100	923
r_y, in.					
3.49		3.49		3.47	
r_x/r_y					
5.10		5.10		5.10	

[c] Shape is slender for compression with F_y = 70 ksi.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.
Note: Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W44–W40

F_y = 70 ksi
F_u = 90 ksi

W44×		W40×				Shape		W44×		W40×			
230^c		655^h		593^h		lb/ft		230^v		655^h		593^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending		ASD	LRFD	ASD	LRFD	ASD	LRFD
2410	3630	8090	12200	7290	11000		0	3840	5780	10800	16200	9640	14500
2320	3490	7810	11700	7030	10600		6	3840	5780	10800	16200	9640	14500
2290	3440	7710	11600	6940	10400		7	3840	5780	10800	16200	9640	14500
2250	3380	7590	11400	6830	10300		8	3840	5780	10800	16200	9640	14500
2210	3320	7470	11200	6710	10100		9	3840	5780	10800	16200	9640	14500
2160	3250	7330	11000	6590	9900		10	3840	5780	10800	16200	9640	14500
2110	3180	7180	10800	6450	9690		11	3780	5680	10800	16200	9640	14500
2060	3100	7020	10500	6300	9460		12	3700	5560	10700	16100	9570	14400
2010	3020	6840	10300	6140	9230		13	3620	5440	10600	15900	9460	14200
1950	2930	6660	10000	5970	8970		14	3540	5310	10500	15800	9350	14100
1890	2840	6480	9730	5800	8710		15	3450	5190	10400	15600	9240	13900
1830	2750	6280	9440	5620	8440		16	3370	5070	10300	15400	9130	13700
1760	2650	6080	9140	5430	8160		17	3290	4950	10100	15300	9020	13600
1700	2550	5870	8820	5240	7880		18	3210	4820	10000	15100	8910	13400
1630	2450	5660	8510	5050	7580		19	3130	4700	9930	14900	8810	13200
1560	2350	5450	8190	4850	7290		20	3050	4580	9820	14800	8700	13100
1430	2150	5010	7530	4450	6690		22	2880	4330	9590	14400	8480	12700
1290	1940	4580	6880	4050	6090		24	2720	4090	9370	14100	8260	12400
1160	1750	4140	6230	3660	5500		26	2560	3840	9150	13800	8040	12100
1030	1550	3720	5600	3280	4920		28	2390	3600	8930	13400	7820	11800
917	1380	3320	4990	2910	4370		30	2130	3200	8700	13100	7610	11400
813	1220	2930	4410	2560	3850		32	1910	2870	8480	12700	7390	11100
720	1080	2600	3900	2270	3410		34	1720	2590	8260	12400	7170	10800
642	966	2320	3480	2020	3040		36	1570	2350	8040	12100	6950	10400
577	867	2080	3120	1820	2730		38	1430	2150	7820	11700	6730	10100
520	782	1880	2820	1640	2460		40	1320	1980	7590	11400	6510	9790
472	709	1700	2560	1490	2230		42	1220	1830	7370	11100	6300	9460
430	646	1550	2330	1350	2040		44	1130	1700	7150	10700	6080	9140
393	591	1420	2130	1240	1860		46	1060	1590	6930	10400	5860	8810
361	543	1300	1960	1140	1710		48	991	1490	6700	10100	5620	8440
333	501	1200	1800	1050	1580		50	932	1400	6480	9740	5360	8060

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2840	4270	8090	12200	7290	11000
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2290	3430	6510	9770	5870	8810
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
723	1090	2400	3610	2160	3230
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
548	824	1890	2850	1680	2530

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.2	28.2	11.5	51.3	11.3	47.3
Area, in.^{2}					
67.8		193		174	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
20800	796	56500	2870	50400	2520
r_y, in.					
3.43		3.86		3.80	
r_x/r_y					
5.10		4.43		4.47	

[c] Shape is slender for compression with F_y = 70 ksi.

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Note: Confirm ASTM A913 material availability before specifying.

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W40

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W40× 503^h P_n/Ω_c	W40× 503^h $\phi_c P_n$	W40× 431^h P_n/Ω_c	W40× 431^h $\phi_c P_n$	W40× 397^h P_n/Ω_c	W40× 397^h $\phi_c P_n$	Shape lb/ft Design: Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	W40× 503^h M_{nx}/Ω_b	W40× 503^h $\phi_b M_{nx}$	W40× 431^h M_{nx}/Ω_b	W40× 431^h $\phi_b M_{nx}$	W40× 397^h M_{nx}/Ω_b	W40× 397^h $\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
6200	9320	5320	8000	4900	7370	0	8100	12200	6850	10300	6290	9450
5970	8970	5120	7690	4710	7080	6	8100	12200	6850	10300	6290	9450
5890	8850	5040	7580	4640	6980	7	8100	12200	6850	10300	6290	9450
5790	8710	4960	7450	4570	6860	8	8100	12200	6850	10300	6290	9450
5690	8550	4870	7320	4480	6740	9	8100	12200	6850	10300	6290	9450
5580	8380	4770	7160	4390	6590	10	8100	12200	6850	10300	6290	9450
5450	8200	4660	7000	4290	6440	11	8100	12200	6840	10300	6270	9430
5320	8000	4540	6820	4180	6280	12	8010	12000	6730	10100	6180	9280
5180	7790	4420	6640	4060	6110	13	7900	11900	6630	9970	6080	9140
5030	7570	4290	6440	3940	5930	14	7790	11700	6530	9820	5980	8990
4880	7340	4150	6240	3820	5740	15	7690	11600	6430	9660	5880	8840
4720	7100	4010	6030	3690	5540	16	7580	11400	6330	9510	5780	8690
4560	6850	3870	5810	3560	5340	17	7470	11200	6230	9360	5690	8540
4390	6600	3720	5590	3420	5140	18	7360	11100	6120	9200	5590	8400
4220	6350	3570	5370	3280	4930	19	7260	10900	6020	9050	5490	8250
4050	6090	3420	5140	3140	4720	20	7150	10700	5920	8900	5390	8100
3700	5570	3120	4680	2860	4300	22	6940	10400	5720	8590	5190	7810
3360	5050	2810	4230	2580	3880	24	6720	10100	5510	8290	5000	7510
3020	4540	2520	3790	2310	3470	26	6510	9780	5310	7980	4800	7220
2690	4050	2240	3360	2050	3080	28	6290	9460	5110	7670	4610	6920
2380	3570	1960	2950	1800	2700	30	6080	9140	4900	7370	4410	6630
2090	3140	1720	2590	1580	2380	32	5860	8810	4700	7060	4210	6330
1850	2780	1530	2300	1400	2100	34	5650	8490	4500	6760	4020	6040
1650	2480	1360	2050	1250	1880	36	5440	8170	4290	6450	3820	5740
1480	2230	1220	1840	1120	1680	38	5220	7850	4070	6110	3550	5340
1340	2010	1100	1660	1010	1520	40	5010	7530	3810	5720	3320	4990
1210	1820	1000	1500	917	1380	42	4770	7170	3580	5370	3110	4680
1100	1660	912	1370	836	1260	44	4510	6780	3370	5070	2930	4410
1010	1520	835	1250	765	1150	46	4280	6430	3190	4800	2770	4170
928	1390	767	1150	702	1060	48	4070	6110	3030	4550	2630	3950
855	1290	706	1060	647	973	50	3880	5830	2880	4330	2500	3760

Properties

503^h		431^h		397^h	
Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
6200	9320	5320	8000	4900	7370
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
5000	7490	4290	6430	3950	5920
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1820	2720	1550	2320	1400	2100
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
1380	2070	1150	1720	1050	1580

503^h		431^h		397^h	
Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.1	41.5	10.9	37.6	10.9	36.1
Area, in.^{2}					
148		127		117	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
41600	2040	34800	1690	32000	1540
r_y, in.					
3.72		3.65		3.64	
r_x/r_y					
4.52		4.55		4.56	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

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W40

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W40×						Shape	W40×					
372^h		362^{h,c}		324^c		lb/ft	372^h		362^h		324	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
4610	6930	4440	6670	3880	5830	0	5870	8820	5730	8610	5100	7670
4430	6650	4260	6410	3740	5620	6	5870	8820	5730	8610	5100	7670
4360	6550	4200	6320	3690	5550	7	5870	8820	5730	8610	5100	7670
4290	6440	4130	6210	3630	5460	8	5870	8820	5730	8610	5100	7670
4200	6320	4050	6090	3570	5370	9	5870	8820	5730	8610	5100	7670
4120	6180	3970	5960	3510	5270	10	5870	8820	5730	8610	5100	7670
4020	6040	3870	5820	3430	5160	11	5840	8780	5700	8570	5070	7620
3910	5880	3770	5670	3350	5040	12	5750	8640	5610	8430	4980	7490
3800	5720	3670	5510	3270	4920	13	5650	8490	5510	8290	4890	7350
3690	5550	3560	5340	3180	4780	14	5560	8350	5420	8140	4800	7220
3570	5370	3440	5170	3080	4630	15	5460	8200	5320	8000	4710	7080
3450	5180	3320	4990	2980	4470	16	5360	8060	5230	7860	4620	6950
3320	4990	3200	4810	2860	4310	17	5270	7920	5130	7710	4530	6810
3190	4790	3070	4620	2750	4140	18	5170	7770	5040	7570	4440	6680
3060	4600	2950	4430	2640	3960	19	5070	7630	4940	7420	4350	6540
2930	4400	2820	4240	2520	3790	20	4980	7480	4840	7280	4260	6410
2660	4000	2560	3850	2290	3440	22	4780	7190	4650	6990	4080	6140
2390	3600	2310	3470	2060	3100	24	4590	6900	4460	6710	3900	5860
2140	3210	2060	3100	1840	2760	26	4400	6610	4270	6420	3720	5590
1890	2840	1820	2740	1620	2440	28	4210	6320	4080	6130	3540	5320
1650	2490	1590	2390	1420	2130	30	4010	6030	3890	5850	3360	5050
1450	2180	1400	2100	1250	1870	32	3820	5740	3700	5560	3180	4780
1290	1930	1240	1860	1100	1660	34	3630	5450	3510	5270	2940	4410
1150	1730	1110	1660	984	1480	36	3380	5080	3250	4880	2700	4060
1030	1550	993	1490	883	1330	38	3140	4710	3020	4530	2500	3760
930	1400	896	1350	797	1200	40	2930	4400	2810	4230	2330	3500
844	1270	813	1220	723	1090	42	2740	4120	2640	3960	2180	3270
769	1160	741	1110	659	990	44	2580	3880	2480	3730	2050	3070
703	1060	678	1020	603	906	46	2440	3660	2340	3520	1930	2900
646	971	622	935	553	832	48	2310	3470	2220	3330	1820	2740
595	895	574	862	510	766	50	2190	3300	2110	3170	1730	2600

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4610	6930	4440	6680	3990	6000
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3710	5570	3580	5370	3220	4820
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1320	1980	1270	1910	1130	1690
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
968	1450	943	1420	835	1250

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.7	34.6	10.7	34.4	10.7	32.6
Area, in.^{2}					
110		106		95.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
29600	1420	28900	1380	25600	1220
r_y, in.					
3.60		3.60		3.58	
r_x/r_y					
4.58		4.58		4.58	

[c] Shape is slender for compression with F_y = 70 ksi.

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

W40

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

Shape	W40×						W40×					
lb/ft	297^c		277^c		249^c		297		277		249^v	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending												
0	3480	5240	3170	4770	2790	4200	4650	6980	4370	6560	3910	5880
6	3360	5050	3060	4600	2690	4050	4650	6980	4370	6560	3910	5880
7	3310	4980	3020	4540	2660	3990	4650	6980	4370	6560	3910	5880
8	3260	4900	2970	4470	2620	3930	4650	6980	4370	6560	3910	5880
9	3210	4820	2920	4390	2570	3860	4650	6980	4370	6560	3910	5880
10	3140	4730	2870	4310	2520	3790	4650	6980	4370	6560	3910	5880
11	3080	4620	2810	4220	2470	3710	4610	6930	4340	6520	3880	5830
12	3010	4520	2740	4120	2410	3620	4520	6800	4260	6400	3810	5720
13	2930	4400	2670	4020	2350	3530	4440	6670	4180	6280	3730	5600
14	2850	4280	2600	3910	2280	3430	4350	6540	4090	6150	3650	5490
15	2770	4160	2530	3800	2220	3330	4270	6410	4010	6030	3580	5380
16	2680	4030	2450	3680	2150	3220	4180	6280	3930	5910	3500	5260
17	2590	3890	2370	3560	2070	3120	4090	6150	3850	5780	3420	5150
18	2500	3750	2290	3440	2000	3010	4010	6020	3770	5660	3350	5030
19	2390	3600	2200	3310	1930	2890	3920	5890	3680	5540	3270	4920
20	2290	3440	2120	3180	1850	2780	3840	5770	3600	5410	3200	4800
22	2070	3110	1940	2920	1700	2550	3660	5510	3440	5170	3040	4570
24	1860	2790	1760	2650	1540	2320	3490	5250	3270	4920	2890	4350
26	1650	2480	1570	2360	1390	2090	3320	4990	3110	4670	2740	4120
28	1460	2190	1390	2080	1230	1850	3150	4730	2940	4430	2590	3890
30	1270	1910	1210	1820	1070	1610	2980	4480	2780	4180	2430	3660
32	1120	1680	1060	1600	944	1420	2770	4160	2570	3860	2190	3300
34	988	1480	943	1420	836	1260	2530	3800	2340	3520	1990	2990
36	881	1320	841	1260	746	1120	2320	3490	2150	3220	1820	2740
38	791	1190	755	1130	670	1010	2150	3220	1980	2970	1680	2520
40	714	1070	681	1020	604	908	1990	3000	1840	2760	1550	2330
42	647	973	618	929	548	824	1860	2800	1710	2570	1440	2170
44	590	887	563	846	499	751	1740	2620	1600	2410	1350	2030
46	540	811	515	774	457	687	1640	2470	1510	2260	1270	1900
48	496	745	473	711	420	631	1550	2330	1420	2140	1190	1790
50	457	687	436	655	387	581	1470	2210	1340	2020	1130	1690

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3660	5500	3420	5130	3080	4630
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2950	4420	2750	4130	2480	3720
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1040	1550	923	1380	743	1120
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
751	1130	713	1070	636	956

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.6	31.4	10.7	31.1	10.6	30.1
Area, in.^{2}					
87.3		81.5		73.5	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
23200	1090	21900	1040	19600	926
r_y, in.					
3.54		3.58		3.55	
r_x/r_y					
4.60		4.58		4.59	

[c] Shape is slender for compression with F_y = 70 ksi.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.
Note: Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W40

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W40×						Shape	W40×					
215^c		199^c		392^h		lb/ft	215^v		199^v		392^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2340	3510	2140	3220	4860	7310	0	3370	5060	3040	4560	5970	8980
2250	3380	2060	3090	4510	6770	6	3370	5060	3040	4560	5970	8980
2220	3340	2030	3050	4380	6590	7	3370	5060	3040	4560	5970	8980
2190	3290	2000	3000	4250	6380	8	3370	5060	3040	4560	5960	8960
2150	3230	1960	2950	4100	6160	9	3370	5060	3040	4560	5840	8780
2110	3160	1920	2890	3940	5910	10	3370	5060	3040	4560	5730	8610
2060	3100	1880	2820	3760	5660	11	3340	5020	2990	4490	5610	8440
2010	3020	1830	2750	3590	5390	12	3270	4910	2930	4400	5500	8260
1960	2940	1780	2680	3400	5110	13	3200	4810	2860	4300	5380	8090
1900	2860	1730	2600	3210	4830	14	3130	4710	2800	4200	5270	7910
1850	2780	1680	2520	3020	4540	15	3060	4610	2730	4110	5150	7740
1790	2690	1620	2440	2830	4250	16	3000	4500	2670	4010	5030	7570
1730	2600	1560	2350	2640	3970	17	2930	4400	2600	3910	4920	7390
1670	2510	1510	2260	2450	3680	18	2860	4300	2540	3820	4800	7220
1600	2410	1450	2170	2270	3410	19	2790	4200	2480	3720	4690	7050
1540	2320	1390	2080	2090	3140	20	2720	4090	2410	3620	4570	6870
1410	2120	1270	1900	1740	2620	22	2590	3890	2280	3430	4340	6520
1280	1930	1150	1720	1470	2200	24	2450	3680	2150	3240	4110	6180
1160	1740	1030	1550	1250	1880	26	2310	3480	2030	3040	3880	5830
1040	1560	916	1380	1080	1620	28	2180	3270	1900	2850	3650	5480
918	1380	812	1220	938	1410	30	1990	3000	1690	2540	3380	5090
811	1220	713	1070	824	1240	32	1790	2690	1510	2270	3120	4690
719	1080	632	950	730	1100	34	1620	2430	1370	2050	2900	4360
641	963	564	847	651	979	36	1470	2220	1240	1870	2710	4070
575	865	506	760	584	878	38	1350	2030	1140	1710	2540	3810
519	780	457	686	527	793	40	1250	1880	1050	1570	2390	3590
471	708	414	622	478	719	42	1160	1740	969	1460	2260	3390
429	645	377	567	436	655	44	1080	1620	901	1350	2140	3220
393	590	345	519			46	1010	1520	841	1260	2040	3060
361	542	317	477			48	947	1420	788	1190	1940	2920
332	499	292	439			50	892	1340	742	1110	1850	2790

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2660	4000	2460	3700	4860	7310
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2140	3210	1980	2980	3920	5870
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
627	943	622	935	1650	2480
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
545	819	479	719	727	1090

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.6	29.1	10.3	28.2	7.88	29.1
Area, in.^{2}					
63.5		58.8		116	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
16700	803	14900	695	29900	803
r_y, in.					
3.54		3.45		2.64	
r_x/r_y					
4.58		4.64		6.10	

[c] Shape is slender for compression with F_y = 70 ksi.
[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

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W40

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W40×						Shape	W40×					
331^h		327^h		294^c		lb/ft	331^h		327^h		294	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
4100	6160	4020	6040	3550	5340	0	5000	7510	4930	7400	4440	6670
3780	5680	3710	5580	3310	4980	6	5000	7510	4930	7400	4440	6670
3670	5520	3610	5420	3230	4850	7	5000	7510	4930	7400	4440	6670
3550	5340	3490	5240	3130	4700	8	4960	7450	4890	7350	4400	6610
3420	5140	3360	5050	3010	4520	9	4850	7290	4780	7190	4290	6450
3280	4920	3220	4840	2880	4330	10	4740	7120	4680	7030	4190	6290
3130	4700	3070	4620	2750	4130	11	4630	6960	4570	6870	4080	6130
2970	4460	2920	4390	2610	3920	12	4520	6790	4460	6700	3980	5970
2810	4220	2760	4160	2460	3700	13	4410	6620	4350	6540	3870	5820
2640	3970	2600	3910	2320	3480	14	4300	6460	4240	6380	3770	5660
2480	3730	2440	3670	2170	3260	15	4190	6290	4130	6210	3660	5500
2310	3480	2280	3430	2020	3040	16	4080	6130	4030	6050	3550	5340
2150	3230	2120	3190	1880	2820	17	3970	5960	3920	5890	3450	5190
1990	2990	1960	2950	1730	2610	18	3860	5800	3810	5730	3340	5030
1830	2750	1810	2720	1590	2400	19	3750	5630	3700	5560	3240	4870
1680	2520	1660	2490	1460	2190	20	3640	5470	3590	5400	3130	4710
1390	2090	1380	2070	1210	1820	22	3420	5130	3380	5070	2920	4400
1170	1760	1160	1740	1020	1530	24	3200	4800	3160	4750	2710	4080
996	1500	986	1480	865	1300	26	2980	4470	2940	4420	2450	3680
859	1290	850	1280	746	1120	28	2690	4050	2660	4000	2210	3320
748	1120	740	1110	650	977	30	2460	3690	2430	3650	2010	3020
658	989	651	978	571	859	32	2260	3400	2230	3360	1850	2770
583	876	576	866	506	761	34	2090	3140	2070	3110	1710	2560
520	781	514	773	451	679	36	1950	2930	1920	2890	1580	2380
466	701	461	694	405	609	38	1820	2740	1800	2710	1480	2220
421	633	416	626	366	550	40	1710	2570	1690	2540	1390	2090
382	574	378	568	332	499	42	1620	2430	1600	2400	1310	1970
						44	1530	2300	1510	2270	1240	1860
						46	1450	2180	1430	2150	1170	1760
						48	1380	2080	1360	2050	1120	1680
						50	1320	1980	1300	1960	1060	1600

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4100	6160	4020	6040	3610	5430
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3300	4950	3240	4850	2910	4360
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1390	2090	1350	2020	1200	1800
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
592	890	587	882	523	785

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.67	26.1	7.70	26.1	7.61	24.7
Area, in.^{2}					
97.7		95.9		86.2	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
24700	644	24500	640	21900	562
r_y, in.					
2.57		2.58		2.55	
r_x/r_y					
6.19		6.20		6.24	

[c] Shape is slender for compression with F_y = 70 ksi.

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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W40

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W40×						Shape	W40×					
278^c		264^c		235^c		lb/ft	278		264		235	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
3350	5040	3090	4650	2650	3990	0	4160	6250	3950	5930	3530	5300
3120	4700	2880	4330	2470	3720	6	4160	6250	3950	5930	3530	5300
3050	4580	2810	4220	2410	3620	7	4160	6250	3950	5930	3530	5300
2960	4440	2730	4100	2340	3520	8	4110	6180	3900	5860	3490	5250
2860	4300	2640	3960	2260	3400	9	4010	6020	3800	5710	3400	5110
2740	4110	2540	3820	2180	3280	10	3910	5870	3700	5570	3310	4970
2600	3920	2440	3660	2090	3150	11	3810	5720	3610	5420	3220	4830
2470	3710	2320	3490	2000	3010	12	3700	5570	3510	5270	3120	4700
2330	3500	2190	3290	1910	2860	13	3600	5420	3410	5120	3030	4560
2190	3290	2060	3090	1810	2720	14	3500	5270	3310	4980	2940	4420
2050	3080	1920	2890	1710	2570	15	3400	5110	3210	4830	2850	4280
1900	2860	1790	2690	1610	2420	16	3300	4960	3110	4680	2760	4150
1760	2650	1660	2490	1500	2250	17	3200	4810	3020	4530	2670	4010
1630	2440	1530	2300	1380	2080	18	3100	4660	2920	4390	2580	3870
1490	2240	1400	2110	1270	1910	19	3000	4510	2820	4240	2490	3740
1360	2050	1280	1930	1160	1750	20	2900	4350	2720	4090	2390	3600
1130	1690	1060	1590	961	1450	22	2700	4050	2530	3800	2210	3320
947	1420	891	1340	808	1210	24	2490	3740	2300	3460	1970	2960
807	1210	759	1140	688	1030	26	2220	3330	2050	3080	1740	2620
696	1050	654	984	594	892	28	2000	3000	1840	2760	1560	2350
606	911	570	857	517	777	30	1810	2730	1670	2510	1410	2120
533	801	501	753	454	683	32	1660	2500	1530	2300	1290	1940
472	709	444	667	403	605	34	1530	2300	1410	2120	1180	1780
421	633	396	595	359	540	36	1420	2140	1310	1960	1100	1650
378	568	355	534	322	484	38	1330	2000	1220	1830	1020	1530
341	512	321	482	291	437	40	1250	1870	1140	1710	953	1430
309	465	291	437	264	396	42	1170	1760	1070	1610	895	1350
						44	1110	1670	1010	1520	844	1270
						46	1050	1580	959	1440	798	1200
						48	998	1500	911	1370	757	1140
						50	951	1430	867	1300	720	1080

Properties

Available Strength in Tensile Yielding, kips						Limiting Unbraced Lengths, ft					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	L_p	L_r	L_p	L_r	L_p	L_r
3450	5180	3240	4880	2900	4350	7.52	24.0	7.52	23.5	7.58	22.8
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips						Area, in.^{2}					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	82.3		77.4		69.1	
2780	4170	2610	3920	2330	3500	Moment of Inertia, in.^{4}					
Available Strength in Shear, kips						I_x	I_y	I_x	I_y	I_x	I_y
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	20500	521	19400	493	17400	444
1160	1740	1080	1610	923	1380	r_y, in.					
Available Strength in Flexure about Y-Y Axis, kip-ft						2.52		2.52		2.54	
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	r_x/r_y					
487	732	461	693	412	620	6.27		6.27		6.26	

^{c} Shape is slender for compression with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W40

F_y = 70 ksi
F_u = 90 ksi

Shape		W40×						W40×					
lb/ft		211^c		183^c		167^c		211^v		183^v		167^v	
		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Design		Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	2320	3480	1910	2870	1740	2620	3160	4760	2700	4060	2420	3640
	6	2150	3240	1770	2670	1610	2420	3160	4760	2700	4060	2420	3640
	7	2100	3150	1730	2600	1570	2350	3160	4760	2700	4060	2420	3640
	8	2040	3060	1670	2520	1520	2280	3120	4690	2660	4000	2360	3550
	9	1970	2960	1620	2430	1460	2200	3040	4560	2580	3880	2290	3440
	10	1890	2850	1560	2340	1400	2110	2950	4430	2510	3770	2220	3330
	11	1820	2730	1490	2240	1340	2010	2860	4300	2430	3650	2140	3220
	12	1730	2610	1420	2140	1270	1910	2780	4170	2350	3540	2070	3110
	13	1650	2480	1350	2030	1210	1810	2690	4040	2270	3420	2000	3010
	14	1560	2350	1280	1920	1140	1710	2610	3920	2200	3300	1930	2900
	15	1470	2220	1210	1810	1070	1610	2520	3790	2120	3190	1860	2790
	16	1390	2080	1130	1700	1000	1500	2430	3660	2040	3070	1780	2680
	17	1300	1950	1060	1590	932	1400	2350	3530	1970	2960	1710	2570
	18	1210	1820	986	1480	864	1300	2260	3400	1890	2840	1640	2460
	19	1120	1680	914	1370	798	1200	2180	3270	1810	2720	1570	2350
	20	1020	1530	844	1270	732	1100	2090	3140	1740	2610	1490	2250
	22	844	1270	713	1070	612	920	1910	2870	1540	2310	1280	1930
	24	709	1070	599	900	515	773	1660	2500	1330	2000	1110	1660
	26	604	908	510	767	438	659	1470	2200	1170	1750	967	1450
	28	521	783	440	661	378	568	1310	1970	1040	1560	857	1290
	30	454	682	383	576	329	495	1180	1770	929	1400	767	1150
	32	399	599	337	506	289	435	1070	1610	842	1270	693	1040
	34	353	531	298	448	256	385	985	1480	769	1160	631	949
	36	315	474	266	400	229	344	909	1370	707	1060	579	870
	38	283	425	239	359	205	309	844	1270	654	982	535	803
	40	255	384	216	324	185	278	787	1180	608	914	496	746
	42							738	1110	568	854	463	695
	44							694	1040	533	801	433	651
	46							656	986	502	754	407	612
	48							621	934	474	713	384	578
	50							590	887	449	676	364	547

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2600	3910	2230	3360	2070	3110
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2100	3140	1800	2700	1660	2500
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
743	1120	627	943	621	933
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
367	551	308	464	265	399

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.49	22.0	7.43	21.1	7.16	20.4
Area, in.^{2}					
62.1		53.3		49.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
15500	390	13200	331	11600	283
r_y, in.					
2.51		2.49		2.40	
r_x/r_y					
6.29		6.31		6.38	

[c] Shape is slender for compression with F_y = 70 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W40–W36

F_y = 70 ksi
F_u = 90 ksi

W40×		W36×				Shape	W40×		W36×			
149^c		925^h		853^h		lb/ft	149^v		925^h		853^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1510	2260	11400	17100	10500	15800	0	2090	3140	14400	21700	13700	20600
1380	2080	11100	16600	10200	15400	6	2090	3140	14400	21700	13700	20600
1340	2020	11000	16500	10100	15200	7	2080	3120	14400	21700	13700	20600
1300	1950	10800	16300	9990	15000	8	2010	3020	14400	21700	13700	20600
1240	1870	10700	16000	9860	14800	9	1950	2920	14400	21700	13700	20600
1190	1790	10500	15800	9710	14600	10	1880	2830	14400	21700	13700	20600
1130	1700	10300	15500	9540	14300	11	1810	2730	14400	21700	13700	20600
1070	1610	10100	15200	9370	14100	12	1750	2630	14400	21700	13700	20600
1010	1520	9940	14900	9180	13800	13	1680	2530	14400	21600	13700	20500
952	1430	9720	14600	8990	13500	14	1620	2430	14300	21500	13600	20400
890	1340	9500	14300	8780	13200	15	1550	2330	14200	21400	13500	20300
828	1240	9260	13900	8560	12900	16	1480	2230	14100	21200	13400	20100
767	1150	9020	13600	8340	12500	17	1420	2130	14000	21100	13300	20000
707	1060	8760	13200	8110	12200	18	1350	2030	13900	20900	13200	19800
647	973	8500	12800	7870	11800	19	1290	1930	13800	20800	13100	19700
595	894	8240	12400	7630	11500	20	1190	1790	13700	20600	13000	19500
495	745	7700	11600	7130	10700	22	1010	1510	13500	20300	12800	19200
416	626	7140	10700	6620	9950	24	866	1300	13300	20100	12600	18900
355	533	6580	9900	6110	9180	26	755	1140	13200	19800	12400	18700
306	460	6030	9060	5600	8410	28	667	1000	13000	19500	12200	18400
266	400	5490	8250	5100	7660	30	595	895	12800	19200	12000	18100
234	352	4960	7460	4620	6940	32	537	806	12600	18900	11800	17800
207	312	4460	6700	4150	6240	34	487	733	12400	18600	11600	17500
185	278	3980	5980	3700	5570	36	446	670	12200	18300	11400	17200
166	250	3570	5360	3320	5000	38	411	617	12000	18000	11300	16900
		3220	4840	3000	4510	40	380	572	11800	17800	11100	16600
		2920	4390	2720	4090	42	354	532	11600	17500	10900	16300
		2660	4000	2480	3730	44	331	497	11400	17200	10700	16000
		2430	3660	2270	3410	46	310	466	11200	16900	10500	15800
		2240	3360	2080	3130	48	292	439	11000	16600	10300	15500
		2060	3100	1920	2890	50	276	415	10900	16300	10100	15200

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1840	2760	11400	17100	10500	15800
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1480	2220	9180	13800	8470	12700
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
577	867	3640	5470	3040	4560
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
217	326	2970	4460	2810	4230

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
6.84	19.5	12.7	76.8	12.8	72.2
Area, in.^{2}					
43.8		272		251	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
9800	229	73000	4940	70000	4600
r_y, in.					
2.29		4.26		4.28	
r_x/r_y					
6.55		3.85		3.90	

[c] Shape is slender for compression with F_y = 70 ksi.

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W36

F_y = 70 ksi
F_u = 90 ksi

W36×						Shape	W36×					
802^h		723^h		652^h		lb/ft	802^h		723^h		652^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
9890	14900	8930	13400	8050	12100	0	12800	19200	11400	17200	10200	15300
9600	14400	8660	13000	7800	11700	6	12800	19200	11400	17200	10200	15300
9500	14300	8560	12900	7710	11600	7	12800	19200	11400	17200	10200	15300
9380	14100	8460	12700	7610	11400	8	12800	19200	11400	17200	10200	15300
9250	13900	8340	12500	7500	11300	9	12800	19200	11400	17200	10200	15300
9110	13700	8200	12300	7370	11100	10	12800	19200	11400	17200	10200	15300
8950	13500	8060	12100	7240	10900	11	12800	19200	11400	17200	10200	15300
8780	13200	7900	11900	7090	10700	12	12800	19200	11400	17200	10200	15300
8600	12900	7740	11600	6940	10400	13	12700	19200	11400	17100	10100	15200
8410	12600	7560	11400	6780	10200	14	12600	19000	11300	16900	10000	15000
8210	12300	7380	11100	6610	9930	15	12600	18900	11200	16800	9910	14900
8000	12000	7190	10800	6430	9660	16	12500	18700	11100	16700	9810	14800
7790	11700	6990	10500	6250	9390	17	12400	18600	11000	16500	9720	14600
7570	11400	6780	10200	6060	9100	18	12300	18400	10900	16400	9630	14500
7340	11000	6570	9880	5860	8810	19	12200	18300	10800	16200	9540	14300
7100	10700	6360	9560	5670	8520	20	12100	18100	10700	16100	9440	14200
6630	9960	5920	8900	5260	7910	22	11900	17900	10500	15800	9260	13900
6140	9230	5480	8240	4860	7300	24	11700	17600	10300	15500	9070	13600
5650	8500	5030	7570	4450	6690	26	11500	17300	10100	15200	8880	13400
5170	7770	4590	6900	4050	6080	28	11300	17000	9940	14900	8700	13100
4700	7060	4160	6260	3660	5490	30	11100	16700	9750	14700	8510	12800
4240	6370	3750	5630	3280	4930	32	10900	16400	9560	14400	8320	12500
3790	5700	3340	5030	2910	4380	34	10700	16100	9370	14100	8140	12200
3380	5090	2980	4480	2600	3910	36	10500	15800	9180	13800	7950	12000
3040	4570	2680	4020	2330	3510	38	10300	15600	8990	13500	7770	11700
2740	4120	2420	3630	2110	3160	40	10200	15300	8800	13200	7580	11400
2490	3740	2190	3290	1910	2870	42	9960	15000	8610	12900	7390	11100
2270	3410	2000	3000	1740	2620	44	9770	14700	8420	12700	7210	10800
2070	3120	1830	2750	1590	2390	46	9580	14400	8230	12400	7020	10600
1900	2860	1680	2520	1460	2200	48	9390	14100	8040	12100	6840	10300
1750	2640	1550	2320	1350	2030	50	9200	13800	7850	11800	6650	9990

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
9890	14900	8930	13400	8050	12100
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
7970	11900	7190	10800	6480	9720
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
2840	4260	2540	3810	2270	3400
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
2600	3910	2300	3450	2030	3050

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.6	68.4	12.4	62.1	12.2	56.8
Area, in.^{2}					
236		213		192	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
64800	4210	57300	3700	50600	3230
r_y, in.					
4.22		4.17		4.10	
r_x/r_y					
3.93		3.93		3.95	

^{h} Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

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W36

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W36×						Shape		W36×					
529^h		487^h		441^h		lb/ft		529^h		487^h		441^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
6540	9830	5990	9010	5450	8190	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	8140	12200	7440	11200	6670	10000
6330	9510	5790	8710	5260	7910		6	8140	12200	7440	11200	6670	10000
6250	9390	5720	8600	5200	7810		7	8140	12200	7440	11200	6670	10000
6160	9270	5640	8480	5120	7700		8	8140	12200	7440	11200	6670	10000
6070	9120	5550	8350	5040	7580		9	8140	12200	7440	11200	6670	10000
5960	8960	5460	8200	4950	7440		10	8140	12200	7440	11200	6670	10000
5850	8790	5350	8040	4850	7290		11	8140	12200	7440	11200	6670	10000
5730	8610	5240	7870	4750	7130		12	8130	12200	7420	11200	6650	9990
5600	8410	5110	7690	4630	6960		13	8040	12100	7330	11000	6560	9860
5460	8200	4990	7490	4520	6790		14	7950	11900	7240	10900	6470	9730
5310	7990	4850	7290	4390	6600		15	7860	11800	7160	10800	6380	9600
5170	7760	4710	7080	4260	6410		16	7770	11700	7070	10600	6300	9470
5010	7530	4570	6870	4130	6210		17	7680	11500	6980	10500	6210	9330
4850	7290	4420	6640	3990	6000		18	7590	11400	6890	10300	6120	9200
4690	7050	4270	6420	3850	5790		19	7490	11300	6800	10200	6040	9070
4520	6800	4120	6190	3710	5580		20	7400	11100	6710	10100	5950	8940
4190	6290	3800	5720	3430	5150		22	7220	10900	6530	9810	5780	8680
3850	5780	3490	5240	3140	4710		24	7040	10600	6350	9540	5600	8420
3510	5270	3180	4770	2850	4280		26	6850	10300	6170	9270	5430	8160
3180	4770	2870	4310	2570	3860		28	6670	10000	5990	9000	5250	7900
2850	4290	2570	3870	2300	3450		30	6490	9750	5810	8730	5080	7630
2540	3820	2290	3440	2040	3060		32	6310	9480	5630	8460	4900	7370
2250	3390	2020	3040	1800	2710		34	6120	9200	5450	8190	4730	7110
2010	3020	1810	2710	1610	2420		36	5940	8930	5270	7920	4560	6850
1800	2710	1620	2440	1440	2170		38	5760	8650	5090	7650	4380	6590
1630	2450	1460	2200	1300	1960		40	5580	8380	4910	7380	4210	6320
1480	2220	1330	1990	1180	1780		42	5390	8110	4730	7110	4030	6060
1350	2020	1210	1820	1080	1620		44	5210	7830	4550	6840	3800	5720
1230	1850	1110	1660	985	1480		46	5030	7560	4340	6530	3600	5410
1130	1700	1020	1530	905	1360		48	4840	7270	4130	6200	3420	5140
1040	1570	936	1410	834	1250		50	4610	6930	3930	5910	3260	4890

						Properties						
Available Strength in Tensile Yielding, kips							Limiting Unbraced Lengths, ft					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		L_p	L_r	L_p	L_r	L_p	L_r
6540	9830	5990	9010	5450	8190		11.9	47.8	11.8	44.9	11.7	42.0
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips							Area, in.^{2}					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		156		143		130	
5270	7900	4830	7240	4390	6580		Moment of Inertia, in.^{4}					
Available Strength in Shear, kips							I_x	I_y	I_x	I_y	I_x	I_y
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$		39600	2490	36000	2250	32100	1990
1790	2690	1650	2480	1480	2220		r_y, in.					
Available Strength in Flexure about Y-Y Axis, kip-ft							4.00		3.96		3.92	
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$		r_x/r_y					
1590	2380	1440	2160	1290	1930		4.00		3.99		4.01	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W36

F_y = 70 ksi
F_u = 90 ksi

W36×						Shape	W36×					
395^h		361^h		330^c		lb/ft	395^h		361^h		330	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
4860	7310	4440	6680	4030	6060	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending 0	5970	8980	5410	8140	4930	7400
4690	7050	4290	6440	3900	5860	6	5970	8980	5410	8140	4930	7400
4630	6970	4230	6360	3860	5800	7	5970	8980	5410	8140	4930	7400
4570	6860	4170	6270	3810	5720	8	5970	8980	5410	8140	4930	7400
4490	6750	4100	6160	3740	5630	9	5970	8980	5410	8140	4930	7400
4410	6630	4020	6050	3670	5520	10	5970	8980	5410	8140	4930	7400
4320	6490	3940	5920	3600	5410	11	5970	8980	5410	8140	4930	7400
4220	6350	3850	5790	3510	5280	12	5940	8920	5370	8080	4880	7340
4120	6190	3760	5640	3430	5150	13	5850	8800	5290	7950	4800	7220
4010	6030	3660	5500	3340	5010	14	5770	8670	5210	7830	4720	7100
3900	5860	3550	5340	3240	4870	15	5680	8540	5130	7710	4650	6980
3780	5690	3440	5180	3140	4720	16	5600	8410	5050	7580	4570	6860
3660	5510	3330	5010	3040	4570	17	5510	8290	4960	7460	4490	6750
3540	5320	3220	4840	2930	4410	18	5430	8160	4880	7340	4410	6630
3410	5130	3100	4660	2830	4250	19	5340	8030	4800	7210	4330	6510
3290	4940	2980	4490	2720	4080	20	5260	7900	4720	7090	4250	6390
3030	4550	2750	4130	2500	3750	22	5090	7650	4550	6840	4100	6160
2770	4160	2510	3770	2280	3420	24	4920	7390	4390	6600	3940	5920
2510	3770	2270	3410	2060	3090	26	4750	7140	4230	6350	3780	5690
2260	3390	2040	3060	1850	2780	28	4580	6880	4060	6110	3630	5450
2010	3030	1820	2730	1640	2470	30	4410	6630	3900	5860	3470	5220
1780	2680	1600	2410	1450	2180	32	4240	6370	3740	5610	3310	4980
1580	2370	1420	2130	1280	1930	34	4070	6120	3570	5370	3160	4750
1410	2110	1270	1900	1140	1720	36	3900	5860	3410	5120	2980	4480
1260	1900	1140	1710	1030	1540	38	3730	5610	3220	4830	2760	4150
1140	1710	1030	1540	927	1390	40	3530	5300	3000	4510	2570	3870
1030	1550	930	1400	841	1260	42	3310	4970	2810	4230	2410	3620
942	1420	847	1270	766	1150	44	3120	4690	2650	3980	2260	3400
861	1290	775	1160	701	1050	46	2950	4430	2500	3750	2130	3200
791	1190	712	1070	644	968	48	2800	4200	2370	3560	2020	3030
729	1100	656	986	593	892	50	2660	4000	2250	3380	1910	2880

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4860	7310	4440	6680	4060	6100
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3920	5870	3580	5370	3270	4910
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1310	1970	1190	1790	1080	1620
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
1140	1710	1020	1540	926	1390

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.6	39.0	11.5	37.3	11.4	35.6
Area, in.^{2}					
116		106		96.9	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
28500	1750	25700	1570	23300	1420
r_y, in.					
3.88		3.85		3.83	
r_x/r_y					
4.05		4.05		4.05	

[c] Shape is slender for compression with F_y = 70 ksi.

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

W36

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W36×						Shape	W36×					
302^c		282^c		262^c		lb/ft	302		282		262	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
3630	5460	3330	5010	3060	4600	0	4470	6720	4160	6250	3840	5780
3520	5290	3230	4850	2960	4450	6	4470	6720	4160	6250	3840	5780
3480	5230	3190	4790	2930	4400	7	4470	6720	4160	6250	3840	5780
3430	5160	3150	4730	2890	4340	8	4470	6720	4160	6250	3840	5780
3380	5080	3100	4660	2840	4270	9	4470	6720	4160	6250	3840	5780
3320	5000	3050	4580	2790	4200	10	4470	6720	4160	6250	3840	5780
3260	4900	2990	4490	2740	4120	11	4470	6720	4160	6250	3840	5780
3200	4800	2930	4400	2680	4030	12	4430	6650	4110	6180	3790	5690
3130	4700	2860	4300	2620	3940	13	4350	6540	4040	6070	3720	5590
3050	4590	2790	4200	2560	3840	14	4280	6430	3970	5960	3650	5490
2970	4470	2720	4090	2490	3740	15	4210	6320	3900	5860	3580	5390
2880	4330	2650	3980	2420	3640	16	4130	6210	3820	5750	3510	5280
2790	4190	2570	3860	2350	3530	17	4060	6100	3750	5640	3450	5180
2690	4040	2490	3740	2270	3420	18	3980	5990	3680	5530	3380	5080
2590	3890	2400	3610	2200	3300	19	3910	5880	3610	5430	3310	4970
2490	3740	2310	3470	2120	3190	20	3840	5760	3540	5320	3240	4870
2290	3440	2120	3190	1950	2940	22	3690	5540	3400	5100	3100	4660
2080	3130	1930	2900	1770	2670	24	3540	5320	3250	4890	2960	4460
1880	2830	1740	2620	1600	2400	26	3390	5100	3110	4670	2830	4250
1690	2540	1560	2350	1430	2150	28	3240	4880	2970	4460	2690	4040
1500	2260	1390	2080	1270	1900	30	3100	4650	2820	4250	2550	3840
1320	1990	1220	1830	1110	1670	32	2950	4430	2680	4030	2410	3630
1170	1760	1080	1620	985	1480	34	2800	4210	2520	3790	2220	3330
1050	1570	964	1450	879	1320	36	2590	3900	2310	3470	2030	3050
939	1410	865	1300	789	1190	38	2400	3600	2130	3210	1870	2810
847	1270	781	1170	712	1070	40	2230	3350	1980	2970	1730	2600
768	1160	708	1060	646	971	42	2080	3130	1850	2770	1610	2420
700	1050	645	970	588	884	44	1950	2930	1730	2600	1510	2270
641	963	591	888	538	809	46	1840	2760	1630	2440	1420	2130
588	884	542	815	494	743	48	1730	2610	1530	2300	1330	2000
542	815	500	751	456	685	50	1640	2470	1450	2180	1260	1900

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3730	5610	3470	5220	3240	4860
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3000	4510	2800	4200	2610	3910
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
987	1480	919	1380	868	1300
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
842	1270	779	1170	713	1070

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.4	34.5	11.3	33.6	11.2	32.6
Area, in.^{2}					
89.0		82.9		77.2	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
21100	1300	19600	1200	17900	1090
r_y, in.					
3.82		3.80		3.76	
r_x/r_y					
4.03		4.05		4.07	

^{c} Shape is slender for compression with F_y = 70 ksi.

Note: Confirm ASTM A913 material availability before specifying.

W36

F_y = 70 ksi
F_u = 90 ksi

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W36×						Shape	W36×					
247^c		231^c		256^c		lb/ft	247		231		256	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2840	4270	2630	3960	3060	4600	0	3600	5410	3360	5060	3630	5460
2740	4130	2550	3830	2870	4310	6	3600	5410	3360	5060	3630	5460
2710	4080	2520	3780	2800	4210	7	3600	5410	3360	5060	3630	5460
2670	4020	2480	3730	2730	4100	8	3600	5410	3360	5060	3630	5450
2630	3960	2440	3670	2640	3980	9	3600	5410	3360	5060	3540	5320
2590	3890	2400	3600	2560	3840	10	3600	5410	3360	5060	3450	5190
2540	3810	2350	3530	2450	3680	11	3600	5410	3360	5060	3370	5060
2480	3730	2300	3460	2330	3510	12	3540	5320	3310	4970	3280	4940
2430	3650	2250	3380	2210	3330	13	3480	5220	3240	4870	3200	4810
2370	3560	2190	3290	2090	3140	14	3410	5130	3180	4780	3110	4680
2300	3460	2130	3210	1970	2960	15	3340	5030	3120	4680	3030	4550
2240	3370	2070	3110	1840	2770	16	3280	4930	3050	4590	2940	4420
2170	3260	2010	3020	1720	2590	17	3210	4830	2990	4490	2860	4290
2100	3160	1940	2920	1600	2400	18	3150	4730	2930	4400	2770	4170
2030	3050	1880	2820	1480	2220	19	3080	4630	2860	4300	2690	4040
1960	2950	1810	2720	1360	2050	20	3010	4530	2800	4210	2600	3910
1810	2730	1670	2510	1140	1710	22	2880	4330	2670	4020	2430	3650
1660	2490	1530	2310	958	1440	24	2750	4130	2550	3830	2260	3400
1490	2240	1390	2080	817	1230	26	2620	3930	2420	3640	2040	3070
1330	2000	1230	1860	704	1060	28	2480	3730	2290	3450	1840	2760
1180	1770	1090	1640	613	922	30	2350	3530	2170	3260	1670	2510
1030	1550	957	1440	539	810	32	2210	3320	2000	3010	1530	2290
916	1380	848	1270	477	718	34	2000	3010	1820	2730	1410	2120
817	1230	756	1140	426	640	36	1830	2750	1660	2490	1310	1960
733	1100	679	1020	382	575	38	1680	2530	1520	2290	1220	1830
662	994	612	920	345	518	40	1560	2340	1410	2120	1140	1710
600	902	555	835	313	470	42	1450	2180	1310	1960	1070	1610
547	822	506	761	285	429	44	1350	2030	1220	1830	1010	1520
500	752	463	696			46	1270	1910	1140	1720	960	1440
459	691	425	639			48	1200	1800	1070	1610	912	1370
423	636	392	589			50	1130	1700	1010	1520	868	1310

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3040	4570	2860	4300	3160	4740
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2450	3670	2300	3450	2540	3810
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
822	1230	777	1170	1010	1510
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
664	998	615	924	479	719

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.2	31.8	11.1	31.2	7.91	24.8
Area, in.^{2}					
72.5		68.2		75.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
16700	1010	15600	940	16800	528
r_y, in.					
3.74		3.71		2.65	
r_x/r_y					
4.06		4.07		5.62	

^{c} Shape is slender for compression with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W36

F_y = 70 ksi
F_u = 90 ksi

W36×						Shape		W36×					
232^c		210^c		194^c		lb/ft		232		210		194	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
2680	4030	2400	3610	2160	3240	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	3270	4910	2910	4370	2680	4030
2510	3780	2240	3370	2010	3030		6	3270	4910	2910	4370	2680	4030
2450	3690	2190	3290	1970	2950		7	3270	4910	2910	4370	2680	4030
2390	3590	2130	3200	1910	2870		8	3260	4890	2890	4340	2650	3990
2310	3480	2060	3100	1850	2780		9	3170	4770	2810	4230	2580	3880
2230	3360	1990	2990	1780	2680		10	3090	4650	2740	4110	2510	3770
2150	3230	1910	2880	1710	2570		11	3010	4530	2660	4000	2440	3660
2060	3100	1830	2750	1640	2460		12	2930	4410	2590	3890	2370	3560
1970	2960	1750	2630	1560	2350		13	2850	4290	2510	3770	2290	3450
1870	2810	1660	2500	1480	2230		14	2770	4170	2430	3660	2220	3340
1760	2640	1570	2370	1400	2110		15	2690	4050	2360	3540	2150	3230
1640	2470	1470	2210	1320	1990		16	2610	3930	2280	3430	2080	3130
1530	2300	1370	2060	1240	1870		17	2530	3800	2210	3320	2010	3020
1420	2140	1270	1900	1150	1730		18	2450	3680	2130	3200	1940	2910
1310	1970	1170	1750	1060	1590		19	2370	3560	2060	3090	1860	2800
1210	1810	1070	1610	972	1460		20	2290	3440	1980	2980	1790	2690
1010	1510	889	1340	806	1210		22	2130	3200	1830	2750	1650	2480
846	1270	747	1120	677	1020		24	1960	2950	1640	2460	1440	2170
721	1080	636	956	577	867		26	1740	2610	1440	2170	1270	1910
621	934	549	825	497	748		28	1560	2340	1290	1940	1130	1700
541	814	478	718	433	651		30	1410	2120	1160	1750	1020	1530
476	715	420	631	381	572		32	1290	1930	1060	1590	925	1390
421	633	372	559	337	507		34	1180	1780	970	1460	847	1270
376	565	332	499	301	452		36	1100	1650	895	1350	780	1170
337	507	298	448	270	406		38	1020	1530	831	1250	723	1090
305	458	269	404	244	366		40	954	1430	776	1170	674	1010
276	415	244	366	221	332		42	896	1350	727	1090	630	948
							44	844	1270	684	1030	593	891
							46	799	1200	646	971	559	840
							48	758	1140	612	920	529	795
							50	721	1080	581	874	502	754

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2850	4280	2590	3900	2390	3590
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2300	3440	2090	3130	1920	2890
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
904	1360	853	1280	782	1170
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
426	641	374	562	341	513

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.82	23.9	7.70	23.0	7.64	22.4
Area, in.^{2}					
68.0		61.9		57.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
15000	468	13200	411	12100	375
r_y, in.					
2.62		2.58		2.56	
r_x/r_y					
5.65		5.66		5.70	

^{c} Shape is slender for compression with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W36

F_y = 70 ksi
F_u = 90 ksi

W36×						Shape	W36×					
182^c		170^c		160^c		lb/ft	182		170^v		160^v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1990	3000	1830	2740	1690	2540	0	2510	3770	2330	3510	2180	3280
1860	2800	1700	2560	1570	2360	6	2510	3770	2330	3510	2180	3280
1810	2730	1660	2490	1530	2300	7	2510	3770	2330	3510	2180	3280
1760	2650	1610	2420	1490	2230	8	2480	3730	2300	3460	2150	3230
1710	2560	1560	2340	1440	2160	9	2410	3630	2240	3370	2080	3130
1650	2470	1500	2260	1380	2080	10	2340	3520	2170	3270	2020	3040
1580	2370	1440	2160	1330	1990	11	2280	3420	2110	3170	1960	2950
1510	2270	1380	2070	1270	1900	12	2210	3320	2040	3070	1900	2850
1440	2170	1310	1970	1200	1810	13	2140	3210	1980	2970	1840	2760
1370	2060	1240	1870	1140	1720	14	2070	3110	1910	2880	1770	2670
1290	1950	1180	1770	1080	1620	15	2000	3010	1850	2780	1710	2570
1220	1830	1110	1660	1010	1520	16	1930	2900	1780	2680	1650	2480
1140	1720	1040	1560	949	1430	17	1860	2800	1720	2580	1590	2390
1070	1610	970	1460	885	1330	18	1790	2700	1650	2480	1520	2290
991	1490	902	1360	822	1240	19	1730	2590	1590	2390	1460	2200
907	1360	834	1250	760	1140	20	1660	2490	1520	2290	1400	2110
752	1130	690	1040	634	952	22	1520	2280	1370	2060	1240	1870
632	949	580	872	532	800	24	1310	1970	1180	1780	1070	1610
538	809	494	743	454	682	26	1150	1730	1040	1560	936	1410
464	697	426	640	391	588	28	1020	1540	920	1380	829	1250
404	608	371	558	341	512	30	921	1380	825	1240	742	1120
355	534	326	490	299	450	32	835	1250	746	1120	670	1010
315	473	289	434	265	399	34	763	1150	681	1020	610	917
281	422	258	387	237	356	36	702	1050	625	940	560	841
252	379	231	348	212	319	38	650	976	578	869	516	776
227	342	209	314	192	288	40	604	908	537	807	479	720
206	310	189	285			42	565	849	501	753	447	671
						44	530	797	470	706	418	629
						46	500	751	442	665	393	591
						48	473	710	418	628	371	557
						50	448	674	396	595	351	528

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2250	3380	2100	3150	1970	2960
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1810	2710	1690	2530	1590	2380
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
737	1110	619	930	589	885
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
317	476	293	440	270	406

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.61	22.0	7.55	21.6	7.46	21.2
Area, in.^{2}					
53.6		50.0		47.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
11300	347	10500	320	9760	295
r_y, in.					
2.55		2.53		2.50	
r_x/r_y					
5.69		5.73		5.76	

[c] Shape is slender for compression with F_y = 70 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W36–W33

F_y = 70 ksi
F_u = 90 ksi

W-Shapes

W36×				W33×		Shape		W36×				W33×	
150^c		135^c		387^h		lb/ft		150^v		135^v		387^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending		ASD	LRFD	ASD	LRFD	ASD	LRFD
1570	2360	1380	2070	4780	7180		0	2030	3050	1780	2670	5450	8190
1460	2190	1270	1910	4600	6920		6	2030	3050	1780	2670	5450	8190
1420	2130	1240	1860	4540	6830		7	2030	3050	1780	2670	5450	8190
1370	2070	1200	1800	4470	6720		8	1990	2990	1730	2600	5450	8190
1330	2000	1150	1730	4390	6600		9	1930	2910	1680	2520	5450	8190
1280	1920	1110	1660	4310	6470		10	1870	2820	1620	2440	5450	8190
1220	1840	1060	1590	4210	6330		11	1810	2730	1570	2350	5450	8190
1170	1750	1010	1510	4120	6190		12	1760	2640	1510	2270	5390	8110
1110	1670	953	1430	4010	6030		13	1700	2550	1460	2190	5320	8000
1050	1580	899	1350	3900	5860		14	1640	2460	1400	2110	5250	7890
990	1490	844	1270	3780	5690		15	1580	2370	1350	2030	5170	7770
930	1400	790	1190	3660	5510		16	1520	2280	1290	1950	5100	7660
870	1310	736	1110	3540	5320		17	1460	2190	1240	1860	5020	7550
810	1220	682	1030	3410	5130		18	1400	2100	1190	1780	4950	7440
752	1130	630	947	3290	4940		19	1340	2010	1130	1700	4880	7330
693	1040	578	869	3160	4740		20	1280	1920	1080	1620	4800	7220
583	876	487	733	2890	4350		22	1120	1680	909	1370	4650	7000
490	736	410	616	2630	3950		24	960	1440	780	1170	4510	6770
417	627	349	525	2370	3560		26	838	1260	679	1020	4360	6550
360	541	301	452	2120	3190		28	741	1110	598	899	4210	6330
313	471	262	394	1880	2820		30	662	994	533	801	4060	6110
275	414	230	346	1650	2480		32	597	897	479	720	3920	5890
244	367	204	307	1460	2200		34	542	815	435	653	3770	5660
218	327	182	274	1300	1960		36	497	746	397	597	3620	5440
195	294	163	246	1170	1760		38	457	688	365	548	3470	5220
176	265			1060	1590		40	424	637	337	507	3320	5000
				959	1440		42	395	593	313	471	3140	4710
				874	1310		44	369	555	292	439	2960	4450
				799	1200		46	346	521	274	412	2810	4220
				734	1100		48	326	491	258	387	2670	4010
				676	1020		50	309	464	243	365	2540	3820

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1860	2790	1670	2510	4780	7180
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1500	2240	1350	2020	3850	5770
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
563	845	514	772	1270	1910
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
248	372	209	313	1090	1640

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.37	20.8	7.10	20.1	11.3	40.3
Area, in.^{2}					
44.3		39.9		114	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
9040	270	7800	225	24300	1620
r_y, in.					
2.47		2.38		3.77	
r_x/r_y					
5.79		5.88		3.87	

[c] Shape is slender for compression with F_y = 70 ksi.

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

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W33

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W33×						Shape		W33×					
354[h]		318		291[c]		lb/ft		354[h]		318		291	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
4360	6550	3930	5900	3570	5370	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	4960	7460	4440	6670	4050	6090
4200	6310	3780	5680	3450	5180		6	4960	7460	4440	6670	4050	6090
4140	6220	3730	5600	3400	5110		7	4960	7460	4440	6670	4050	6090
4070	6120	3670	5510	3350	5030		8	4960	7460	4440	6670	4050	6090
4000	6020	3600	5410	3290	4940		9	4960	7460	4440	6670	4050	6090
3920	5900	3530	5300	3220	4840		10	4960	7460	4440	6670	4050	6090
3840	5770	3450	5190	3150	4730		11	4960	7460	4440	6670	4050	6090
3750	5630	3370	5060	3070	4610		12	4900	7370	4370	6570	3980	5990
3650	5480	3280	4930	2990	4490		13	4830	7260	4300	6470	3920	5890
3550	5330	3180	4790	2900	4360		14	4760	7150	4230	6360	3850	5790
3440	5170	3090	4640	2810	4220		15	4690	7040	4160	6260	3780	5690
3330	5000	2990	4490	2720	4080		16	4610	6930	4100	6160	3720	5580
3210	4830	2880	4330	2620	3940		17	4540	6830	4030	6050	3650	5480
3100	4660	2780	4170	2520	3790		18	4470	6720	3960	5950	3580	5380
2980	4480	2670	4010	2420	3640		19	4400	6610	3890	5840	3510	5280
2860	4300	2560	3850	2320	3490		20	4330	6500	3820	5740	3450	5180
2620	3930	2340	3520	2120	3180		22	4180	6290	3680	5530	3310	4980
2380	3570	2120	3190	1920	2880		24	4040	6070	3540	5320	3180	4780
2140	3210	1900	2860	1720	2580		26	3900	5860	3400	5120	3050	4580
1910	2870	1700	2550	1530	2300		28	3750	5640	3270	4910	2910	4380
1690	2540	1500	2250	1340	2020		30	3610	5430	3130	4700	2780	4170
1480	2230	1310	1980	1180	1780		32	3470	5210	2990	4490	2640	3970
1310	1970	1160	1750	1050	1570		34	3330	5000	2850	4290	2510	3770
1170	1760	1040	1560	934	1400		36	3180	4780	2710	4070	2320	3490
1050	1580	932	1400	838	1260		38	3040	4570	2520	3790	2150	3240
949	1430	841	1260	756	1140		40	2840	4280	2350	3540	2010	3020
861	1290	763	1150	686	1030		42	2670	4020	2210	3320	1880	2830
784	1180	695	1050	625	939		44	2520	3790	2080	3120	1770	2660
718	1080	636	956	572	859		46	2390	3590	1960	2950	1670	2510
659	991	584	878	525	789		48	2270	3400	1860	2800	1580	2370
607	913	538	809	484	727		50	2160	3240	1770	2660	1500	2260

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4360	6550	3930	5900	3590	5390
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3510	5270	3160	4740	2890	4330
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1160	1730	1030	1540	935	1400
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
985	1480	873	1310	789	1190

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.2	38.1	11.1	36.0	11.0	34.2
Area, in.^{2}					
104		93.7		85.6	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
22000	1460	19500	1290	17700	1160
r_y, in.					
3.74		3.71		3.68	
r_x/r_y					
3.88		3.91		3.91	

[c] Shape is slender for compression with F_y = 70 ksi.

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

W33

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W33×						Shape	W33×					
263^c		241^c		221^c		lb/ft	263		241		221	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
3150	4740	2860	4300	2580	3880	0	3630	5460	3280	4940	2990	4500
3040	4580	2760	4150	2490	3740	6	3630	5460	3280	4940	2990	4500
3010	4520	2730	4100	2460	3690	7	3630	5460	3280	4940	2990	4500
2960	4450	2690	4040	2420	3640	8	3630	5460	3280	4940	2990	4500
2910	4380	2640	3970	2380	3580	9	3630	5460	3280	4940	2990	4500
2860	4300	2590	3900	2330	3510	10	3630	5460	3280	4940	2990	4500
2800	4210	2540	3820	2290	3440	11	3630	5450	3270	4920	2980	4480
2740	4120	2480	3730	2230	3360	12	3560	5360	3210	4830	2920	4390
2670	4020	2420	3640	2180	3270	13	3500	5260	3150	4740	2860	4300
2610	3920	2360	3540	2120	3190	14	3440	5170	3090	4650	2810	4220
2530	3810	2290	3440	2060	3100	15	3380	5070	3030	4560	2750	4130
2450	3680	2220	3340	2000	3000	16	3310	4980	2970	4470	2690	4050
2360	3550	2150	3230	1930	2900	17	3250	4880	2910	4380	2640	3960
2270	3410	2070	3110	1860	2800	18	3190	4790	2850	4290	2580	3880
2180	3280	1990	2980	1800	2700	19	3120	4690	2790	4200	2520	3790
2090	3140	1900	2860	1730	2600	20	3060	4600	2730	4110	2470	3710
1900	2860	1730	2600	1570	2370	22	2930	4410	2610	3930	2350	3540
1720	2590	1560	2340	1420	2130	24	2810	4220	2490	3750	2240	3370
1540	2320	1390	2090	1260	1900	26	2680	4030	2370	3570	2130	3190
1370	2060	1230	1850	1120	1680	28	2550	3840	2260	3390	2010	3020
1200	1810	1080	1620	976	1470	30	2430	3650	2140	3210	1900	2850
1060	1590	950	1430	858	1290	32	2300	3460	2000	3010	1740	2610
936	1410	841	1260	760	1140	34	2140	3210	1830	2740	1580	2380
835	1260	750	1130	678	1020	36	1970	2950	1680	2520	1450	2180
749	1130	674	1010	608	914	38	1820	2730	1550	2330	1330	2010
676	1020	608	914	549	825	40	1690	2540	1440	2160	1240	1860
614	922	551	829	498	748	42	1580	2380	1340	2010	1150	1730
559	840	502	755	454	682	44	1490	2230	1260	1890	1080	1620
511	769	460	691	415	624	46	1400	2100	1180	1780	1010	1520
470	706	422	634	381	573	48	1320	1990	1110	1680	953	1430
433	651	389	585	351	528	50	1260	1890	1060	1590	901	1350

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3240	4880	2980	4480	2740	4110
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2610	3920	2400	3600	2200	3310
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
840	1260	795	1190	736	1100
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
706	1060	636	956	573	861

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.9	32.9	10.8	31.7	10.7	30.7
Area, in.^{2}					
77.4		71.1		65.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
15900	1040	14200	933	12900	840
r_y, in.					
3.66		3.62		3.59	
r_x/r_y					
3.91		3.90		3.93	

[c] Shape is slender for compression with F_y = 70 ksi.

Note: Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W33

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W33×						Shape	W33×					
201^c		169^c		152^c		lb/ft	201		169		152^v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2290	3440	1860	2800	1650	2480	0	2700	4060	2200	3300	1950	2930
2210	3310	1730	2600	1530	2310	6	2700	4060	2200	3300	1950	2930
2180	3270	1680	2530	1490	2250	7	2700	4060	2200	3300	1950	2930
2140	3220	1630	2460	1450	2180	8	2700	4060	2160	3250	1920	2880
2110	3170	1580	2370	1400	2100	9	2700	4060	2100	3160	1860	2800
2070	3110	1520	2280	1340	2020	10	2700	4060	2040	3070	1810	2710
2020	3040	1460	2190	1290	1930	11	2680	4030	1980	2980	1750	2630
1980	2970	1390	2090	1230	1850	12	2630	3950	1920	2890	1690	2550
1930	2890	1320	1980	1170	1750	13	2570	3870	1860	2800	1640	2460
1870	2820	1250	1880	1100	1660	14	2520	3790	1800	2710	1580	2380
1820	2730	1180	1770	1040	1560	15	2470	3710	1740	2620	1530	2290
1760	2650	1110	1660	975	1470	16	2410	3630	1680	2530	1470	2210
1700	2560	1040	1560	911	1370	17	2360	3550	1620	2440	1410	2120
1640	2470	965	1450	847	1270	18	2310	3470	1560	2340	1360	2040
1580	2380	886	1330	785	1180	19	2250	3390	1500	2250	1300	1960
1520	2290	807	1210	715	1070	20	2200	3310	1440	2160	1250	1870
1400	2100	667	1000	591	888	22	2090	3150	1300	1950	1100	1650
1270	1910	561	843	496	746	24	1990	2990	1130	1700	949	1430
1130	1700	478	718	423	636	26	1880	2830	997	1500	834	1250
995	1500	412	619	365	548	28	1770	2670	890	1340	741	1110
869	1310	359	539	318	478	30	1660	2490	803	1210	666	1000
763	1150	315	474	279	420	32	1490	2240	730	1100	604	908
676	1020	279	420	247	372	34	1350	2030	670	1010	552	830
603	907	249	375	221	332	36	1240	1860	618	929	508	763
541	814	224	336	198	298	38	1140	1710	574	862	470	707
489	734	202	303	179	269	40	1050	1580	535	805	438	658
443	666					42	976	1470	502	754	409	615
404	607					44	911	1370	472	710	384	577
369	555					46	854	1280	446	670	362	544
339	510					48	804	1210	422	635	342	514
313	470					50	759	1140	401	603	324	488

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2480	3720	2070	3120	1880	2830
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1990	2990	1670	2510	1520	2270
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
675	1010	634	951	535	804
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
513	772	295	443	258	388

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.6	29.8	7.46	21.6	7.37	21.0
Area, in.^{2}					
59.1		49.5		44.9	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
11600	749	9290	310	8160	273
r_y, in.					
3.56		2.50		2.47	
r_x/r_y					
3.93		5.48		5.47	

[c] Shape is slender for compression with F_y = 70 ksi.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W33

F_y = 70 ksi
F_u = 90 ksi

W33×						Shape	W33×					
141^c		130^c		118^c		lb/ft	141^v		130^v		118^f, v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1500	2250	1360	2040	1200	1800	0	1800	2700	1630	2450	1450	2180
1390	2090	1260	1890	1100	1660	6	1800	2700	1630	2450	1450	2180
1350	2030	1220	1830	1070	1610	7	1800	2700	1630	2450	1450	2170
1310	1970	1180	1770	1040	1560	8	1760	2640	1590	2390	1400	2100
1260	1900	1140	1710	996	1500	9	1700	2560	1540	2310	1350	2040
1210	1820	1090	1640	953	1430	10	1650	2480	1490	2240	1310	1970
1160	1740	1040	1570	908	1370	11	1600	2400	1440	2160	1260	1900
1100	1660	992	1490	862	1300	12	1540	2320	1390	2090	1220	1830
1050	1570	939	1410	814	1220	13	1490	2240	1340	2010	1170	1760
989	1490	886	1330	765	1150	14	1440	2160	1290	1940	1130	1690
930	1400	832	1250	716	1080	15	1390	2080	1240	1860	1080	1620
871	1310	777	1170	667	1000	16	1330	2000	1190	1790	1030	1550
812	1220	724	1090	619	930	17	1280	1920	1140	1710	988	1490
754	1130	671	1010	571	859	18	1230	1840	1090	1640	942	1420
698	1050	619	930	524	788	19	1170	1770	1040	1570	896	1350
639	961	568	854	481	723	20	1120	1690	989	1490	830	1250
528	794	472	709	403	605	22	968	1460	837	1260	700	1050
444	667	396	596	338	509	24	836	1260	721	1080	601	903
378	569	338	508	288	433	26	732	1100	630	946	524	787
326	490	291	438	249	374	28	649	976	557	837	462	694
284	427	254	381	217	326	30	582	875	498	749	412	619
250	375	223	335	190	286	32	527	792	450	676	371	558
221	333	198	297	169	253	34	480	722	409	615	337	506
197	297	176	265	150	226	36	441	663	375	564	308	463
177	266	158	238	135	203	38	408	613	346	520	283	426
160	240					40	379	569	321	482	262	394
						42	353	531	299	449	244	366
						44	331	498	280	420	228	342
						46	312	468	263	395	213	321
						48	294	442	248	372	201	302
						50	279	419	234	352	190	285

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1740	2610	1610	2410	1450	2190
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1400	2100	1290	1940	1170	1760
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
507	762	483	726	432	649
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
234	351	208	312	179	269

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.25	20.5	7.13	20.0	6.95	19.4
Area, in.^{2}					
41.5		38.3		34.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
7450	246	6710	218	5900	187
r_y, in.					
2.43		2.39		2.32	
r_x/r_y					
5.51		5.52		5.60	

[c] Shape is slender for compression with F_y = 70 ksi.

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W30

F_y = 70 ksi
F_u = 90 ksi

W30×						Shape	W30×					
391^h		357^h		326^h		lb/ft	391^h		357^h		326^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
4820	7240	4400	6610	4020	6040	0	5060	7610	4610	6930	4160	6250
4630	6970	4230	6360	3860	5800	6	5060	7610	4610	6930	4160	6250
4570	6870	4170	6260	3800	5710	7	5060	7610	4610	6930	4160	6250
4490	6750	4100	6160	3740	5620	8	5060	7610	4610	6930	4160	6250
4410	6630	4020	6040	3670	5510	9	5060	7610	4610	6930	4160	6250
4320	6490	3940	5920	3590	5390	10	5060	7610	4610	6930	4160	6250
4220	6350	3850	5780	3500	5260	11	5060	7610	4600	6920	4140	6230
4120	6190	3750	5640	3410	5130	12	5000	7520	4540	6830	4080	6140
4010	6020	3650	5480	3320	4990	13	4940	7420	4480	6730	4020	6050
3890	5850	3540	5320	3220	4830	14	4880	7330	4420	6640	3970	5960
3770	5660	3430	5150	3110	4680	15	4820	7240	4360	6550	3910	5870
3640	5470	3310	4980	3000	4520	16	4750	7150	4300	6460	3850	5780
3510	5280	3190	4800	2890	4350	17	4690	7050	4240	6370	3790	5700
3380	5080	3070	4610	2780	4180	18	4630	6960	4180	6280	3730	5610
3250	4880	2950	4430	2670	4010	19	4570	6870	4110	6180	3670	5520
3110	4680	2820	4240	2550	3830	20	4510	6780	4050	6090	3610	5430
2840	4270	2570	3860	2320	3480	22	4390	6590	3930	5910	3500	5260
2570	3860	2320	3490	2090	3140	24	4260	6410	3810	5730	3380	5080
2300	3460	2070	3120	1860	2800	26	4140	6220	3690	5540	3260	4900
2040	3070	1840	2770	1650	2480	28	4020	6040	3570	5360	3150	4730
1800	2700	1610	2430	1440	2170	30	3890	5850	3440	5180	3030	4550
1580	2370	1420	2130	1270	1900	32	3770	5670	3320	4990	2910	4370
1400	2100	1260	1890	1120	1690	34	3650	5480	3200	4810	2790	4200
1250	1880	1120	1680	1000	1500	36	3530	5300	3080	4630	2680	4020
1120	1680	1010	1510	898	1350	38	3400	5110	2960	4440	2560	3850
1010	1520	908	1360	811	1220	40	3280	4930	2830	4260	2410	3610
917	1380	823	1240	735	1110	42	3160	4740	2690	4040	2270	3400
835	1260	750	1130	670	1010	44	3020	4550	2540	3820	2140	3220
764	1150	686	1030	613	921	46	2870	4320	2410	3630	2030	3050
702	1050	630	947	563	846	48	2740	4110	2300	3450	1930	2900
647	972	581	873	519	780	50	2610	3930	2190	3290	1840	2770

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4820	7250	4400	6620	4020	6040
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3880	5820	3540	5320	3240	4850
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1260	1900	1140	1710	1030	1550
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
1080	1630	975	1460	880	1320

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.0	43.6	10.9	40.8	10.7	38.3
Area, in.^{2}					
115		105		95.9	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
20700	1550	18700	1390	16800	1240
r_y, in.					
3.67		3.64		3.60	
r_x/r_y					
3.65		3.65		3.67	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W30

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W30×						Shape		W30×					
292		261		235^c		lb/ft		292		261		235	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending		ASD	LRFD	ASD	LRFD	ASD	LRFD
3600	5420	3230	4850	2870	4310		0	3700	5570	3290	4950	2960	4450
3460	5200	3090	4650	2760	4150		6	3700	5570	3290	4950	2960	4450
3410	5120	3050	4580	2720	4090		7	3700	5570	3290	4950	2960	4450
3350	5030	2990	4500	2680	4020		8	3700	5570	3290	4950	2960	4450
3280	4940	2930	4410	2630	3950		9	3700	5570	3290	4950	2960	4450
3210	4830	2870	4310	2580	3870		10	3700	5570	3290	4950	2960	4450
3140	4710	2800	4200	2510	3780		11	3680	5540	3270	4910	2930	4410
3050	4590	2720	4090	2450	3670		12	3630	5450	3210	4830	2880	4330
2970	4460	2640	3970	2370	3570		13	3570	5370	3160	4750	2830	4250
2880	4320	2560	3850	2300	3450		14	3510	5280	3100	4670	2780	4170
2780	4180	2470	3720	2220	3340		15	3460	5200	3050	4580	2720	4090
2690	4040	2380	3580	2140	3210		16	3400	5110	2990	4500	2670	4010
2590	3890	2290	3450	2060	3090		17	3340	5030	2940	4420	2620	3940
2480	3730	2200	3310	1970	2960		18	3290	4940	2890	4340	2570	3860
2380	3580	2110	3160	1890	2830		19	3230	4860	2830	4250	2510	3780
2280	3420	2010	3020	1800	2710		20	3170	4770	2780	4170	2460	3700
2070	3110	1820	2740	1630	2450		22	3060	4600	2670	4010	2360	3540
1860	2790	1630	2450	1460	2190		24	2950	4430	2560	3840	2250	3390
1660	2490	1450	2180	1290	1940		26	2830	4260	2450	3680	2150	3230
1460	2200	1280	1920	1140	1710		28	2720	4090	2340	3510	2050	3080
1280	1920	1110	1670	990	1490		30	2610	3920	2230	3350	1940	2920
1120	1690	978	1470	870	1310		32	2490	3750	2120	3180	1840	2760
995	1500	866	1300	771	1160		34	2380	3580	2000	3000	1690	2540
888	1330	773	1160	688	1030		36	2260	3400	1850	2780	1560	2340
797	1200	694	1040	617	928		38	2110	3170	1720	2580	1450	2170
719	1080	626	941	557	837		40	1980	2970	1610	2410	1350	2030
652	980	568	853	505	759		42	1860	2790	1510	2270	1260	1900
594	893	517	778	460	692		44	1750	2640	1420	2130	1190	1790
544	817	473	711	421	633		46	1660	2500	1340	2020	1120	1690
499	751	435	653	387	581		48	1580	2370	1270	1910	1060	1600
460	692	401	602	356	536		50	1500	2260	1210	1820	1010	1520

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3600	5420	3230	4850	2900	4370
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2900	4350	2600	3900	2340	3510
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
914	1370	823	1230	727	1090
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
779	1170	685	1030	611	919

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.7	35.9	10.5	33.7	10.5	32.2
Area, in.^{2}					
86.0		77.0		69.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
14900	1100	13100	959	11700	855
r_y, in.					
3.58		3.53		3.51	
r_x/r_y					
3.69		3.71		3.70	

[c] Shape is slender for compression with F_y = 70 ksi.

Note: Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W30

F_y = 70 ksi
F_u = 90 ksi

W30×						Shape	W30×					
211^c		191^c		173^c		lb/ft	211		191		173	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2530	3810	2230	3360	1990	2990	0	2620	3940	2360	3540	2120	3190
2440	3670	2150	3230	1910	2870	6	2620	3940	2360	3540	2120	3190
2410	3620	2120	3180	1880	2830	7	2620	3940	2360	3540	2120	3190
2370	3560	2080	3130	1850	2780	8	2620	3940	2360	3540	2120	3190
2320	3490	2050	3080	1820	2730	9	2620	3940	2360	3540	2120	3190
2280	3420	2000	3010	1780	2670	10	2620	3940	2360	3540	2120	3190
2230	3350	1960	2940	1740	2610	11	2590	3900	2330	3500	2090	3140
2170	3270	1910	2870	1690	2550	12	2550	3830	2280	3430	2040	3070
2120	3180	1860	2790	1650	2480	13	2500	3750	2230	3360	2000	3010
2060	3090	1810	2710	1600	2400	14	2450	3680	2190	3290	1960	2940
1990	2990	1750	2630	1550	2330	15	2400	3610	2140	3220	1910	2880
1920	2880	1690	2540	1500	2250	16	2350	3530	2100	3150	1870	2810
1840	2770	1630	2450	1440	2170	17	2300	3460	2050	3080	1830	2750
1760	2650	1570	2360	1390	2090	18	2250	3380	2000	3010	1780	2680
1690	2540	1510	2270	1330	2000	19	2200	3310	1960	2940	1740	2620
1610	2420	1440	2160	1280	1920	20	2150	3240	1910	2870	1700	2550
1450	2180	1300	1950	1160	1740	22	2060	3090	1820	2730	1610	2420
1300	1950	1160	1740	1030	1550	24	1960	2940	1730	2590	1530	2290
1150	1730	1020	1540	910	1370	26	1860	2800	1630	2450	1440	2160
1010	1520	894	1340	793	1190	28	1760	2650	1540	2310	1350	2030
880	1320	779	1170	690	1040	30	1660	2500	1430	2150	1230	1850
773	1160	685	1030	607	912	32	1530	2300	1300	1950	1110	1670
685	1030	606	911	538	808	34	1400	2100	1180	1770	1010	1510
611	919	541	813	479	721	36	1290	1940	1080	1630	922	1390
549	824	485	730	430	647	38	1190	1790	1000	1500	850	1280
495	744	438	659	388	584	40	1110	1670	928	1400	787	1180
449	675	397	597	352	529	42	1040	1560	866	1300	733	1100
409	615	362	544	321	482	44	973	1460	812	1220	685	1030
374	563	331	498	294	441	46	917	1380	763	1150	643	967
344	517	304	457	270	405	48	866	1300	720	1080	606	911
317	476	280	421	249	374	50	822	1230	682	1030	573	861

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2610	3920	2350	3530	2130	3210
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2100	3150	1890	2840	1720	2580
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
671	1010	610	915	558	836
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
541	814	482	725	430	646

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.4	30.8	10.3	29.6	10.2	28.7
Area, in.^{2}					
62.3		56.1		50.9	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
10300	757	9200	673	8230	598
r_y, in.					
3.49		3.46		3.42	
r_x/r_y					
3.70		3.70		3.71	

^{c} Shape is slender for compression with F_y = 70 ksi.
Note: Confirm ASTM A913 material availability before specifying.

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W30

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W30× 148c		W30× 132c		W30× 124c		Shape lb/ft	W30× 148		W30× 132		W30× 124v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
1670	2510	1460	2190	1350	2030	0	1750	2630	1530	2290	1430	2140
1530	2300	1330	2000	1230	1850	6	1750	2630	1530	2290	1430	2140
1480	2230	1290	1940	1190	1790	7	1740	2610	1510	2270	1410	2120
1430	2150	1240	1870	1150	1720	8	1690	2530	1470	2200	1360	2050
1370	2060	1190	1790	1100	1650	9	1630	2460	1420	2130	1320	1980
1310	1970	1140	1710	1050	1570	10	1580	2380	1370	2060	1270	1910
1250	1870	1080	1620	993	1490	11	1530	2300	1320	1990	1230	1850
1180	1770	1020	1530	938	1410	12	1480	2220	1280	1920	1180	1780
1110	1660	958	1440	880	1320	13	1430	2150	1230	1850	1140	1710
1040	1560	896	1350	823	1240	14	1380	2070	1180	1780	1090	1640
966	1450	834	1250	765	1150	15	1330	1990	1130	1710	1050	1580
884	1330	772	1160	707	1060	16	1270	1910	1090	1630	1000	1510
805	1210	701	1050	650	976	17	1220	1840	1040	1560	958	1440
729	1100	633	951	585	879	18	1170	1760	993	1490	912	1370
655	985	568	854	525	789	19	1120	1680	946	1420	866	1300
591	889	513	770	474	712	20	1070	1610	878	1320	794	1190
489	735	424	637	391	588	22	919	1380	751	1130	677	1020
411	617	356	535	329	494	24	805	1210	654	983	588	884
350	526	303	456	280	421	26	714	1070	578	868	518	779
302	454	262	393	242	363	28	641	963	516	776	462	695
263	395	228	342	211	316	30	581	874	466	701	417	626
231	347	200	301	185	278	32	531	799	425	638	379	569
205	308	177	267	164	246	34	489	736	390	586	347	522
183	274	158	238	146	220	36	454	682	360	541	320	481
164	246					38	423	635	335	503	297	446
						40	396	595	312	469	277	416
						42	372	559	293	440	259	390
						44	351	528	276	415	244	367
						46	332	500	261	392	230	346
						48	316	474	247	371	218	328
						50	300	452	235	353	207	311

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1830	2750	1630	2440	1530	2300
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1470	2210	1310	1960	1230	1850
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
559	838	522	783	444	668
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
238	357	204	307	189	284

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
6.81	20.0	6.72	19.3	6.66	19.0
Area, in.^{2}					
43.6		38.8		36.5	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
6680	227	5770	196	5360	181
r_y, in.					
2.28		2.25		2.23	
r_x/r_y					
5.44		5.42		5.43	

[c] Shape is slender for compression with F_y = 70 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W30

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W30×						Shape	W30×					
116^c		108^c		99^c		lb/ft	116^v		108^v		99^{f,v}	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
1250	1870	1130	1700	1020	1530	0	1320	1980	1210	1820	1090	1630
1130	1700	1030	1550	918	1380	6	1320	1980	1210	1820	1090	1630
1100	1650	994	1490	885	1330	7	1300	1950	1180	1780	1060	1600
1050	1580	955	1440	849	1280	8	1260	1890	1140	1720	1020	1540
1010	1520	912	1370	809	1220	9	1210	1830	1100	1660	986	1480
960	1440	867	1300	768	1150	10	1170	1760	1060	1600	948	1420
909	1370	820	1230	724	1090	11	1130	1700	1020	1540	910	1370
856	1290	771	1160	679	1020	12	1090	1630	982	1480	872	1310
802	1210	721	1080	634	952	13	1040	1570	941	1410	834	1250
748	1120	671	1010	588	884	14	1000	1500	900	1350	796	1200
694	1040	621	933	543	815	15	957	1440	860	1290	758	1140
640	962	572	859	498	748	16	915	1370	819	1230	720	1080
588	883	523	787	454	682	17	872	1310	779	1170	682	1030
528	794	472	710	412	619	18	829	1250	738	1110	634	952
474	713	424	637	370	556	19	772	1160	675	1010	575	865
428	643	382	575	334	502	20	707	1060	617	927	525	790
354	532	316	475	276	415	22	602	904	524	788	445	669
297	447	266	399	232	348	24	521	784	453	681	384	577
253	381	226	340	197	297	26	458	689	397	597	336	504
218	328	195	293	170	256	28	408	613	353	530	297	447
190	286	170	255	148	223	30	367	552	317	476	266	400
167	251	149	224	130	196	32	333	501	287	431	241	362
148	223	132	199	115	174	34	305	458	262	393	219	329
132	199					36	281	422	241	362	201	302
						38	260	391	222	334	186	279
						40	242	364	207	311	172	259
						42	226	340	193	290	161	241
						44	213	320	181	272	150	226
						46	201	301	171	257	142	213
						48	190	285	161	242	134	201
						50	180	271	153	230	126	190

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1430	2150	1330	2000	1220	1830
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1150	1730	1070	1600	979	1470
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
426	641	408	614	387	582
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
172	258	153	230	134	202

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
6.54	18.6	6.42	18.2	6.33	17.6
Area, in.^{2}					
34.2		31.7		29.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
4930	164	4470	146	3990	128
r_y, in.					
2.19		2.15		2.10	
r_x/r_y					
5.48		5.53		5.57	

[c] Shape is slender for compression with F_y = 70 ksi.

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W30–W27

F_y = 70 ksi
F_u = 90 ksi

W30×		W27×				Shape	W30×		W27×			
90^c		539^h		368^h		lb/ft	90$^{f\,v}$		539^h		368^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
892	1340	6660	10000	4570	6870	0	964	1450	6600	9920	4330	6510
805	1210	6400	9630	4370	6570	6	964	1450	6600	9920	4330	6510
776	1170	6310	9490	4300	6470	7	962	1450	6600	9920	4330	6510
743	1120	6210	9330	4230	6350	8	927	1390	6600	9920	4330	6510
708	1060	6090	9160	4140	6220	9	892	1340	6600	9920	4330	6510
671	1010	5970	8970	4050	6080	10	857	1290	6600	9920	4330	6510
633	951	5830	8760	3940	5930	11	822	1230	6600	9910	4300	6460
593	891	5680	8540	3830	5760	12	787	1180	6540	9840	4250	6390
553	831	5530	8310	3720	5590	13	751	1130	6490	9760	4200	6320
512	770	5370	8060	3600	5410	14	716	1080	6440	9680	4150	6240
472	710	5200	7810	3470	5220	15	681	1020	6390	9600	4100	6170
433	651	5020	7550	3350	5030	16	646	971	6340	9520	4050	6090
394	593	4840	7280	3210	4830	17	611	919	6280	9440	4000	6020
359	539	4660	7000	3080	4630	18	559	840	6230	9370	3950	5940
328	493	4470	6720	2940	4430	19	507	762	6180	9290	3900	5870
300	451	4280	6430	2810	4220	20	462	695	6130	9210	3850	5790
248	372	3900	5860	2530	3810	22	390	587	6020	9050	3760	5640
208	313	3520	5300	2270	3410	24	335	504	5920	8890	3660	5500
177	267	3150	4740	2010	3020	26	293	440	5810	8740	3560	5350
153	230	2800	4210	1760	2640	28	259	389	5710	8580	3460	5200
133	200	2460	3690	1530	2300	30	231	347	5600	8420	3360	5050
117	176	2160	3250	1350	2020	32	208	313	5500	8270	3260	4900
104	156	1910	2870	1190	1790	34	189	284	5400	8110	3160	4750
		1710	2560	1060	1600	36	173	260	5290	7950	3060	4600
		1530	2300	954	1430	38	159	240	5190	7800	2960	4450
		1380	2080	861	1290	40	148	222	5080	7640	2860	4300
		1250	1880	781	1170	42	137	206	4980	7480	2760	4150
		1140	1720	712	1070	44	128	193	4870	7320	2670	4010
		1040	1570	651	979	46	121	181	4770	7170	2560	3840
		960	1440	598	899	48	114	171	4660	7010	2440	3670
		884	1330	551	828	50	107	161	4560	6850	2330	3510

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1100	1660	6660	10000	4570	6870
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
888	1330	5370	8050	3680	5520
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
314	472	1790	2690	1170	1760
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
117	176	1530	2290	975	1460

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
6.93	17.3	10.9	63.8	10.4	45.5
Area, in.^{2}					
26.3		159		109	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
3610	115	25600	2110	16200	1310
r_y, in.					
2.09		3.65		3.48	
r_x/r_y					
5.60		3.48		3.51	

^{c} Shape is slender for compression with F_y = 70 ksi.
^{f} Shape exceeds compact limit for flexure with F_y = 70 ksi.
^{h} Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
^{v} Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W27

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W27×						Shape		W27×					
336^h		307^h		281		lb/ft		336^h		307^h		281	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
4160	6250	3780	5680	3480	5240	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	3950	5930	3600	5410	3270	4910
3980	5980	3610	5430	3330	5000		6	3950	5930	3600	5410	3270	4910
3910	5880	3550	5340	3270	4920		7	3950	5930	3600	5410	3270	4910
3840	5770	3490	5240	3210	4820		8	3950	5930	3600	5410	3270	4910
3760	5650	3410	5130	3140	4720		9	3950	5930	3600	5410	3270	4910
3670	5520	3330	5010	3060	4610		10	3950	5930	3600	5410	3270	4910
3580	5380	3240	4870	2980	4480		11	3910	5880	3560	5350	3230	4850
3480	5230	3150	4730	2900	4350		12	3860	5810	3510	5270	3180	4780
3370	5070	3050	4590	2800	4220		13	3810	5730	3460	5200	3130	4710
3260	4900	2950	4430	2710	4070		14	3760	5660	3410	5120	3080	4640
3150	4730	2840	4270	2610	3920		15	3720	5580	3360	5050	3040	4560
3030	4550	2730	4110	2510	3770		16	3670	5510	3310	4980	2990	4490
2910	4370	2620	3940	2400	3610		17	3620	5440	3260	4900	2940	4420
2780	4180	2510	3770	2300	3460		18	3570	5360	3210	4830	2890	4350
2660	4000	2390	3600	2190	3290		19	3520	5290	3160	4750	2850	4280
2530	3810	2280	3420	2090	3130		20	3470	5210	3110	4680	2800	4210
2280	3430	2050	3080	1870	2810		22	3370	5070	3020	4530	2700	4060
2040	3060	1820	2740	1660	2500		24	3270	4920	2920	4380	2610	3920
1800	2710	1600	2410	1460	2200		26	3170	4770	2820	4240	2510	3780
1570	2360	1400	2100	1270	1910		28	3070	4620	2720	4090	2420	3630
1370	2060	1220	1830	1110	1660		30	2980	4470	2620	3940	2320	3490
1200	1810	1070	1610	973	1460		32	2880	4320	2520	3790	2230	3350
1070	1600	947	1420	862	1300		34	2780	4180	2420	3640	2130	3200
951	1430	845	1270	769	1160		36	2680	4030	2330	3500	2040	3060
853	1280	758	1140	690	1040		38	2580	3880	2230	3350	1920	2890
770	1160	684	1030	623	936		40	2480	3730	2120	3180	1810	2710
699	1050	621	933	565	849		42	2380	3580	2000	3000	1700	2560
637	957	565	850	515	774		44	2260	3400	1890	2850	1610	2420
582	875	517	778	471	708		46	2150	3230	1800	2700	1530	2300
535	804	475	714	433	650		48	2050	3080	1710	2580	1460	2190
493	741	438	658	399	599		50	1960	2940	1640	2460	1390	2090

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4160	6250	3780	5680	3480	5240
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3350	5020	3040	4570	2800	4210
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1060	1590	961	1440	870	1300
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
880	1320	793	1190	720	1080

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.3	42.1	10.2	39.2	10.1	36.9
Area, in.^{2}					
99.2		90.2		83.1	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
14600	1180	13100	1050	11900	953
r_y, in.					
3.45		3.41		3.39	
r_x/r_y					
3.51		3.52		3.54	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W27

F_y = 70 ksi
F_u = 90 ksi

W27×						Shape		W27×					
258		235		217		lb/ft		258		235		217	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
3190	4790	2910	4370	2680	4030	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	2980	4470	2700	4050	2480	3730
3040	4570	2770	4170	2550	3840		6	2980	4470	2700	4050	2480	3730
2990	4500	2730	4100	2510	3770		7	2980	4470	2700	4050	2480	3730
2930	4410	2670	4020	2460	3700		8	2980	4470	2700	4050	2480	3730
2870	4310	2610	3930	2400	3610		9	2980	4470	2700	4050	2480	3730
2800	4210	2550	3830	2340	3520		10	2980	4470	2690	4050	2480	3730
2720	4090	2480	3720	2280	3420		11	2930	4410	2650	3980	2440	3660
2640	3970	2400	3610	2210	3320		12	2880	4340	2600	3910	2390	3600
2560	3840	2320	3490	2140	3210		13	2840	4270	2560	3850	2350	3530
2470	3710	2240	3370	2060	3100		14	2790	4200	2510	3780	2300	3460
2380	3570	2160	3240	1980	2980		15	2750	4130	2470	3710	2260	3400
2280	3430	2070	3110	1900	2860		16	2700	4060	2420	3640	2220	3330
2190	3290	1980	2980	1820	2740		17	2650	3990	2380	3570	2170	3270
2090	3140	1890	2840	1740	2610		18	2610	3920	2330	3510	2130	3200
1990	2990	1800	2710	1650	2480		19	2560	3850	2290	3440	2090	3140
1890	2840	1710	2570	1570	2360		20	2510	3780	2240	3370	2040	3070
1700	2550	1530	2300	1400	2110		22	2420	3640	2150	3230	1950	2940
1500	2260	1350	2030	1240	1860		24	2330	3500	2060	3100	1870	2810
1320	1980	1180	1780	1080	1630		26	2230	3360	1970	2960	1780	2680
1140	1720	1020	1540	938	1410		28	2140	3220	1880	2830	1690	2540
996	1500	893	1340	817	1230		30	2050	3080	1790	2690	1610	2410
876	1320	784	1180	718	1080		32	1950	2940	1700	2550	1510	2270
776	1170	695	1040	636	956		34	1860	2800	1590	2390	1390	2090
692	1040	620	932	567	853		36	1750	2630	1470	2220	1290	1940
621	933	556	836	509	765		38	1630	2460	1380	2070	1200	1800
560	842	502	755	459	691		40	1530	2300	1290	1940	1120	1690
508	764	455	684	417	626		42	1450	2170	1210	1820	1060	1590
463	696	415	624	380	571		44	1370	2050	1150	1720	997	1500
424	637	380	571	347	522		46	1300	1950	1090	1630	944	1420
389	585	349	524	319	480		48	1230	1850	1030	1550	896	1350
359	539	321	483	294	442		50	1180	1770	984	1480	853	1280

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3190	4790	2910	4370	2680	4030
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2570	3850	2340	3510	2160	3230
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
796	1190	731	1100	660	990
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
653	982	587	882	538	809

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.0	34.9	9.94	33.0	9.91	31.6
Area, in.^{2}					
76.1		69.4		63.9	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
10800	859	9700	769	8910	704
r_y, in.					
3.36		3.33		3.32	
r_x/r_y					
3.54		3.54		3.55	

Note: Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W27

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W27×						Shape		W27×					
194^c		178^c		161^c		lb/ft		194		178		161	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
2370	3560	2160	3250	1920	2880	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	2200	3310	1990	2990	1800	2700
2270	3410	2070	3110	1830	2750		6	2200	3310	1990	2990	1800	2700
2230	3350	2030	3060	1800	2710		7	2200	3310	1990	2990	1800	2700
2190	3290	2000	3000	1770	2660		8	2200	3310	1990	2990	1800	2700
2140	3220	1950	2940	1730	2600		9	2200	3310	1990	2990	1800	2700
2090	3140	1910	2870	1690	2540		10	2200	3300	1980	2970	1790	2680
2030	3050	1860	2790	1650	2480		11	2160	3240	1940	2920	1750	2630
1970	2960	1800	2710	1600	2410		12	2110	3180	1900	2860	1710	2570
1900	2860	1740	2610	1550	2330		13	2070	3110	1860	2800	1670	2510
1830	2750	1670	2520	1500	2250		14	2030	3050	1820	2740	1640	2460
1760	2650	1610	2420	1450	2170		15	1990	2990	1780	2680	1600	2400
1690	2540	1540	2310	1390	2090		16	1950	2930	1740	2620	1560	2350
1610	2430	1470	2210	1330	1990		17	1910	2860	1700	2560	1520	2290
1540	2310	1400	2100	1260	1900		18	1860	2800	1660	2500	1490	2230
1460	2200	1330	2000	1200	1800		19	1820	2740	1620	2440	1450	2180
1390	2090	1260	1890	1130	1700		20	1780	2680	1580	2380	1410	2120
1240	1860	1120	1680	1010	1510		22	1700	2550	1510	2260	1340	2010
1090	1640	985	1480	884	1330		24	1620	2430	1430	2140	1260	1900
953	1430	856	1290	767	1150		26	1530	2300	1350	2030	1190	1780
823	1240	738	1110	661	994		28	1450	2180	1270	1910	1100	1660
717	1080	643	967	576	866		30	1360	2050	1160	1750	993	1490
630	947	565	850	506	761		32	1240	1870	1060	1590	900	1350
558	839	501	753	448	674		34	1140	1720	969	1460	823	1240
498	748	447	671	400	601		36	1060	1590	894	1340	757	1140
447	671	401	602	359	540		38	982	1480	829	1250	701	1050
403	606	362	544	324	487		40	918	1380	773	1160	652	980
366	550	328	493	294	442		42	861	1290	724	1090	610	916
333	501	299	449	268	402		44	811	1220	681	1020	572	860
305	458	274	411	245	368		46	767	1150	643	966	539	811
280	421	251	378	225	338		48	727	1090	608	914	510	766
258	388	232	348	207	312		50	692	1040	578	868	484	727

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2390	3600	2200	3310	2000	3000
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1930	2890	1770	2660	1610	2410
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
590	885	564	847	510	765
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
475	714	426	641	381	572

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.82	30.0	9.70	28.9	9.64	27.8
Area, in.^{2}					
57.1		52.5		47.6	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
7860	619	7020	555	6310	497
r_y, in.					
3.29		3.25		3.23	
r_x/r_y					
3.56		3.57		3.56	

[c] Shape is slender for compression with F_y = 70 ksi.

Note: Confirm ASTM A913 material availability before specifying.

W27

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W27×						Shape	W27×					
146[c]		129[c]		114[c]		lb/ft	146		129		114	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1700	2560	1470	2210	1280	1920	0	1620	2440	1380	2070	1200	1800
1630	2450	1340	2010	1160	1750	6	1620	2440	1380	2070	1200	1800
1600	2410	1300	1950	1120	1690	7	1620	2440	1360	2050	1180	1770
1570	2360	1250	1870	1080	1620	8	1620	2440	1320	1990	1140	1720
1540	2310	1190	1790	1030	1550	9	1620	2440	1280	1920	1100	1660
1500	2260	1130	1710	980	1470	10	1610	2410	1240	1860	1070	1600
1460	2200	1070	1610	927	1390	11	1570	2360	1200	1800	1030	1540
1420	2130	1010	1520	872	1310	12	1540	2310	1150	1730	989	1490
1370	2070	948	1420	816	1230	13	1500	2250	1110	1670	951	1430
1330	2000	877	1320	760	1140	14	1470	2200	1070	1610	913	1370
1280	1920	803	1210	701	1050	15	1430	2150	1030	1550	874	1310
1230	1850	732	1100	637	957	16	1400	2100	987	1480	836	1260
1180	1770	662	995	575	864	17	1360	2040	945	1420	798	1200
1130	1700	595	894	514	773	18	1330	1990	904	1360	760	1140
1080	1620	534	802	462	694	19	1290	1940	862	1300	715	1070
1020	1530	482	724	417	626	20	1250	1890	805	1210	657	988
902	1360	398	598	344	518	22	1180	1780	695	1050	564	848
790	1190	335	503	289	435	24	1110	1680	611	918	493	740
683	1030	285	428	247	371	26	1040	1570	544	817	436	655
589	885	246	369	213	320	28	949	1430	489	736	391	587
513	771	214	322	185	278	30	851	1280	445	669	354	532
451	678	188	283	163	245	32	769	1160	408	613	323	485
399	600	167	251	144	217	34	701	1050	376	566	297	446
356	535	149	223	129	193	36	644	967	349	525	275	413
320	481					38	594	893	326	490	256	384
289	434					40	552	830	306	460	239	359
262	393					42	515	774	288	433	225	338
239	358					44	483	725	272	409	212	318
218	328					46	454	682	258	388	200	301
200	301					48	429	644	245	368	190	286
185	278					50	406	610	234	351	181	272

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1810	2720	1580	2380	1410	2120
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1460	2190	1280	1910	1130	1700
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
464	696	471	707	436	654
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
341	513	201	302	172	259

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.55	26.9	6.60	19.4	6.51	18.8
Area, in.^{2}					
43.2		37.8		33.6	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
5660	443	4760	184	4080	159
r_y, in.					
3.20		2.21		2.18	
r_x/r_y					
3.59		5.07		5.05	

[c] Shape is slender for compression with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W27

F_y = 70 ksi
F_u = 90 ksi

W27×						Shape	W27×					
102^c		94^c		84^c		lb/ft	102^v		94^v		$84^{f,v}$	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1110	1670	1000	1500	872	1310	0	1070	1600	971	1460	851	1280
1000	1510	904	1360	785	1180	6	1070	1600	971	1460	851	1280
969	1460	872	1310	756	1140	7	1040	1570	949	1430	827	1240
930	1400	836	1260	723	1090	8	1010	1520	916	1380	797	1200
888	1330	797	1200	688	1030	9	975	1470	883	1330	767	1150
843	1270	756	1140	651	979	10	940	1410	850	1280	737	1110
796	1200	713	1070	613	921	11	905	1360	817	1230	707	1060
747	1120	668	1000	573	861	12	870	1310	784	1180	676	1020
698	1050	623	937	533	801	13	835	1250	751	1130	646	971
649	975	578	869	493	741	14	800	1200	718	1080	616	926
599	901	533	802	453	681	15	765	1150	685	1030	586	880
551	828	489	735	414	623	16	729	1100	652	979	555	835
500	752	446	670	376	565	17	694	1040	619	930	525	789
447	671	400	601	341	512	18	659	991	578	869	477	717
401	603	359	539	306	460	19	605	910	527	791	433	651
362	544	324	487	276	415	20	555	834	482	725	396	595
299	450	268	402	228	343	22	474	713	411	618	336	505
251	378	225	338	192	288	24	413	620	356	535	290	437
214	322	192	288	163	246	26	364	547	313	471	255	383
185	277	165	248	141	212	28	325	488	279	419	226	340
161	242	144	216	123	184	30	293	441	251	377	203	305
141	212	126	190	108	162	32	267	401	228	343	184	276
125	188	112	168	95.6	144	34	245	368	209	314	167	252
						36	226	340	192	289	154	231
						38	210	315	178	268	142	214
						40	196	294	166	249	132	199
						42	183	276	155	233	123	186
						44	173	260	146	219	116	174
						46	163	245	138	207	109	164
						48	155	232	130	196	103	155
						50	147	221	124	186	97.5	147

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1260	1890	1160	1740	1040	1560
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1010	1520	932	1400	834	1250
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
351	528	331	498	303	456
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
152	228	136	204	116	174

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
6.42	18.2	6.33	17.7	6.22	17.1
Area, in.^{2}					
30.0		27.6		24.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
3620	139	3270	124	2850	106
r_y, in.					
2.15		2.12		2.07	
r_x/r_y					
5.12		5.14		5.17	

[c] Shape is slender for compression with F_y = 70 ksi.

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W24

F_y = 70 ksi
F_u = 90 ksi

W24×						Shape	W24×					
370^h		335^h		306^h		lb/ft	370^h		335^h		306^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
4570	6870	4120	6190	3760	5650	0	3950	5930	3560	5360	3220	4840
4350	6530	3920	5890	3570	5370	6	3950	5930	3560	5360	3220	4840
4270	6420	3840	5780	3500	5270	7	3950	5930	3560	5360	3220	4840
4180	6290	3760	5660	3430	5150	8	3950	5930	3560	5360	3220	4840
4090	6140	3670	5520	3350	5030	9	3950	5930	3560	5360	3220	4840
3980	5980	3580	5380	3260	4890	10	3940	5920	3550	5330	3200	4810
3870	5810	3470	5220	3160	4750	11	3900	5860	3510	5270	3160	4760
3750	5630	3360	5050	3060	4590	12	3860	5800	3470	5210	3120	4700
3620	5440	3250	4880	2950	4430	13	3820	5740	3430	5150	3090	4640
3490	5240	3120	4690	2840	4260	14	3780	5680	3390	5090	3050	4580
3350	5040	3000	4510	2720	4090	15	3740	5620	3350	5030	3010	4520
3210	4830	2870	4310	2600	3910	16	3700	5560	3310	4970	2970	4460
3070	4610	2740	4120	2480	3730	17	3660	5500	3270	4910	2930	4400
2920	4390	2610	3920	2360	3540	18	3620	5440	3230	4850	2890	4340
2780	4170	2470	3720	2240	3360	19	3580	5380	3190	4790	2850	4280
2630	3960	2340	3520	2110	3180	20	3540	5320	3150	4730	2810	4220
2340	3520	2080	3130	1870	2820	22	3460	5200	3070	4610	2730	4110
2070	3100	1830	2740	1640	2470	24	3380	5080	2990	4490	2650	3990
1800	2700	1580	2380	1420	2130	26	3300	4960	2910	4370	2570	3870
1550	2330	1370	2050	1220	1840	28	3220	4840	2830	4250	2500	3750
1350	2030	1190	1790	1070	1600	30	3140	4720	2750	4130	2420	3630
1190	1790	1050	1570	936	1410	32	3060	4600	2670	4010	2340	3510
1050	1580	926	1390	829	1250	34	2980	4480	2590	3890	2260	3400
939	1410	826	1240	740	1110	36	2900	4360	2510	3770	2180	3280
843	1270	741	1110	664	998	38	2820	4240	2430	3650	2100	3160
760	1140	669	1010	599	901	40	2740	4120	2350	3530	2020	3040
690	1040	607	912	544	817	42	2660	4000	2270	3410	1950	2920
628	944	553	831	495	744	44	2580	3880	2190	3290	1850	2780
575	864	506	760	453	681	46	2510	3770	2110	3170	1760	2650
528	794	465	698	416	625	48	2430	3650	2020	3030	1680	2530
487	731	428	644	384	576	50	2350	3530	1930	2900	1610	2420

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4570	6870	4120	6190	3760	5650
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3680	5520	3320	4980	3030	4540
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1190	1790	1060	1590	956	1430
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
933	1400	831	1250	748	1120

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.76	50.1	9.64	46.0	9.55	42.4
Area, in.^{2}					
109		98.3		89.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
13400	1160	11900	1030	10700	919
r_y, in.					
3.27		3.23		3.20	
r_x/r_y					
3.39		3.41		3.41	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
Note: Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W24

F_y = 70 ksi
F_u = 90 ksi

W24×						Shape	W24×					
279^h		250		229		lb/ft	279^h		250		229	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
3430	5160	3080	4630	2820	4230	0	2920	4380	2600	3910	2360	3540
3260	4890	2920	4390	2670	4010	6	2920	4380	2600	3910	2360	3540
3190	4800	2860	4300	2610	3930	7	2920	4380	2600	3910	2360	3540
3130	4700	2800	4210	2560	3840	8	2920	4380	2600	3910	2360	3540
3050	4580	2730	4100	2490	3740	9	2920	4380	2600	3910	2360	3540
2960	4460	2650	3990	2420	3640	10	2900	4350	2570	3870	2330	3500
2870	4320	2570	3860	2340	3520	11	2860	4290	2540	3810	2290	3450
2780	4180	2480	3730	2260	3400	12	2820	4240	2500	3760	2260	3390
2680	4030	2390	3600	2180	3270	13	2780	4180	2460	3700	2220	3340
2580	3870	2300	3450	2090	3140	14	2740	4120	2420	3640	2180	3280
2470	3710	2200	3310	2000	3000	15	2700	4060	2380	3580	2140	3220
2360	3540	2100	3160	1910	2870	16	2660	4000	2350	3530	2110	3170
2250	3380	2000	3010	1810	2730	17	2620	3940	2310	3470	2070	3110
2130	3210	1900	2850	1720	2580	18	2590	3890	2270	3410	2030	3060
2020	3040	1800	2700	1620	2440	19	2550	3830	2230	3360	2000	3000
1910	2870	1690	2550	1530	2300	20	2510	3770	2190	3300	1960	2940
1690	2540	1490	2250	1350	2020	22	2430	3650	2120	3180	1880	2830
1470	2220	1300	1960	1170	1760	24	2350	3540	2040	3070	1810	2720
1270	1910	1120	1680	1000	1510	26	2280	3420	1970	2960	1730	2610
1100	1650	965	1450	865	1300	28	2200	3300	1890	2840	1660	2500
955	1430	840	1260	754	1130	30	2120	3190	1810	2730	1590	2380
839	1260	739	1110	663	996	32	2040	3070	1740	2610	1510	2270
743	1120	654	983	587	882	34	1970	2950	1660	2500	1440	2160
663	996	584	877	523	787	36	1890	2840	1590	2380	1340	2010
595	894	524	787	470	706	38	1810	2720	1490	2240	1260	1890
537	807	473	711	424	637	40	1720	2590	1400	2110	1180	1780
487	732	429	645	385	578	42	1630	2450	1330	2000	1120	1680
444	667	391	587	350	527	44	1550	2330	1260	1890	1060	1590
406	610	357	537	321	482	46	1480	2220	1200	1800	1010	1510
373	560	328	493	294	443	48	1410	2120	1140	1720	958	1440
344	516	303	455	271	408	50	1350	2020	1090	1640	915	1380

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3430	5160	3080	4630	2820	4230
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2760	4150	2480	3720	2270	3400
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
867	1300	766	1150	699	1050
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
674	1010	597	898	538	809

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.46	39.4	9.37	36.3	9.28	34.0
Area, in.^{2}					
81.9		73.5		67.2	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
9600	823	8490	724	7650	651
r_y, in.					
3.17		3.14		3.11	
r_x/r_y					
3.41		3.41		3.44	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W24

F_y = 70 ksi
F_u = 90 ksi

W24× 207 P_n/Ω_c	W24× 207 $\phi_c P_n$	W24× 192 P_n/Ω_c	W24× 192 $\phi_c P_n$	W24× 176 P_n/Ω_c	W24× 176 $\phi_c P_n$	Shape lb/ft / Design	W24× 207 M_{nx}/Ω_b	W24× 207 $\phi_b M_{nx}$	W24× 192 M_{nx}/Ω_b	W24× 192 $\phi_b M_{nx}$	W24× 176 M_{nx}/Ω_b	W24× 176 $\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2540	3820	2370	3560	2170	3260	0	2120	3180	1950	2930	1780	2680
2410	3620	2240	3360	2050	3080	6	2120	3180	1950	2930	1780	2680
2360	3540	2190	3300	2000	3010	7	2120	3180	1950	2930	1780	2680
2300	3460	2140	3220	1960	2940	8	2120	3180	1950	2930	1780	2680
2240	3370	2090	3140	1900	2860	9	2120	3180	1950	2930	1780	2680
2180	3270	2030	3040	1850	2780	10	2090	3140	1920	2890	1750	2640
2110	3170	1960	2950	1790	2690	11	2050	3080	1890	2840	1720	2580
2030	3060	1890	2840	1720	2590	12	2010	3030	1850	2780	1680	2530
1960	2940	1820	2730	1660	2490	13	1980	2970	1820	2730	1650	2480
1880	2820	1740	2620	1590	2380	14	1940	2920	1780	2680	1620	2430
1790	2700	1670	2500	1510	2270	15	1910	2860	1750	2620	1580	2380
1710	2570	1590	2390	1440	2170	16	1870	2810	1710	2570	1550	2330
1620	2440	1510	2270	1370	2050	17	1830	2760	1680	2520	1510	2270
1540	2310	1430	2140	1290	1940	18	1800	2700	1640	2470	1480	2220
1450	2180	1350	2020	1220	1830	19	1760	2650	1600	2410	1440	2170
1370	2050	1270	1900	1140	1720	20	1720	2590	1570	2360	1410	2120
1200	1800	1110	1670	1000	1510	22	1650	2480	1500	2250	1340	2020
1040	1560	962	1450	865	1300	24	1580	2370	1430	2150	1270	1910
889	1340	822	1240	738	1110	26	1510	2260	1360	2040	1200	1810
767	1150	709	1070	636	956	28	1430	2160	1290	1930	1140	1710
668	1000	618	928	554	833	30	1360	2050	1220	1830	1050	1580
587	882	543	816	487	732	32	1280	1930	1120	1690	963	1450
520	781	481	723	431	648	34	1190	1790	1040	1560	889	1340
464	697	429	645	385	578	36	1110	1660	966	1450	826	1240
416	626	385	579	345	519	38	1040	1560	903	1360	771	1160
376	565	347	522	312	468	40	975	1460	848	1270	723	1090
341	512	315	474	283	425	42	920	1380	800	1200	681	1020
310	467	287	432	258	387	44	871	1310	757	1140	643	967
284	427	263	395	236	354	46	827	1240	718	1080	610	916
261	392	241	363	216	325	48	787	1180	683	1030	580	871
240	361	222	334	199	300	50	752	1130	652	979	552	830

Properties

207		192		176	
Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2540	3820	2370	3560	2170	3260
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2050	3070	1910	2860	1740	2620
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
626	939	578	868	529	794
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
479	719	440	662	402	604

207		192		176	
Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.19	31.7	9.16	30.4	9.08	29.0
Area, in.^{2}					
60.7		56.5		51.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
6820	578	6260	530	5680	479
r_y, in.					
3.08		3.07		3.04	
r_x/r_y					
3.44		3.42		3.45	

Note: Confirm ASTM A913 material availability before specifying.

W24

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W24×						Shape	W24×					
162^c		146^c		131^c		lb/ft	162		146		131	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
2000	3010	1770	2650	1560	2340	0	1630	2460	1460	2190	1290	1940
1890	2840	1680	2520	1480	2220	6	1630	2460	1460	2190	1290	1940
1850	2790	1650	2470	1450	2180	7	1630	2460	1460	2190	1290	1940
1810	2720	1610	2420	1420	2130	8	1630	2460	1460	2190	1290	1940
1760	2650	1570	2360	1380	2080	9	1630	2460	1460	2190	1290	1940
1710	2570	1530	2300	1340	2020	10	1610	2410	1430	2150	1260	1890
1650	2490	1480	2220	1300	1960	11	1570	2360	1400	2100	1230	1850
1590	2400	1430	2140	1260	1890	12	1540	2310	1370	2050	1200	1800
1530	2300	1370	2060	1210	1830	13	1510	2260	1330	2010	1170	1760
1470	2210	1310	1970	1170	1750	14	1470	2210	1300	1960	1140	1720
1400	2110	1250	1880	1110	1670	15	1440	2170	1270	1910	1110	1670
1340	2010	1190	1790	1050	1590	16	1410	2120	1240	1870	1080	1630
1270	1900	1130	1690	998	1500	17	1370	2070	1210	1820	1050	1580
1200	1800	1060	1600	942	1420	18	1340	2020	1180	1770	1030	1540
1130	1700	1000	1510	885	1330	19	1310	1970	1150	1720	996	1500
1060	1600	940	1410	829	1250	20	1280	1920	1120	1680	967	1450
931	1400	820	1230	721	1080	22	1210	1820	1050	1580	908	1360
804	1210	706	1060	617	927	24	1140	1720	991	1490	850	1280
687	1030	602	904	526	790	26	1080	1620	929	1400	781	1170
592	890	519	780	453	681	28	1010	1520	844	1270	697	1050
516	775	452	679	395	594	30	917	1380	763	1150	628	943
453	681	397	597	347	522	32	839	1260	696	1050	570	857
402	603	352	529	307	462	34	773	1160	639	960	523	785
358	538	314	472	274	412	36	716	1080	591	888	482	724
321	483	282	423	246	370	38	667	1000	549	826	447	672
290	436	254	382	222	334	40	625	939	513	771	417	626
263	395	231	346	201	303	42	587	883	482	724	390	586
240	360	210	316	184	276	44	554	833	454	682	367	551
219	330	192	289	168	252	46	525	789	429	645	346	520
201	303	176	265	154	232	48	498	749	407	611	328	493
186	279	163	244			50	474	713	387	581	311	468

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2000	3010	1800	2710	1620	2430
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1610	2420	1450	2180	1300	1950
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
494	740	450	674	415	623
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
367	551	326	489	285	428

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.11	28.0	8.99	26.7	8.87	25.5
Area, in.^{2}					
47.8		43.0		38.6	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
5170	443	4580	391	4020	340
r_y, in.					
3.05		3.01		2.97	
r_x/r_y					
3.41		3.42		3.43	

^{c} Shape is slender for compression with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W24

F_y = 70 ksi
F_u = 90 ksi

W24×						Shape	W24×					
117^c		104^c		103^c		lb/ft	117		104^f		103	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
1360	2040	1180	1780	1180	1780	0	1140	1720	987	1480	978	1470
1280	1930	1120	1680	1050	1590	6	1140	1720	987	1480	976	1470
1260	1890	1100	1650	1010	1520	7	1140	1720	987	1480	943	1420
1230	1850	1070	1610	964	1450	8	1140	1720	987	1480	911	1370
1200	1800	1040	1570	913	1370	9	1140	1710	987	1480	878	1320
1170	1750	1010	1520	859	1290	10	1110	1670	977	1470	846	1270
1130	1700	982	1480	803	1210	11	1080	1630	952	1430	813	1220
1090	1640	948	1420	743	1120	12	1050	1590	927	1390	780	1170
1050	1580	912	1370	677	1020	13	1030	1540	902	1360	748	1120
1010	1520	875	1320	612	920	14	1000	1500	877	1320	715	1070
967	1450	837	1260	550	826	15	973	1460	852	1280	683	1030
923	1390	799	1200	489	735	16	946	1420	826	1240	650	977
879	1320	759	1140	433	651	17	919	1380	801	1200	617	928
830	1250	720	1080	387	581	18	892	1340	776	1170	576	865
779	1170	680	1020	347	521	19	865	1300	751	1130	529	796
729	1100	641	963	313	471	20	838	1260	726	1090	490	736
632	949	554	833	259	389	22	783	1180	676	1020	425	639
539	810	471	708	217	327	24	729	1100	622	935	375	563
459	690	401	603	185	278	26	651	979	544	817	335	504
396	595	346	520	160	240	28	578	869	481	723	303	455
345	518	302	453	139	209	30	519	781	430	647	276	415
303	456	265	398	122	184	32	470	707	389	584	254	382
268	404	235	353			34	430	646	354	532	235	353
239	360	209	315			36	395	594	324	488	219	329
215	323	188	282			38	365	549	299	450	205	308
194	292	170	255			40	340	511	278	417	192	289
176	264	154	231			42	317	477	259	389	181	273
160	241	140	211			44	298	448	242	364	172	258
147	220	128	193			46	281	422	228	342	163	245
135	202	118	177			48	265	398	215	323	155	233
						50	251	378	203	305	148	222

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1440	2170	1290	1930	1270	1910
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1160	1740	1040	1550	1020	1530
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
374	561	337	506	377	566
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
249	375	211	317	145	218

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.78	24.6	9.60	23.8	5.94	17.6
Area, in.^{2}					
34.4		30.7		30.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
3540	297	3100	259	3000	119
r_y, in.					
2.94		2.91		1.99	
r_x/r_y					
3.44		3.47		5.03	

[c] Shape is slender for compression with F_y = 70 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

W24

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W24×						Shape		W24×					
94^c		84^c		76^c		lb/ft		94		84^v		76^v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending		ASD	LRFD	ASD	LRFD	ASD	LRFD
1060	1590	920	1380	816	1230		0	887	1330	782	1180	699	1050
944	1420	816	1230	721	1080		6	884	1330	777	1170	692	1040
905	1360	782	1170	690	1040		7	854	1280	749	1130	665	1000
862	1300	744	1120	655	985		8	823	1240	721	1080	639	961
816	1230	703	1060	618	929		9	793	1190	693	1040	613	922
768	1150	660	991	580	871		10	762	1150	664	999	587	882
717	1080	615	925	539	811		11	731	1100	636	956	561	843
666	1000	570	857	499	750		12	701	1050	608	914	535	804
614	923	525	788	458	688		13	670	1010	580	871	509	765
556	835	480	721	418	628		14	639	961	551	829	483	725
498	749	433	650	379	569		15	609	915	523	786	456	686
443	665	383	576	337	506		16	578	869	495	744	430	647
392	590	339	510	298	448		17	547	823	458	688	387	582
350	526	303	455	266	400		18	501	753	416	626	351	528
314	472	272	408	239	359		19	460	691	381	572	320	482
283	426	245	368	215	324		20	424	638	350	527	294	442
234	352	203	304	178	268		22	366	551	301	453	252	379
197	296	170	256	150	225		24	322	484	264	396	220	330
168	252	145	218	128	192		26	287	431	234	351	194	292
145	217	125	188	110	165		28	258	388	210	315	174	261
126	189	109	164	95.8	144		30	235	353	190	286	157	236
111	166	95.7	144	84.2	127		32	215	324	174	261	143	215
							34	199	299	160	241	131	198
							36	185	277	148	223	121	183
							38	172	259	138	208	113	170
							40	162	243	129	194	106	159
							42	152	229	122	183	99.0	149
							44	144	216	115	172	93.2	140
							46	136	205	109	163	88.1	132
							48	130	195	103	155	83.6	126
							50	124	186	98.2	148	79.5	119

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1160	1750	1040	1560	939	1410
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
935	1400	834	1250	756	1130
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
350	526	285	428	264	398
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
131	197	114	171	99.9	150

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.91	17.2	5.82	16.6	5.73	16.0
Area, in.^{2}					
27.7		24.7		22.4	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
2700	109	2370	94.4	2100	82.5
r_y, in.					
1.98		1.95		1.92	
r_x/r_y					
4.98		5.02		5.05	

[c] Shape is slender for compression with F_y = 70 ksi.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W24

F_y = 70 ksi
F_u = 90 ksi

W24×						Shape	W24×					
68^c		62^c		55^c		lb/ft	68^v		62^v		55^v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
714	1070	638	959	547	823	0	618	929	534	803	468	704
627	943	507	762	429	645	6	608	914	483	725	418	628
599	900	467	702	394	592	7	584	878	455	684	393	590
568	853	424	638	356	535	8	560	841	428	643	368	552
534	803	381	573	318	478	9	535	805	400	601	342	515
499	750	338	508	280	421	10	511	768	372	560	317	477
463	696	295	444	243	365	11	487	732	345	518	292	439
427	641	251	378	211	317	12	463	696	315	473	259	390
390	587	214	322	180	270	13	439	659	274	411	225	338
355	533	185	277	155	233	14	414	623	241	362	197	296
320	480	161	242	135	203	15	390	586	214	322	175	263
287	431	141	212	119	178	16	358	538	192	289	157	235
254	382	125	188	105	158	17	322	483	174	262	141	212
226	340	112	168	93.7	141	18	291	437	159	239	129	193
203	305	100	151	84.1	126	19	265	398	146	219	118	177
183	276	90.4	136	75.9	114	20	243	365	135	202	108	163
152	228	74.7	112	62.7	94.3	22	207	311	116	175	93.3	140
127	191					24	180	270	102	154	81.7	123
109	163					26	158	238	91.2	137	72.6	109
93.6	141					28	141	212	82.2	124	65.2	98.0
81.5	123					30	127	191	74.8	112	59.1	88.9
						32	116	174	68.7	103	54.1	81.3
						34	106	159	63.4	95.3	49.8	74.9
						36	97.5	147	58.9	88.6	46.2	69.4
						38	90.4	136	55.1	82.8	43.1	64.7
						40	84.3	127	51.7	77.7	40.3	60.6
						42	78.9	119	48.7	73.2	37.9	57.0
						44	74.2	112	46.0	69.2	35.8	53.8
						46	70.0	105	43.6	65.6	33.9	51.0
						48	66.3	99.6	41.5	62.4	32.2	48.4
						50	62.9	94.6	39.6	59.5	30.7	46.1

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
843	1270	763	1150	679	1020
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
678	1020	614	921	547	820
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
246	370	256	385	222	334
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
85.6	129	54.8	82.3	46.4	69.7

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.58	15.6	4.12	11.9	4.00	11.5
Area, in.^{2}					
20.1		18.2		16.2	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1830	70.4	1550	34.5	1350	29.1
r_y, in.					
1.87		1.38		1.34	
r_x/r_y					
5.11		6.69		6.80	

[c] Shape is slender for compression with F_y = 70 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W21

F_y = 70 ksi
F_u = 90 ksi

W21×						Shape	W21×					
275[h]		248		223		lb/ft	275[h]		248		223	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
3430	5150	3090	4650	2790	4190	0	2620	3930	2340	3520	2100	3160
3240	4880	2930	4400	2630	3960	6	2620	3930	2340	3520	2100	3160
3180	4780	2870	4310	2580	3870	7	2620	3930	2340	3520	2100	3160
3110	4670	2800	4210	2520	3780	8	2620	3930	2340	3520	2100	3160
3030	4550	2730	4100	2450	3680	9	2620	3930	2340	3520	2100	3160
2940	4420	2650	3980	2380	3570	10	2590	3900	2320	3490	2070	3120
2850	4280	2560	3850	2300	3450	11	2570	3860	2290	3440	2040	3070
2750	4130	2470	3720	2220	3330	12	2540	3810	2260	3400	2020	3030
2650	3980	2380	3580	2130	3200	13	2510	3770	2230	3360	1990	2990
2540	3820	2280	3430	2040	3060	14	2480	3720	2210	3310	1960	2940
2430	3650	2180	3280	1950	2930	15	2450	3680	2180	3270	1930	2900
2320	3480	2080	3120	1850	2790	16	2420	3640	2150	3230	1900	2860
2200	3310	1970	2970	1760	2640	17	2390	3590	2120	3190	1870	2810
2090	3140	1870	2810	1660	2500	18	2360	3550	2090	3140	1840	2770
1970	2960	1770	2650	1570	2360	19	2330	3500	2060	3100	1810	2730
1860	2790	1660	2500	1470	2210	20	2300	3460	2030	3060	1790	2680
1630	2450	1460	2190	1290	1940	22	2240	3370	1980	2970	1730	2600
1420	2130	1260	1900	1110	1670	24	2190	3290	1920	2880	1670	2510
1210	1820	1080	1620	949	1430	26	2130	3200	1860	2800	1610	2420
1050	1570	932	1400	818	1230	28	2070	3110	1800	2710	1560	2340
912	1370	812	1220	713	1070	30	2010	3020	1750	2620	1500	2250
801	1200	714	1070	626	942	32	1950	2930	1690	2540	1440	2170
710	1070	632	950	555	834	34	1890	2850	1630	2450	1380	2080
633	952	564	847	495	744	36	1840	2760	1570	2360	1330	1990
568	854	506	761	444	668	38	1780	2670	1510	2280	1270	1900
513	771	457	686	401	603	40	1720	2580	1460	2190	1200	1800
465	699	414	623	364	547	42	1660	2500	1400	2100	1130	1700
424	637	377	567	331	498	44	1600	2410	1330	2000	1080	1620
388	583	345	519	303	456	46	1540	2310	1270	1900	1020	1540
356	535	317	477	278	418	48	1470	2210	1210	1820	978	1470
328	493	292	439	257	386	50	1410	2120	1160	1740	936	1410

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3430	5150	3090	4650	2790	4190
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2760	4140	2490	3740	2240	3370
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
823	1230	730	1090	655	983
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
667	1000	594	893	524	788

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.25	45.4	9.19	41.7	9.08	37.9
Area, in.^{2}					
81.8		73.8		66.5	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
7690	787	6830	699	6080	614
r_y, in.					
3.10		3.08		3.04	
r_x/r_y					
3.13		3.12		3.14	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W21

F_y = 70 ksi
F_u = 90 ksi

W-Shapes

W21×						Shape	W21×					
201		182		166		lb/ft	201		182		166	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Design	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2490	3740	2250	3380	2050	3070	0	1850	2780	1660	2500	1510	2270
2350	3520	2120	3180	1930	2900	6	1850	2780	1660	2500	1510	2270
2300	3450	2070	3120	1890	2840	7	1850	2780	1660	2500	1510	2270
2240	3370	2020	3040	1840	2770	8	1850	2780	1660	2500	1510	2270
2180	3280	1970	2960	1790	2690	9	1850	2780	1660	2500	1510	2260
2110	3180	1910	2870	1730	2610	10	1820	2740	1630	2460	1480	2220
2040	3070	1840	2770	1680	2520	11	1790	2700	1610	2410	1450	2180
1970	2960	1770	2670	1610	2420	12	1770	2650	1580	2370	1430	2140
1890	2840	1700	2560	1550	2330	13	1740	2610	1550	2330	1400	2100
1810	2720	1630	2450	1480	2230	14	1710	2570	1520	2290	1370	2060
1730	2600	1550	2340	1410	2120	15	1680	2530	1500	2250	1340	2020
1640	2470	1480	2220	1340	2020	16	1650	2480	1470	2210	1320	1980
1560	2340	1400	2100	1270	1910	17	1620	2440	1440	2160	1290	1940
1470	2210	1320	1990	1200	1800	18	1600	2400	1410	2120	1260	1900
1390	2080	1240	1870	1130	1700	19	1570	2360	1380	2080	1240	1860
1300	1960	1170	1750	1060	1590	20	1540	2310	1360	2040	1210	1820
1140	1710	1020	1530	921	1380	22	1480	2230	1300	1960	1160	1740
980	1470	874	1310	791	1190	24	1420	2140	1250	1870	1100	1660
835	1260	745	1120	674	1010	26	1370	2060	1190	1790	1050	1570
720	1080	642	965	581	873	28	1310	1970	1130	1710	993	1490
627	943	559	841	506	760	30	1250	1880	1080	1620	939	1410
551	829	492	739	445	668	32	1200	1800	1020	1540	868	1310
488	734	436	655	394	592	34	1140	1710	952	1430	805	1210
436	655	389	584	351	528	36	1070	1610	888	1330	750	1130
391	588	349	524	315	474	38	1000	1510	833	1250	703	1060
353	530	315	473	285	428	40	947	1420	784	1180	661	993
320	481	285	429	258	388	42	896	1350	741	1110	624	938
292	438	260	391	235	354	44	850	1280	703	1060	591	888
267	401	238	358	215	323	46	809	1220	668	1000	562	844
245	368	219	328	198	297	48	771	1160	637	957	535	804
226	339	201	303			50	737	1110	609	915	511	768

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2490	3740	2250	3380	2050	3070
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2000	3000	1810	2710	1650	2470
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
586	879	528	791	473	709
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
465	698	416	625	377	567

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.02	34.5	8.96	32.2	8.93	30.4
Area, in.^{2}					
59.3		53.6		48.8	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
5310	542	4730	483	4280	435
r_y, in.					
3.02		3.00		2.99	
r_x/r_y					
3.14		3.13		3.13	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W21

F_y = 70 ksi
F_u = 90 ksi

W21×						Shape		W21×					
147		132		122^c		lb/ft		147		132		122	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending		ASD	LRFD	ASD	LRFD	ASD	LRFD
1810	2720	1630	2440	1490	2250		0	1300	1960	1160	1750	1070	1610
1700	2560	1530	2300	1410	2130		6	1300	1960	1160	1750	1070	1610
1670	2500	1500	2250	1380	2080		7	1300	1960	1160	1750	1070	1610
1620	2440	1460	2190	1350	2020		8	1300	1960	1160	1750	1070	1610
1580	2370	1420	2130	1310	1970		9	1300	1950	1160	1740	1070	1600
1530	2300	1370	2060	1270	1900		10	1270	1910	1130	1700	1040	1570
1480	2220	1320	1990	1220	1830		11	1250	1870	1110	1670	1020	1530
1420	2130	1270	1910	1170	1760		12	1220	1830	1080	1630	995	1500
1360	2040	1220	1830	1120	1690		13	1190	1800	1060	1590	971	1460
1300	1950	1160	1750	1070	1610		14	1170	1760	1030	1560	948	1420
1240	1860	1110	1660	1020	1530		15	1140	1720	1010	1520	924	1390
1170	1760	1050	1570	967	1450		16	1120	1680	986	1480	900	1350
1110	1670	990	1490	913	1370		17	1090	1640	961	1440	877	1320
1050	1570	932	1400	859	1290		18	1070	1600	937	1410	853	1280
982	1480	875	1320	806	1210		19	1040	1560	912	1370	830	1250
920	1380	818	1230	754	1130		20	1010	1520	888	1330	806	1210
798	1200	708	1060	652	980		22	963	1450	839	1260	759	1140
681	1020	604	907	555	834		24	911	1370	790	1190	712	1070
580	872	514	773	473	710		26	860	1290	741	1110	662	995
501	752	443	667	408	613		28	808	1210	677	1020	594	892
436	655	386	581	355	534		30	737	1110	615	924	538	808
383	576	340	510	312	469		32	676	1020	563	846	491	738
339	510	301	452	276	415		34	625	939	519	779	452	679
303	455	268	403	247	371		36	581	873	481	723	418	629
272	408	241	362	221	333		38	542	815	448	674	390	585
245	369	217	327	200	300		40	509	765	420	631	364	548
222	334	197	296	181	272		42	480	721	395	594	342	515
203	305	180	270	165	248		44	453	681	373	561	323	485
185	279	164	247	151	227		46	430	646	353	531	306	459
170	256	151	227	139	208		48	409	615	336	505	290	436
							50	390	586	320	481	276	415

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1810	2720	1630	2440	1500	2260
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1460	2190	1310	1960	1210	1820
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
446	668	397	595	365	547
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
323	486	287	432	264	397

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.81	28.1	8.75	26.8	8.72	25.9
Area, in.^{2}					
43.2		38.8		35.9	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
3630	376	3220	333	2960	305
r_y, in.					
2.95		2.93		2.92	
r_x/r_y					
3.11		3.11		3.11	

[c] Shape is slender for compression with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

W21

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W21×						Shape	W21×					
111[c]		101[c]		93[c]		lb/ft	111		101		93	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
1330	2000	1190	1790	1120	1690	0	975	1460	884	1330	772	1160
1260	1890	1130	1690	978	1470	6	975	1460	884	1330	759	1140
1230	1850	1100	1660	924	1390	7	975	1460	884	1330	732	1100
1210	1810	1080	1620	866	1300	8	975	1460	884	1330	706	1060
1170	1760	1050	1580	804	1210	9	967	1450	876	1320	679	1020
1140	1710	1020	1530	740	1110	10	944	1420	855	1280	653	981
1100	1660	986	1480	676	1020	11	922	1390	834	1250	626	941
1060	1600	951	1430	611	919	12	899	1350	813	1220	600	901
1020	1530	914	1370	548	824	13	877	1320	791	1190	573	861
969	1460	876	1320	487	733	14	855	1280	770	1160	547	822
921	1380	837	1260	429	644	15	832	1250	749	1130	520	782
872	1310	795	1190	377	566	16	810	1220	728	1090	494	742
823	1240	750	1130	334	502	17	787	1180	707	1060	465	700
774	1160	705	1060	298	448	18	765	1150	686	1030	427	642
726	1090	661	993	267	402	19	742	1120	665	999	395	594
678	1020	617	927	241	363	20	720	1080	644	968	367	551
585	879	532	799	199	300	22	675	1010	602	904	321	482
497	747	451	678	167	252	24	630	947	559	841	285	429
423	636	384	578	143	215	26	570	857	495	744	257	386
365	549	331	498	123	185	28	510	767	441	663	233	351
318	478	289	434	107	161	30	461	692	397	597	214	321
279	420	254	381			32	420	631	361	543	197	297
248	372	225	338			34	385	579	331	497	183	276
221	332	200	301			36	356	535	305	458	171	258
198	298	180	270			38	331	497	283	425	161	242
179	269	162	244			40	309	464	263	396	151	228
162	244	147	221			42	290	436	247	371	143	215
148	222	134	202			44	273	410	232	349	136	204
135	203	123	185			46	258	388	219	329	129	194
124	187	113	169			48	245	367	207	311	123	185
						50	232	349	197	296	118	177

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1370	2050	1250	1880	1140	1720
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1100	1650	1010	1510	921	1380
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
331	497	300	449	351	526
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
238	358	216	324	121	182

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.66	24.9	8.63	24.2	5.49	16.9
Area, in.^{2}					
32.6		29.8		27.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
2670	274	2420	248	2070	92.9
r_y, in.					
2.9		2.89		1.84	
r_x/r_y					
3.12		3.12		4.73	

[c] Shape is slender for compression with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W21

F_y = 70 ksi
F_u = 90 ksi

W21×						Shape	W21×					
83^c		73^c		68^c		lb/ft	83		73		68	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
973	1460	828	1240	757	1140	0	685	1030	601	903	559	840
848	1280	720	1080	658	989	6	671	1010	587	883	545	819
808	1210	685	1030	625	939	7	646	972	564	848	523	786
763	1150	646	971	589	886	8	622	934	542	814	501	753
715	1070	605	909	552	829	9	597	897	519	780	479	720
659	990	562	845	512	770	10	572	859	496	746	457	687
600	902	518	778	472	709	11	547	822	474	712	435	654
543	816	471	709	431	648	12	522	785	451	677	413	621
486	731	421	633	389	584	13	497	747	428	643	392	588
432	649	373	561	344	517	14	472	710	405	609	370	555
379	570	327	491	301	452	15	447	673	383	575	348	522
333	501	287	432	264	397	16	423	635	353	531	315	474
295	444	254	382	234	352	17	387	581	320	481	285	429
263	396	227	341	209	314	18	354	532	292	440	260	391
236	355	204	306	187	282	19	326	491	269	404	239	359
213	320	184	276	169	254	20	302	455	248	373	220	331
176	265	152	228	140	210	22	264	396	215	324	191	286
148	223	128	192	117	176	24	233	351	190	285	168	252
126	190	109	163	100	150	26	209	314	169	255	149	224
109	164	93.8	141	86.3	130	28	190	285	153	230	135	202
94.8	142	81.7	123	75.2	113	30	173	261	140	210	122	184
						32	160	240	128	193	112	169
						34	148	223	119	178	104	156
						36	138	208	110	166	96.4	145
						38	129	195	103	155	90.0	135
						40	122	183	96.9	146	84.5	127
						42	115	173	91.3	137	79.6	120
						44	109	164	86.4	130	75.2	113
						46	104	156	82.0	123	71.3	107
						48	98.6	148	78.0	117	67.8	102
						50	94.2	142	74.4	112	64.7	97.2

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1020	1540	901	1350	838	1260
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
824	1240	726	1090	675	1010
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
309	463	270	405	254	381
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
107	160	92.9	140	85.2	128

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.46	16.2	5.40	15.6	5.37	15.2
Area, in.^{2}					
24.4		21.5		20.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1830	81.4	1600	70.6	1480	64.7
r_y, in.					
1.83		1.81		1.80	
r_x/r_y					
4.74		4.77		4.78	

^{c} Shape is slender for compression with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W21

F_y = 70 ksi
F_u = 90 ksi

W21× 62^c		W21× 55^c		W21× 48^c		Shape, lb/ft	W21× 62^v		W21× 55^{f,v}		W21× 48^{f,v}	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
678	1020	585	879	492	740	0	503	756	438	659	354	531
586	881	502	755	418	629	6	489	734	425	638	354	531
556	836	476	715	395	593	7	468	704	406	610	340	510
523	786	447	671	369	554	8	448	674	387	582	323	485
488	734	416	625	342	513	9	428	644	368	554	306	460
452	680	384	578	314	472	10	408	613	350	526	289	435
416	625	352	529	286	429	11	388	583	331	498	273	410
379	569	320	481	258	387	12	368	553	312	470	256	385
343	515	288	433	230	346	13	348	523	294	441	239	360
305	458	257	386	204	307	14	328	492	275	413	219	329
266	400	225	338	180	271	15	305	458	248	372	193	290
234	351	198	297	158	238	16	273	410	221	332	172	259
207	311	175	263	140	211	17	246	370	199	299	155	233
185	278	156	235	125	188	18	224	337	181	272	140	210
166	249	140	211	112	169	19	205	309	165	248	128	192
150	225	127	190	101	152	20	189	284	152	228	117	176
124	186	105	157	83.8	126	22	163	245	130	195	99.8	150
104	156	87.9	132	70.4	106	24	143	214	113	170	86.7	130
88.5	133	74.9	113	60.0	90.2	26	127	190	100	151	76.3	115
76.3	115	64.6	97.0			28	114	171	89.8	135	68.1	102
						30	103	155	81.2	122	61.3	92.2
						32	94.4	142	74.0	111	55.8	83.8
						34	87.0	131	68.0	102	51.1	76.8
						36	80.7	121	62.9	94.6	47.1	70.8
						38	75.2	113	58.5	87.9	43.7	65.7
						40	70.4	106	54.7	82.2	40.7	61.2
						42	66.2	99.6	51.3	77.1	38.2	57.4
						44	62.5	94.0	48.4	72.7	35.9	53.9
						46	59.2	89.0	45.7	68.7	33.9	50.9
						48	56.3	84.5	43.4	65.2	32.1	48.2
						50	53.6	80.5	41.2	62.0	30.4	45.8

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
767	1150	679	1020	591	888
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
618	926	547	820	476	714
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
211	318	196	295	175	263
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
75.8	114	63.9	96.0	48.1	72.3

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.28	14.8	5.26	14.3	6.16	13.7
Area, in.^{2}					
18.3		16.2		14.1	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1330	57.5	1140	48.4	959	38.7
r_y, in.					
1.77		1.73		1.66	
r_x/r_y					
4.82		4.86		4.96	

[c] Shape is slender for compression with F_y = 70 ksi.

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W21

F_y = 70 ksi
F_u = 90 ksi

W21×						Shape		W21×					
57^c		50^c		44^c		lb/ft		57^v		50^v		44^v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending		ASD	LRFD	ASD	LRFD	ASD	LRFD
612	920	523	785	446	671		0	451	677	384	578	333	501
479	720	402	605	338	508		6	405	608	340	511	290	436
438	659	366	550	306	460		7	381	573	319	479	271	408
396	595	328	494	273	410		8	358	538	298	448	252	379
353	530	290	436	240	360		9	334	503	277	416	233	350
310	466	253	380	207	312		10	311	467	256	385	214	322
263	395	214	322	177	267		11	288	432	235	353	191	287
221	332	180	271	150	225		12	259	390	204	307	164	246
188	283	153	231	127	192		13	227	341	178	267	142	214
162	244	132	199	110	165		14	201	302	157	236	125	188
141	212	115	173	95.7	144		15	180	270	140	210	111	167
124	187	101	152	84.2	126		16	163	244	126	189	99.9	150
110	165	89.7	135	74.5	112		17	148	222	114	172	90.4	136
98.1	147	80.0	120	66.5	99.9		18	136	204	105	157	82.4	124
88.0	132	71.8	108	59.7	89.7		19	125	188	96.2	145	75.6	114
79.4	119	64.8	97.4	53.9	80.9		20	116	175	89.0	134	69.8	105
65.6	98.7						22	101	153	77.3	116	60.3	90.7
							24	90.0	135	68.2	103	53.0	79.7
							26	80.8	121	61.0	91.7	47.3	71.0
							28	73.4	110	55.2	82.9	42.6	64.0
							30	67.2	101	50.4	75.7	38.8	58.3
							32	62.0	93.1	46.3	69.6	35.6	53.4
							34	57.5	86.4	42.9	64.5	32.8	49.4
							36	53.7	80.7	39.9	60.0	30.5	45.9
							38	50.3	75.6	37.4	56.2	28.5	42.8
							40	47.4	71.2	35.1	52.8	26.7	40.2
							42	44.8	67.3	33.1	49.8	25.2	37.9
							44	42.4	63.8	31.4	47.2	23.8	35.8
							46	40.3	60.6	29.8	44.8	22.6	33.9
							48	38.5	57.8	28.4	42.6	21.5	32.3
							50	36.7	55.2	27.1	40.7	20.5	30.8

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
700	1050	616	926	545	819
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
564	845	496	744	439	658
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
215	323	199	299	176	264
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
51.7	77.7	42.6	64.1	35.6	53.5

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
4.03	11.7	3.88	11.2	3.76	10.8
Area, in.^{2}					
16.7		14.7		13.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1170	30.6	984	24.9	843	20.7
r_y, in.					
1.35		1.30		1.26	
r_x/r_y					
6.19		6.29		6.40	

[c] Shape is slender for compression with F_y = 70 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W18

F_y = 70 ksi
F_u = 90 ksi

W18×						Shape		W18×					
311[h]		283[h]		258[h]		lb/ft		311[h]		283[h]		258[h]	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$			M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Design		Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
3840	5770	3490	5250	3190	4790	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	2630	3960	2360	3550	2130	3210
3610	5430	3280	4930	2990	4490		6	2630	3960	2360	3550	2130	3210
3530	5310	3210	4820	2920	4390		7	2630	3960	2360	3550	2130	3210
3450	5180	3120	4690	2840	4270		8	2630	3960	2360	3550	2130	3210
3350	5030	3030	4560	2760	4150		9	2630	3950	2350	3540	2130	3190
3240	4870	2930	4410	2670	4010		10	2610	3920	2330	3510	2100	3160
3130	4700	2830	4250	2570	3860		11	2580	3880	2310	3470	2080	3130
3010	4520	2720	4080	2470	3710		12	2560	3850	2290	3440	2060	3100
2880	4330	2600	3910	2360	3550		13	2540	3820	2270	3410	2040	3060
2750	4140	2480	3730	2250	3380		14	2520	3780	2240	3370	2020	3030
2620	3940	2360	3550	2140	3210		15	2490	3750	2220	3340	1990	3000
2490	3740	2240	3360	2020	3040		16	2470	3720	2200	3310	1970	2960
2350	3540	2110	3170	1910	2860		17	2450	3680	2180	3270	1950	2930
2220	3330	1990	2990	1790	2690		18	2430	3650	2160	3240	1930	2900
2080	3130	1860	2800	1680	2520		19	2410	3620	2130	3210	1910	2860
1950	2930	1740	2620	1560	2350		20	2380	3580	2110	3170	1880	2830
1690	2540	1500	2260	1350	2030		22	2340	3510	2070	3110	1840	2770
1440	2170	1280	1920	1140	1720		24	2290	3450	2020	3040	1800	2700
1230	1850	1090	1640	973	1460		26	2250	3380	1980	2970	1750	2630
1060	1600	939	1410	839	1260		28	2200	3310	1930	2910	1710	2570
925	1390	818	1230	731	1100		30	2160	3240	1890	2840	1660	2500
813	1220	719	1080	643	966		32	2110	3180	1850	2770	1620	2430
720	1080	637	957	569	855		34	2070	3110	1800	2710	1580	2370
642	965	568	854	508	763		36	2020	3040	1760	2640	1530	2300
576	866	510	766	456	685		38	1980	2980	1710	2570	1490	2240
520	782	460	692	411	618		40	1930	2910	1670	2510	1440	2170
472	709	417	627	373	561		42	1890	2840	1620	2440	1400	2100
430	646	380	572	340	511		44	1850	2770	1580	2380	1360	2040
393	591	348	523	311	467		46	1800	2710	1540	2310	1310	1970
361	543	320	480	286	429		48	1760	2640	1490	2240	1270	1910
							50	1710	2570	1450	2180	1220	1830

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3840	5770	3490	5250	3190	4790
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3090	4640	2810	4220	2570	3850
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
949	1420	858	1290	771	1160
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
723	1090	646	971	580	872

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.81	58.2	8.69	53.0	8.60	48.5
Area, in.^{2}					
91.6		83.3		76.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
6970	795	6170	704	5510	628
r_y, in.					
2.95		2.91		2.88	
r_x/r_y					
2.96		2.96		2.96	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W18

F_y = 70 ksi
F_u = 90 ksi

W18×						Shape	W18×					
234^h		211		192		lb/ft	234^h		211		192	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2880	4320	2610	3920	2360	3540	0	1920	2880	1710	2570	1540	2320
2690	4050	2440	3670	2200	3310	6	1920	2880	1710	2570	1540	2320
2630	3950	2380	3580	2150	3230	7	1920	2880	1710	2570	1540	2320
2560	3850	2320	3490	2090	3140	8	1920	2880	1710	2570	1540	2320
2480	3730	2250	3380	2020	3040	9	1910	2870	1700	2550	1530	2300
2400	3600	2170	3260	1950	2930	10	1890	2830	1680	2520	1510	2270
2310	3470	2090	3140	1870	2820	11	1860	2800	1660	2490	1490	2240
2210	3330	2000	3010	1790	2700	12	1840	2770	1630	2460	1470	2200
2120	3180	1910	2870	1710	2570	13	1820	2740	1610	2420	1440	2170
2010	3030	1820	2730	1630	2440	14	1800	2700	1590	2390	1420	2140
1910	2870	1720	2590	1540	2310	15	1780	2670	1570	2360	1400	2110
1810	2720	1620	2440	1450	2180	16	1760	2640	1550	2330	1380	2080
1700	2560	1530	2300	1360	2050	17	1730	2610	1530	2300	1360	2040
1600	2400	1430	2150	1280	1920	18	1710	2570	1510	2260	1340	2010
1490	2240	1340	2010	1190	1790	19	1690	2540	1480	2230	1320	1980
1390	2090	1240	1870	1100	1660	20	1670	2510	1460	2200	1300	1950
1190	1800	1060	1600	942	1420	22	1630	2440	1420	2140	1250	1880
1010	1520	898	1350	793	1190	24	1580	2380	1380	2070	1210	1820
860	1290	765	1150	675	1020	26	1540	2310	1330	2010	1170	1760
742	1120	660	991	582	875	28	1500	2250	1290	1940	1130	1690
646	971	575	864	507	763	30	1450	2180	1250	1880	1080	1630
568	854	505	759	446	670	32	1410	2120	1210	1810	1040	1560
503	756	447	672	395	594	34	1370	2050	1160	1750	999	1500
449	675	399	600	352	530	36	1320	1990	1120	1680	956	1440
403	605	358	538	316	475	38	1280	1920	1080	1620	909	1370
364	546	323	486	285	429	40	1240	1860	1030	1560	860	1290
330	496	293	441	259	389	42	1190	1790	984	1480	815	1230
300	452	267	401	236	355	44	1150	1730	937	1410	776	1170
275	413	244	367	216	324	46	1100	1650	894	1340	740	1110
						48	1050	1580	854	1280	707	1060
						50	1010	1510	818	1230	677	1020

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2880	4320	2610	3920	2360	3540
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2320	3470	2100	3150	1900	2850
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
685	1030	614	922	548	823
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
520	782	461	693	416	625

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.51	44.4	8.42	40.5	8.33	37.3
Area, in.^{2}					
68.6		62.3		56.2	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
4900	558	4330	493	3870	440
r_y, in.					
2.85		2.82		2.79	
r_x/r_y					
2.96		2.96		2.97	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W18

F_y = 70 ksi
F_u = 90 ksi

W18×						Shape	W18×					
175		158		143		lb/ft	175		158		143	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Design	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2150	3240	1940	2920	1760	2650	0	1390	2090	1240	1870	1120	1690
2010	3020	1810	2720	1640	2460	6	1390	2090	1240	1870	1120	1690
1960	2950	1760	2650	1600	2400	7	1390	2090	1240	1870	1120	1690
1900	2860	1710	2570	1550	2330	8	1390	2090	1240	1870	1120	1690
1840	2770	1660	2490	1500	2250	9	1370	2070	1230	1840	1110	1660
1780	2670	1590	2400	1440	2170	10	1350	2030	1210	1810	1090	1630
1700	2560	1530	2300	1380	2080	11	1330	2000	1190	1780	1070	1600
1630	2450	1460	2200	1320	1990	12	1310	1970	1170	1750	1050	1570
1550	2330	1390	2090	1260	1890	13	1290	1940	1140	1720	1030	1540
1470	2220	1320	1990	1190	1790	14	1270	1910	1120	1690	1010	1510
1390	2100	1250	1880	1120	1690	15	1250	1880	1100	1660	986	1480
1310	1970	1170	1760	1060	1590	16	1230	1850	1080	1630	966	1450
1230	1850	1100	1650	990	1490	17	1210	1810	1060	1600	946	1420
1150	1730	1030	1540	923	1390	18	1190	1780	1040	1570	926	1390
1070	1610	955	1440	858	1290	19	1170	1750	1020	1530	906	1360
994	1490	885	1330	793	1190	20	1140	1720	1000	1500	886	1330
845	1270	750	1130	670	1010	22	1100	1660	960	1440	845	1270
710	1070	630	947	563	846	24	1060	1590	919	1380	805	1210
605	909	537	807	480	721	26	1020	1530	878	1320	765	1150
521	784	463	696	414	622	28	977	1470	836	1260	724	1090
454	683	403	606	360	542	30	935	1410	795	1200	682	1020
399	600	354	533	317	476	32	893	1340	752	1130	631	948
354	531	314	472	281	422	34	851	1280	701	1050	587	883
315	474	280	421	250	376	36	801	1200	657	988	549	826
283	425	251	378	225	338	38	754	1130	618	929	516	776
255	384	227	341	203	305	40	713	1070	584	877	487	732
232	348	206	309	184	276	42	676	1020	553	831	461	693
211	317	187	282	168	252	44	643	966	525	790	438	658
193	290					46	612	920	500	752	417	626
						48	585	879	478	718	398	598
						50	560	842	457	687	380	572

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2150	3240	1940	2920	1760	2650
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1730	2600	1560	2340	1420	2130
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
498	748	447	670	399	598
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
370	557	331	498	298	448

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.24	34.5	8.18	31.8	8.12	29.7
Area, in.^{2}					
51.4		46.3		42.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
3450	391	3060	347	2750	311
r_y, in.					
2.76		2.74		2.72	
r_x/r_y					
2.97		2.96		2.97	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W18

F_y = 70 ksi
F_u = 90 ksi

W18×						Shape		W18×					
130		119		106		lb/ft		130		119		106	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending		ASD	LRFD	ASD	LRFD	ASD	LRFD
1610	2410	1470	2210	1300	1960		0	1010	1520	915	1380	803	1210
1490	2240	1370	2050	1210	1820		6	1010	1520	915	1380	803	1210
1450	2190	1330	2000	1180	1770		7	1010	1520	915	1380	803	1210
1410	2120	1290	1940	1140	1710		8	1010	1520	915	1380	802	1210
1360	2050	1250	1870	1100	1660		9	995	1490	897	1350	784	1180
1310	1970	1200	1800	1060	1590		10	975	1470	877	1320	766	1150
1260	1890	1150	1730	1010	1520		11	955	1440	858	1290	748	1120
1200	1800	1100	1650	966	1450		12	936	1410	839	1260	730	1100
1140	1710	1040	1570	917	1380		13	916	1380	820	1230	712	1070
1080	1620	987	1480	867	1300		14	896	1350	801	1200	694	1040
1020	1530	930	1400	816	1230		15	877	1320	782	1170	676	1020
957	1440	873	1310	765	1150		16	857	1290	763	1150	658	988
895	1350	817	1230	714	1070		17	838	1260	743	1120	640	961
834	1250	760	1140	664	998		18	818	1230	724	1090	622	934
774	1160	705	1060	615	924		19	798	1200	705	1060	603	907
715	1070	651	979	567	852		20	779	1170	686	1030	585	880
602	905	548	823	475	713		22	740	1110	648	973	549	825
506	760	460	692	399	599		24	700	1050	609	916	513	771
431	648	392	589	340	511		26	661	994	571	858	466	701
372	559	338	508	293	440		28	620	931	520	782	421	633
324	487	295	443	255	384		30	568	854	476	716	384	578
285	428	259	389	224	337		32	525	789	439	660	353	531
252	379	229	345	199	299		34	488	734	407	612	327	492
225	338	205	307	177	266		36	456	686	380	571	305	458
202	303	184	276	159	239		38	428	644	356	535	285	428
182	274	166	249	144	216		40	404	607	335	504	268	402
165	248	150	226	130	196		42	382	574	316	476	252	379
151	226	137	206	119	178		44	362	544	300	451	239	359
							46	345	518	285	428	227	341
							48	329	494	272	408	216	325
							50	314	472	259	390	206	310

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1610	2410	1470	2210	1300	1960
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1290	1940	1180	1780	1050	1570
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
362	543	348	523	309	463
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
268	403	241	363	211	318

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.06	27.8	8.03	26.3	7.94	24.8
Area, in.^{2}					
38.3		35.1		31.1	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
2460	278	2190	253	1910	220
r_y, in.					
2.70		2.69		2.66	
r_x/r_y					
2.97		2.94		2.95	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W18

F_y = 70 ksi
F_u = 90 ksi

W-Shapes

W18×						Shape	W18×					
97		86^c		76^c		lb/ft	97		86		76^f	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1190	1800	1040	1560	892	1340	0	737	1110	650	977	563	846
1110	1660	970	1460	832	1250	6	737	1110	650	977	563	846
1080	1620	946	1420	811	1220	7	737	1110	650	977	563	846
1040	1570	920	1380	788	1180	8	735	1110	647	973	563	846
1010	1510	890	1340	763	1150	9	718	1080	631	948	551	828
968	1460	857	1290	735	1100	10	701	1050	615	924	536	805
927	1390	819	1230	706	1060	11	683	1030	598	899	521	783
883	1330	780	1170	675	1010	12	666	1000	582	874	506	760
838	1260	740	1110	643	967	13	649	975	565	850	490	737
792	1190	698	1050	611	918	14	631	949	549	825	475	714
745	1120	657	987	574	863	15	614	923	533	801	460	692
698	1050	615	924	537	807	16	596	896	516	776	445	669
651	979	573	861	500	752	17	579	870	500	751	430	646
605	910	532	799	464	697	18	562	844	484	727	415	623
560	842	491	738	428	643	19	544	818	467	702	400	601
516	775	452	680	393	591	20	527	792	451	678	384	578
432	649	377	567	328	492	22	492	740	418	628	352	528
363	545	317	477	275	414	24	456	685	374	562	306	461
309	464	270	406	235	353	26	406	610	332	499	271	407
266	400	233	350	202	304	28	366	550	298	448	242	364
232	349	203	305	176	265	30	333	501	270	406	219	329
204	307	178	268	155	233	32	306	460	247	372	200	300
181	272	158	237	137	206	34	283	425	228	343	183	276
161	242	141	212	122	184	36	263	395	211	318	170	255
145	217	126	190	110	165	38	245	369	197	296	158	237
131	196	114	172	99.1	149	40	230	346	184	277	147	221
118	178	104	156	89.9	135	42	217	326	173	261	138	208
108	162					44	205	308	164	246	130	196
						46	194	292	155	233	123	185
						48	185	278	147	221	117	175
						50	176	265	140	211	111	167

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1190	1800	1060	1590	935	1400
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
962	1440	854	1280	753	1130
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
279	418	247	371	217	325
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
193	290	169	254	145	218

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.91	23.9	7.85	22.7	8.21	21.8
Area, in.^{2}					
28.5		25.3		22.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1750	201	1530	175	1330	152
r_y, in.					
2.65		2.63		2.61	
r_x/r_y					
2.95		2.95		2.96	

[c] Shape is slender for compression with F_y = 70 ksi.

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

W18

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

F_y = 70 ksi
F_u = 90 ksi

W-Shapes

W18×						Shape	W18×					
71^c		65^c		60^c		lb/ft	71		65		60	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
861	1290	767	1150	692	1040	0	510	767	465	698	430	646
729	1100	653	981	588	883	6	492	740	447	673	413	620
682	1030	616	925	554	833	7	473	711	430	646	396	595
632	950	575	865	518	778	8	454	683	412	619	378	569
580	871	527	792	479	720	9	435	654	394	592	361	543
526	791	478	718	438	658	10	416	625	376	565	344	517
473	710	429	644	392	589	11	397	597	358	538	327	491
420	632	381	572	348	523	12	378	568	340	511	310	465
370	556	335	503	305	459	13	359	539	322	484	292	440
322	483	291	437	265	398	14	340	511	304	457	275	414
280	421	253	380	230	346	15	321	482	286	430	254	382
246	370	222	334	203	304	16	297	446	259	389	230	345
218	328	197	296	179	270	17	272	408	236	355	209	314
195	292	176	264	160	241	18	250	376	217	326	192	288
175	262	158	237	144	216	19	232	348	201	302	177	266
158	237	142	214	130	195	20	216	324	187	281	164	247
130	196	118	177	107	161	22	190	285	164	246	144	216
109	165	98.9	149	90.0	135	24	169	254	145	219	127	192
93.3	140	84.2	127	76.7	115	26	153	230	131	197	115	172
80.4	121	72.6	109	66.1	99.4	28	139	209	119	179	104	156
						30	128	192	109	164	95.2	143
						32	118	178	101	152	87.9	132
						34	110	166	93.9	141	81.6	123
						36	103	155	87.8	132	76.1	114
						38	96.9	146	82.4	124	71.4	107
						40	91.4	137	77.6	117	67.2	101
						42	86.6	130	73.4	110	63.5	95.5
						44	82.2	124	69.7	105	60.2	90.5
						46	78.2	118	66.3	99.6	57.3	86.1
						48	74.7	112	63.2	95.0	54.6	82.1
						50	71.4	107	60.4	90.8	52.2	78.4

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
876	1320	801	1200	738	1110
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
705	1060	645	967	594	891
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
256	385	232	348	211	317
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
86.3	130	78.6	118	72.0	108

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.07	15.5	5.05	15.0	5.02	14.7
Area, in.^{2}					
20.9		19.1		17.6	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1170	60.3	1070	54.8	984	50.1
r_y, in.					
1.70		1.69		1.68	
r_x/r_y					
4.41		4.43		4.45	

[c] Shape is slender for compression with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W18

F_y = 70 ksi
F_u = 90 ksi

W18×						Shape	W18×					
55^c		50^c		46^c		lb/ft	55		50		46	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
626	941	554	832	505	758	0	391	588	353	530	317	476
531	798	468	703	384	578	6	375	563	336	506	280	421
500	752	440	661	349	524	7	358	539	321	483	263	395
467	702	410	616	311	468	8	342	514	306	460	246	370
432	649	379	569	274	412	9	326	489	291	437	229	344
396	595	346	521	233	351	10	309	465	276	414	212	319
358	538	314	472	194	291	11	293	440	260	391	195	293
317	477	282	424	163	245	12	277	416	245	368	170	255
278	418	247	371	139	209	13	260	391	230	345	149	224
241	362	213	320	120	180	14	244	367	212	319	133	200
210	315	186	279	104	157	15	220	331	189	284	119	179
184	277	163	245	91.6	138	16	198	298	170	256	108	163
163	245	145	217	81.1	122	17	180	271	154	232	99.0	149
146	219	129	194	72.4	109	18	165	248	141	212	91.1	137
131	196	116	174	65.0	97.6	19	152	228	130	195	84.4	127
118	177	104	157	58.6	88.1	20	141	212	120	180	78.5	118
97.4	146	86.3	130			22	123	184	104	156	69.0	104
81.9	123	72.5	109			24	108	163	91.5	137	61.5	92.4
69.8	105	61.8	92.9			26	97.1	146	81.7	123	55.4	83.3
						28	87.9	132	73.8	111	50.5	75.9
						30	80.4	121	67.3	101	46.4	69.7
						32	74.0	111	61.8	92.9	42.9	64.5
						34	68.6	103	57.2	85.9	39.9	60.0
						36	63.9	96.1	53.2	79.9	37.4	56.2
						38	59.9	90.0	49.7	74.8	35.1	52.8
						40	56.3	84.6	46.7	70.2	33.1	49.8
						42	53.2	79.9	44.0	66.2	31.3	47.1
						44	50.3	75.7	41.7	62.6	29.7	44.7
						46	47.8	71.9	39.5	59.4	28.3	42.5
						48	45.6	68.5	37.6	56.6	27.0	40.6
						50	43.5	65.4	35.9	54.0	25.8	38.8

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
679	1020	616	926	566	851
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
547	820	496	744	456	683
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
198	296	179	268	182	274
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
64.6	97.1	58.0	87.2	40.9	61.4

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
4.99	14.2	4.93	13.8	3.85	11.1
Area, in.^{2}					
16.2		14.7		13.5	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
890	44.9	800	40.1	712	22.5
r_y, in.					
1.67		1.65		1.29	
r_x/r_y					
4.44		4.47		5.62	

^{c} Shape is slender for compression with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W18–W16

F_y = 70 ksi
F_u = 90 ksi

W18×				W16×		Shape		W18×				W16×	
40^c		35^c		100		lb/ft		40^v		35^v		100	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending		ASD	LRFD	ASD	LRFD	ASD	LRFD
425	638	360	540	1230	1850		0	274	412	232	349	692	1040
320	482	266	400	1130	1700		6	240	361	199	300	692	1040
290	435	239	359	1100	1650		7	225	338	185	279	692	1040
258	387	211	317	1060	1590		8	209	315	171	258	684	1030
226	339	183	276	1020	1530		9	194	292	158	237	669	1010
195	293	156	235	975	1470		10	179	268	144	216	654	983
164	247	132	199	928	1400		11	160	241	123	185	639	960
138	207	111	167	880	1320		12	138	208	106	159	624	938
118	177	94.7	142	830	1250		13	121	182	91.9	138	609	915
101	152	81.6	123	779	1170		14	107	161	81.1	122	594	892
88.3	133	71.1	107	728	1090		15	95.9	144	72.4	109	579	870
77.6	117	62.5	93.9	677	1020		16	86.6	130	65.2	97.9	564	847
68.7	103	55.4	83.2	627	942		17	78.9	119	59.2	88.9	549	825
61.3	92.2	49.4	74.2	577	868		18	72.4	109	54.1	81.3	534	802
55.0	82.7	44.3	66.6	530	796		19	66.8	100	49.8	74.8	519	780
49.7	74.6	40.0	60.1	483	727		20	62.0	93.2	46.1	69.2	504	757
				399	600		22	54.2	81.4	40.0	60.1	474	712
				336	504		24	48.0	72.2	35.3	53.1	443	667
				286	430		26	43.2	64.9	31.6	47.5	407	612
				247	371		28	39.2	58.9	28.6	43.0	370	557
				215	323		30	35.9	53.9	26.1	39.2	340	511
				189	284		32	33.1	49.8	24.0	36.1	314	472
				167	251		34	30.7	46.2	22.2	33.4	292	439
				149	224		36	28.7	43.1	20.7	31.1	273	410
				134	201		38	26.9	40.4	19.4	29.1	256	385
				121	182		40	25.3	38.1	18.2	27.4	242	363
							42	23.9	36.0	17.2	25.8	228	343
							44	22.7	34.1	16.3	24.4	217	326
							46	21.6	32.4	15.4	23.2	206	310
							48	20.6	30.9	14.7	22.1	197	296
							50	19.6	29.5	14.0	21.1	188	283

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
495	743	432	649	1230	1850
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
398	597	348	521	992	1490
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
142	213	129	194	278	418
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
34.9	52.5	28.2	42.3	192	288

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
3.79	10.7	3.64	10.2	7.49	25.0
Area, in.^{2}					
11.8		10.3		29.4	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
612	19.1	510	15.3	1490	186
r_y, in.					
1.27		1.22		2.51	
r_x/r_y					
5.68		5.77		2.83	

[c] Shape is slender for compression with F_y = 70 ksi.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W16

F_y = 70 ksi
F_u = 90 ksi

W16×						Shape	W16×					
89		77^c		67^c		lb/ft	89		77		67	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1100	1650	942	1420	795	1190	0	611	919	524	788	454	683
1010	1520	868	1310	734	1100	6	611	919	524	788	454	683
977	1470	842	1260	714	1070	7	611	919	524	788	454	683
943	1420	812	1220	690	1040	8	603	906	516	775	446	670
906	1360	779	1170	665	1000	9	588	884	502	755	433	651
866	1300	744	1120	638	959	10	574	863	489	734	421	633
824	1240	707	1060	609	916	11	559	841	475	714	409	614
780	1170	669	1010	579	869	12	545	819	462	694	396	595
735	1100	630	946	544	818	13	530	797	448	674	384	577
689	1040	590	887	510	766	14	516	775	435	653	371	558
643	967	550	827	475	714	15	501	753	421	633	359	539
598	898	510	767	440	662	16	487	731	408	613	346	520
552	830	471	708	406	611	17	472	709	394	592	334	502
508	764	433	651	373	561	18	457	688	381	572	321	483
466	700	396	595	341	513	19	443	666	367	552	309	464
424	637	360	541	310	465	20	428	644	354	532	296	445
350	527	297	447	256	384	22	399	600	326	490	262	395
294	442	250	376	215	323	24	365	549	287	432	230	346
251	377	213	320	183	275	26	328	493	257	386	205	308
216	325	184	276	158	237	28	298	447	232	349	184	277
188	283	160	240	138	207	30	272	409	212	318	167	252
166	249	141	211	121	182	32	251	377	194	292	153	230
147	220	124	187	107	161	34	233	350	180	270	141	213
131	197	111	167	95.5	144	36	217	327	167	252	131	197
117	176	99.7	150	85.7	129	38	204	306	157	235	122	184
106	159	89.9	135	77.4	116	40	192	288	147	221	115	172
						42	181	272	139	209	108	162
						44	172	258	131	197	102	153
						46	163	245	125	187	96.7	145
						48	156	234	119	178	91.9	138
						50	149	223	113	170	87.5	132

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1100	1650	947	1420	822	1230
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
884	1330	763	1140	662	992
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
247	370	210	315	180	270
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
168	253	144	216	124	186

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.43	23.4	7.37	21.9	7.34	20.8
Area, in.^{2}					
26.2		22.6		19.6	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1300	163	1110	138	954	119
r_y, in.					
2.49		2.47		2.46	
r_x/r_y					
2.83		2.83		2.83	

[c] Shape is slender for compression with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

W16

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W16× 57^c P_n/Ω_c	W16× 57^c $\phi_c P_n$	W16× 50^c P_n/Ω_c	W16× 50^c $\phi_c P_n$	W16× 45^c P_n/Ω_c	W16× 45^c $\phi_c P_n$	Shape lb/ft	W16× 57 M_{nx}/Ω_b	W16× 57 $\phi_b M_{nx}$	W16× 50 M_{nx}/Ω_b	W16× 50 $\phi_b M_{nx}$	W16× 45 M_{nx}/Ω_b	W16× 45 $\phi_b M_{nx}$
Available Compressive Strength, kips						Design	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
689	1040	584	878	516	775	0	367	551	321	483	287	432
572	860	486	731	427	643	6	349	525	304	457	271	407
531	798	455	684	399	600	7	335	503	291	437	258	388
487	732	422	634	369	555	8	320	481	277	416	245	369
442	664	384	577	338	508	9	306	459	263	396	233	350
396	595	344	517	306	460	10	291	437	250	375	220	331
351	527	304	457	270	406	11	277	416	236	355	207	312
307	462	266	400	236	354	12	262	394	222	334	195	292
266	399	230	345	202	304	13	248	372	209	314	182	273
229	344	198	297	175	262	14	233	350	193	290	164	246
200	300	172	259	152	229	15	215	323	173	260	147	220
175	264	152	228	134	201	16	195	293	157	236	133	199
155	233	134	202	118	178	17	179	269	143	216	121	181
139	208	120	180	106	159	18	165	248	132	198	111	166
124	187	107	162	94.8	142	19	153	230	122	183	102	154
112	169	97.0	146	85.5	129	20	143	215	114	171	94.9	143
92.8	139	80.1	120	70.7	106	22	126	189	99.6	150	82.9	125
77.9	117	67.3	101	59.4	89.3	24	112	169	88.6	133	73.5	111
66.4	99.8	57.4	86.2	50.6	76.1	26	102	153	79.9	120	66.1	99.3
						28	92.9	140	72.7	109	60.0	90.2
						30	85.5	128	66.8	100	55.0	82.6
						32	79.2	119	61.7	92.7	50.7	76.2
						34	73.8	111	57.4	86.3	47.1	70.8
						36	69.1	104	53.7	80.6	43.9	66.0
						38	65.0	97.7	50.4	75.7	41.2	61.9
						40	61.3	92.2	47.5	71.4	38.8	58.3
						42	58.1	87.3	44.9	67.5	36.7	55.1
						44	55.2	83.0	42.6	64.1	34.8	52.3
						46	52.6	79.0	40.6	61.0	33.1	49.7
						48	50.2	75.4	38.7	58.2	31.5	47.4
						50	48.0	72.2	37.0	55.6	30.1	45.2

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
704	1060	616	926	557	838
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
567	851	496	744	449	673
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
197	296	173	260	156	233
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
66.0	99.2	56.9	85.6	50.6	76.1

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
4.78	14.5	4.75	13.8	4.69	13.3
Area, in.^{2}					
16.8		14.7		13.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
758	43.1	659	37.2	586	32.8
r_y, in.					
1.60		1.59		1.57	
r_x/r_y					
4.20		4.20		4.24	

[c] Shape is slender for compression with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W16

F_y = 70 ksi
F_u = 90 ksi

W16×						Shape	W16×					
40^c		36^c		31^c		lb/ft	40^v		$36^{f,v}$		31^v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
444	668	392	589	327	492	0	255	383	221	332	189	284
368	553	321	483	235	353	6	240	360	208	312	159	239
344	516	299	449	208	313	7	228	343	197	296	147	222
318	477	275	414	182	273	8	216	325	187	280	136	204
291	437	251	377	155	234	9	205	307	176	264	124	186
263	396	226	339	130	196	10	193	290	165	248	110	165
236	355	201	302	108	162	11	181	272	154	232	93.1	140
209	314	177	266	90.6	136	12	170	255	144	216	80.4	121
180	270	151	227	77.2	116	13	157	236	129	194	70.5	106
155	233	130	196	66.6	100	14	139	209	114	171	62.6	94.0
135	203	114	171	58.0	87.1	15	124	186	101	152	56.1	84.4
119	178	99.9	150	51.0	76.6	16	112	168	90.8	136	50.8	76.4
105	158	88.5	133	45.1	67.8	17	101	152	82.2	124	46.4	69.7
93.7	141	78.9	119	40.3	60.5	18	92.8	139	75.0	113	42.6	64.0
84.1	126	70.8	106	36.1	54.3	19	85.4	128	68.8	103	39.4	59.2
75.9	114	63.9	96.1			20	79.0	119	63.6	95.5	36.6	55.0
62.7	94.3	52.8	79.4			22	68.7	103	55.0	82.6	32.0	48.2
52.7	79.2	44.4	66.7			24	60.7	91.2	48.4	72.7	28.5	42.8
44.9	67.5					26	54.3	81.7	43.1	64.8	25.6	38.5
						28	49.2	73.9	38.9	58.4	23.3	35.1
						30	44.9	67.5	35.4	53.2	21.4	32.1
						32	41.3	62.1	32.5	48.8	19.8	29.7
						34	38.3	57.5	30.0	45.1	18.4	27.6
						36	35.7	53.6	27.9	41.9	17.2	25.8
						38	33.4	50.2	26.0	39.1	16.1	24.2
						40	31.4	47.2	24.4	36.7	15.2	22.8
						42	29.6	44.5	23.0	34.6	14.4	21.6
						44	28.0	42.1	21.8	32.7	13.6	20.5
						46	26.6	40.0	20.6	31.0	13.0	19.5
						48	25.4	38.1	19.6	29.5	12.4	18.6
						50	24.2	36.4	18.7	28.1	11.8	17.7

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
495	743	444	668	383	575
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
398	597	358	537	308	462
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
123	184	118	177	110	165
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
44.4	66.7	37.1	55.8	24.6	36.9

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
4.69	13.0	4.78	12.5	3.49	9.72
Area, in.^{2}					
11.8		10.6		9.13	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
518	28.9	448	24.5	375	12.4
r_y, in.					
1.57		1.52		1.17	
r_x/r_y					
4.22		4.28		5.48	

[c] Shape is slender for compression with F_y = 70 ksi.

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W16–W14

F_y = 70 ksi
F_u = 90 ksi

Shape	W16×		W14×				Shape		W16×		W14×			
lb/ft	26^c		873^h		808^h		lb/ft		$26^{f,v}$		873^h		808^h	
	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending		ASD	LRFD	ASD	LRFD	ASD	LRFD
	264	397	10800	16200	9980	15000		0	153	230	7090	10700	6390	9610
	184	277	10500	15800	9750	14700		6	127	191	7090	10700	6390	9610
	162	244	10500	15700	9670	14500		7	117	176	7090	10700	6390	9610
	140	210	10400	15600	9580	14400		8	107	160	7090	10700	6390	9610
	118	178	10200	15400	9480	14200		9	96.5	145	7090	10700	6390	9610
	99.1	149	10100	15200	9370	14100		10	81.5	122	7090	10700	6390	9610
	83.1	125	10000	15000	9240	13900		11	68.8	103	7090	10700	6390	9610
	69.8	105	9860	14800	9110	13700		12	59.1	88.8	7090	10700	6390	9610
	59.5	89.4	9710	14600	8970	13500		13	51.5	77.5	7090	10700	6390	9610
	51.3	77.1	9550	14400	8810	13200		14	45.5	68.4	7090	10700	6390	9610
	44.7	67.2	9380	14100	8650	13000		15	40.6	61.1	7090	10600	6380	9590
	39.3	59.0	9210	13800	8490	12800		16	36.6	55.0	7070	10600	6370	9570
	34.8	52.3	9020	13600	8310	12500		17	33.2	50.0	7050	10600	6350	9550
	31.0	46.6	8830	13300	8130	12200		18	30.4	45.7	7040	10600	6340	9530
			8630	13000	7940	11900		19	28.0	42.1	7020	10600	6330	9510
			8430	12700	7750	11600		20	25.9	39.0	7010	10500	6310	9490
			8000	12000	7350	11000		22	22.6	33.9	6980	10500	6280	9440
			7560	11400	6930	10400		24	19.9	30.0	6950	10400	6250	9400
			7110	10700	6510	9780		26	17.8	26.8	6920	10400	6220	9360
			6660	10000	6080	9140		28	16.2	24.3	6890	10400	6200	9310
			6200	9320	5650	8490		30	14.8	22.2	6860	10300	6170	9270
			5740	8630	5220	7850		32	13.6	20.4	6830	10300	6140	9220
			5300	7960	4810	7220		34	12.6	18.9	6800	10200	6110	9180
			4860	7310	4400	6610		36	11.7	17.6	6770	10200	6080	9140
			4440	6670	4010	6020		38	11.0	16.5	6740	10100	6050	9090
			4030	6050	3620	5440		40	10.3	15.5	6710	10100	6020	9050
			3650	5490	3290	4940		42	9.74	14.6	6680	10000	5990	9010
			3330	5000	2990	4500		44	9.22	13.9	6640	9990	5960	8960
			3040	4570	2740	4120		46	8.76	13.2	6610	9940	5930	8920
			2800	4200	2520	3780		48	8.34	12.5	6580	9900	5910	8880
			2580	3870	2320	3480		50	7.96	12.0	6550	9850	5880	8830

Properties

	W16×26		W14×873		W14×808	
Available Strength in Tensile Yielding, kips						
	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	322	484	10800	16200	9980	15000
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips						
	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	259	389	8670	13000	8030	12000
Available Strength in Shear, kips						
	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	89.9	135	2600	3910	2390	3580
Available Strength in Flexure about Y-Y Axis, kip-ft						
	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	18.9	28.5	3560	5360	3250	4880

	W16×26		W14×873		W14×808	
Limiting Unbraced Lengths, ft						
	L_p	L_r	L_p	L_r	L_p	L_r
	3.45	9.25	14.6	235	14.4	221
Area, in.^{2}						
	7.68		257		238	
Moment of Inertia, in.^{4}						
	I_x	I_y	I_x	I_y	I_x	I_y
	301	9.59	18100	6170	15900	5550
r_y, in.						
	1.12		4.90		4.83	
r_x/r_y						
	5.59		1.71		1.69	

[c] Shape is slender for compression with F_y = 70 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.
[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.
Note: Heavy line indicates L_c/r equal to or greater than 200.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W14

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W14×						Shape	W14×					
730^h		665^h		605^h		lb/ft	730^h		665^h		605^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
9010	13500	8220	12300	7460	11200	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending 0	5800	8720	5170	7770	4610	6930
8800	13200	8010	12000	7270	10900	6	5800	8720	5170	7770	4610	6930
8720	13100	7940	11900	7210	10800	7	5800	8720	5170	7770	4610	6930
8630	13000	7860	11800	7130	10700	8	5800	8720	5170	7770	4610	6930
8540	12800	7770	11700	7040	10600	9	5800	8720	5170	7770	4610	6930
8430	12700	7670	11500	6950	10400	10	5800	8720	5170	7770	4610	6930
8310	12500	7560	11400	6850	10300	11	5800	8720	5170	7770	4610	6930
8180	12300	7440	11200	6730	10100	12	5800	8720	5170	7770	4610	6930
8050	12100	7310	11000	6620	9940	13	5800	8720	5170	7770	4610	6930
7900	11900	7180	10800	6490	9750	14	5800	8720	5170	7770	4610	6920
7750	11600	7030	10600	6360	9550	15	5780	8690	5150	7740	4590	6900
7590	11400	6880	10300	6220	9350	16	5770	8670	5140	7720	4580	6880
7430	11200	6730	10100	6070	9130	17	5750	8650	5120	7700	4560	6860
7250	10900	6570	9870	5920	8900	18	5740	8630	5110	7680	4550	6840
7080	10600	6400	9620	5770	8670	19	5730	8610	5100	7660	4540	6820
6890	10400	6230	9370	5610	8430	20	5710	8580	5080	7640	4520	6800
6520	9790	5880	8840	5290	7950	22	5680	8540	5050	7600	4500	6760
6130	9210	5520	8300	4950	7440	24	5650	8500	5030	7550	4470	6720
5730	8610	5150	7740	4610	6930	26	5620	8450	5000	7510	4440	6680
5330	8010	4780	7190	4270	6420	28	5590	8410	4970	7470	4410	6640
4930	7410	4410	6630	3930	5910	30	5560	8360	4940	7430	4390	6590
4540	6820	4050	6090	3600	5410	32	5540	8320	4910	7380	4360	6550
4150	6240	3700	5560	3280	4920	34	5510	8280	4880	7340	4330	6510
3780	5680	3360	5050	2970	4460	36	5480	8230	4860	7300	4310	6470
3420	5140	3020	4550	2660	4000	38	5450	8190	4830	7260	4280	6430
3090	4640	2730	4100	2400	3610	40	5420	8140	4800	7210	4250	6390
2800	4210	2480	3720	2180	3280	42	5390	8100	4770	7170	4220	6350
2550	3830	2260	3390	1990	2990	44	5360	8060	4740	7130	4200	6310
2330	3510	2060	3100	1820	2730	46	5330	8010	4720	7090	4170	6270
2140	3220	1900	2850	1670	2510	48	5300	7970	4690	7040	4140	6230
1970	2970	1750	2630	1540	2310	50	5270	7920	4660	7000	4120	6190

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
9010	13500	8220	12300	7460	11200
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
7260	10900	6620	9920	6010	9010
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1930	2890	1710	2570	1520	2280
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
2850	4280	2550	3830	2280	3420

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
14.0	197	13.8	181	13.6	166
Area, in.^{2}					
215		196		178	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
14300	4720	12400	4170	10800	3680
r_y, in.					
4.69		4.62		4.55	
r_x/r_y					
1.74		1.73		1.71	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

W14

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W14×						Shape	W14×					
550^h		500^h		455^h		lb/ft	550^h		500^h		455^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
6790	10200	6160	9260	5620	8440	0	4120	6200	3670	5510	3270	4910
6610	9940	6000	9010	5460	8210	6	4120	6200	3670	5510	3270	4910
6550	9850	5940	8930	5410	8130	7	4120	6200	3670	5510	3270	4910
6480	9740	5870	8830	5350	8040	8	4120	6200	3670	5510	3270	4910
6400	9620	5800	8710	5280	7930	9	4120	6200	3670	5510	3270	4910
6310	9490	5720	8590	5200	7820	10	4120	6200	3670	5510	3270	4910
6220	9340	5630	8460	5120	7690	11	4120	6200	3670	5510	3270	4910
6110	9190	5530	8310	5030	7560	12	4120	6200	3670	5510	3270	4910
6000	9020	5430	8160	4930	7410	13	4120	6200	3670	5510	3270	4910
5880	8840	5320	7990	4830	7260	14	4110	6180	3660	5500	3260	4900
5760	8660	5200	7820	4730	7100	15	4100	6160	3640	5480	3250	4880
5630	8460	5080	7640	4610	6930	16	4090	6140	3630	5460	3230	4860
5500	8260	4960	7450	4500	6760	17	4070	6120	3620	5440	3220	4840
5360	8050	4830	7260	4380	6580	18	4060	6100	3610	5420	3210	4820
5220	7840	4700	7060	4260	6400	19	4050	6080	3590	5400	3200	4800
5070	7620	4560	6860	4130	6210	20	4030	6060	3580	5380	3180	4790
4770	7160	4280	6440	3870	5820	22	4010	6020	3560	5340	3160	4750
4460	6700	4000	6010	3610	5420	24	3980	5980	3530	5310	3130	4710
4140	6230	3710	5570	3340	5020	26	3950	5940	3500	5270	3110	4670
3830	5750	3420	5140	3080	4620	28	3930	5900	3480	5230	3090	4640
3520	5290	3130	4710	2810	4230	30	3900	5860	3450	5190	3060	4600
3210	4830	2860	4290	2560	3840	32	3880	5820	3430	5150	3040	4560
2920	4380	2590	3890	2310	3470	34	3850	5780	3400	5110	3010	4530
2630	3950	2320	3490	2070	3110	36	3820	5740	3380	5070	2990	4490
2360	3550	2090	3130	1860	2790	38	3800	5700	3350	5040	2960	4450
2130	3200	1880	2830	1680	2520	40	3770	5660	3330	5000	2940	4410
1930	2900	1710	2570	1520	2290	42	3740	5620	3300	4960	2910	4380
1760	2650	1560	2340	1390	2080	44	3720	5580	3270	4920	2890	4340
1610	2420	1420	2140	1270	1910	46	3690	5550	3250	4880	2860	4300
1480	2220	1310	1960	1160	1750	48	3660	5510	3220	4840	2840	4270
1360	2050	1200	1810	1070	1610	50	3640	5470	3200	4810	2810	4230

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
6790	10200	6160	9260	5620	8440
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
5470	8200	4960	7440	4520	6780
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1350	2020	1200	1800	1070	1610
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
2040	3060	1820	2740	1630	2460

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
13.4	153	13.2	140	13.1	128
Area, in.^{2}					
162		147		134	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
9430	3250	8210	2880	7190	2560
r_y, in.					
4.49		4.43		4.38	
r_x/r_y					
1.70		1.69		1.67	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

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W14

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W14×						Shape	W14×					
426^h		398^h		370^h		lb/ft	426^h		398^h		370^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
5240	7870	4900	7370	4570	6870	0	3040	4560	2800	4210	2570	3860
5090	7660	4770	7160	4440	6670	6	3040	4560	2800	4210	2570	3860
5040	7580	4720	7090	4390	6600	7	3040	4560	2800	4210	2570	3860
4980	7490	4660	7010	4340	6520	8	3040	4560	2800	4210	2570	3860
4920	7390	4600	6910	4280	6430	9	3040	4560	2800	4210	2570	3860
4850	7280	4530	6810	4210	6330	10	3040	4560	2800	4210	2570	3860
4770	7160	4460	6700	4140	6230	11	3040	4560	2800	4210	2570	3860
4680	7040	4370	6580	4070	6110	12	3040	4560	2800	4210	2570	3860
4590	6900	4290	6450	3990	5990	13	3030	4560	2800	4200	2570	3860
4490	6760	4200	6310	3900	5860	14	3020	4540	2780	4180	2560	3840
4390	6600	4100	6170	3810	5720	15	3010	4520	2770	4170	2540	3820
4290	6450	4000	6020	3710	5580	16	3000	4510	2760	4150	2530	3810
4180	6280	3900	5860	3620	5440	17	2990	4490	2750	4130	2520	3790
4070	6110	3790	5700	3520	5280	18	2970	4470	2740	4110	2510	3770
3950	5940	3680	5540	3410	5130	19	2960	4450	2720	4100	2500	3750
3830	5760	3570	5370	3310	4970	20	2950	4430	2710	4080	2490	3740
3590	5390	3340	5020	3090	4640	22	2930	4400	2690	4040	2460	3700
3340	5020	3110	4670	2870	4310	24	2900	4360	2670	4010	2440	3670
3090	4640	2870	4310	2650	3980	26	2880	4320	2640	3970	2420	3630
2840	4260	2630	3960	2420	3640	28	2850	4290	2620	3930	2390	3600
2590	3890	2400	3610	2210	3320	30	2830	4250	2590	3900	2370	3560
2350	3530	2180	3270	2000	3000	32	2800	4210	2570	3860	2350	3530
2120	3190	1960	2950	1790	2700	34	2780	4180	2550	3830	2320	3490
1900	2850	1750	2630	1600	2410	36	2750	4140	2520	3790	2300	3460
1700	2560	1570	2360	1440	2160	38	2730	4100	2500	3750	2280	3420
1540	2310	1420	2130	1300	1950	40	2710	4070	2470	3720	2250	3390
1390	2090	1290	1930	1180	1770	42	2680	4030	2450	3680	2230	3350
1270	1910	1170	1760	1070	1610	44	2660	3990	2430	3650	2210	3320
1160	1750	1070	1610	980	1470	46	2630	3960	2400	3610	2180	3280
1070	1600	985	1480	900	1350	48	2610	3920	2380	3580	2160	3250
983	1480	907	1360	830	1250	50	2580	3880	2350	3540	2140	3210

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
5240	7880	4900	7370	4570	6870
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4220	6330	3950	5920	3680	5520
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
984	1480	907	1360	832	1250
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
1520	2280	1400	2110	1290	1940

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
13.0	120	12.9	113	12.7	106
Area, in.^{2}					
125		117		109	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
6600	2360	6000	2170	5440	1990
r_y, in.					
4.34		4.31		4.27	
r_x/r_y					
1.67		1.66		1.66	

^{h} Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W14

F_y = 70 ksi
F_u = 90 ksi

W14×						Shape	W14×					
342^h		311^h		283^h		lb/ft	342^h		311^h		283^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
4230	6360	3830	5760	3490	5250	0	2350	3530	2110	3170	1890	2850
4110	6180	3720	5590	3390	5090	6	2350	3530	2110	3170	1890	2850
4070	6110	3680	5530	3350	5030	7	2350	3530	2110	3170	1890	2850
4020	6040	3630	5460	3310	4970	8	2350	3530	2110	3170	1890	2850
3960	5950	3580	5380	3260	4900	9	2350	3530	2110	3170	1890	2850
3900	5860	3520	5300	3210	4820	10	2350	3530	2110	3170	1890	2850
3830	5760	3460	5200	3150	4740	11	2350	3530	2110	3170	1890	2850
3760	5650	3400	5110	3090	4640	12	2350	3530	2110	3170	1890	2850
3690	5540	3330	5000	3030	4550	13	2340	3520	2100	3160	1890	2840
3610	5420	3250	4890	2960	4440	14	2330	3500	2090	3140	1880	2820
3520	5290	3170	4770	2890	4340	15	2320	3490	2080	3120	1860	2800
3430	5160	3090	4650	2810	4220	16	2310	3470	2070	3110	1850	2790
3340	5020	3010	4520	2730	4110	17	2300	3450	2060	3090	1840	2770
3250	4880	2920	4390	2650	3990	18	2290	3440	2040	3070	1830	2750
3150	4730	2830	4260	2570	3860	19	2270	3420	2030	3060	1820	2740
3050	4580	2740	4120	2490	3740	20	2260	3400	2020	3040	1810	2720
2850	4280	2560	3840	2320	3480	22	2240	3370	2000	3010	1790	2690
2640	3970	2370	3560	2140	3220	24	2220	3330	1980	2970	1770	2650
2430	3660	2180	3270	1970	2960	26	2190	3300	1960	2940	1740	2620
2230	3350	1990	2990	1800	2700	28	2170	3260	1930	2900	1720	2590
2020	3040	1810	2710	1630	2450	30	2150	3230	1910	2870	1700	2550
1830	2750	1630	2450	1470	2200	32	2130	3200	1890	2840	1680	2520
1640	2460	1460	2190	1310	1970	34	2100	3160	1870	2800	1650	2490
1460	2200	1300	1950	1170	1750	36	2080	3130	1840	2770	1630	2450
1310	1970	1170	1750	1050	1570	38	2060	3090	1820	2740	1610	2420
1180	1780	1050	1580	945	1420	40	2030	3060	1800	2700	1590	2390
1070	1610	954	1430	857	1290	42	2010	3020	1780	2670	1570	2350
979	1470	869	1310	781	1170	44	1990	2990	1750	2630	1540	2320
896	1350	795	1200	715	1070	46	1970	2960	1730	2600	1520	2290
823	1240	730	1100	656	986	48	1940	2920	1710	2570	1500	2250
758	1140	673	1010	605	909	50	1920	2890	1690	2530	1480	2220

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4230	6360	3830	5760	3490	5250
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3410	5110	3080	4630	2810	4220
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
755	1130	675	1010	603	905
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
1180	1770	1060	1600	957	1440

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.7	98.7	12.5	89.9	12.4	82.1
Area, in.^{2}					
101		91.4		83.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
4900	1810	4330	1610	3840	1440
r_y, in.					
4.24		4.20		4.17	
r_x/r_y					
1.65		1.64		1.63	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W14

F_y = 70 ksi
F_u = 90 ksi

W14×						Shape		W14×					
257		233		211		lb/ft		257		233		211	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
3170	4760	2870	4320	2600	3910	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	1700	2560	1520	2290	1360	2050
3070	4620	2780	4180	2520	3780		6	1700	2560	1520	2290	1360	2050
3040	4570	2750	4130	2490	3740		7	1700	2560	1520	2290	1360	2050
3000	4510	2710	4080	2450	3690		8	1700	2560	1520	2290	1360	2050
2950	4440	2670	4020	2420	3630		9	1700	2560	1520	2290	1360	2050
2910	4370	2630	3950	2380	3570		10	1700	2560	1520	2290	1360	2050
2850	4290	2580	3880	2330	3510		11	1700	2560	1520	2290	1360	2050
2800	4210	2530	3800	2290	3440		12	1700	2560	1520	2290	1360	2050
2740	4120	2480	3720	2240	3360		13	1690	2550	1510	2280	1350	2030
2680	4020	2420	3630	2180	3280		14	1680	2530	1500	2260	1340	2020
2610	3920	2360	3540	2130	3200		15	1670	2510	1490	2240	1330	2000
2540	3820	2290	3450	2070	3110		16	1660	2500	1480	2230	1320	1990
2470	3710	2230	3350	2010	3020		17	1650	2480	1470	2210	1310	1970
2390	3600	2160	3250	1950	2930		18	1640	2460	1460	2200	1300	1950
2320	3490	2090	3140	1880	2830		19	1630	2450	1450	2180	1290	1940
2240	3370	2020	3040	1820	2740		20	1620	2430	1440	2160	1280	1920
2090	3130	1880	2820	1690	2540		22	1600	2400	1420	2130	1260	1890
1930	2900	1730	2600	1560	2340		24	1570	2360	1400	2100	1240	1860
1770	2660	1590	2390	1420	2140		26	1550	2330	1370	2070	1220	1830
1610	2420	1440	2170	1290	1940		28	1530	2300	1350	2030	1190	1790
1460	2190	1300	1960	1170	1750		30	1510	2270	1330	2000	1170	1760
1310	1970	1170	1760	1040	1570		32	1490	2230	1310	1970	1150	1730
1160	1750	1040	1560	927	1390		34	1460	2200	1290	1940	1130	1700
1040	1560	927	1390	827	1240		36	1440	2170	1270	1900	1110	1670
932	1400	832	1250	742	1120		38	1420	2130	1250	1870	1090	1640
841	1260	751	1130	670	1010		40	1400	2100	1220	1840	1070	1600
763	1150	681	1020	608	913		42	1380	2070	1200	1810	1050	1570
695	1040	621	933	554	832		44	1350	2040	1180	1770	1020	1540
636	956	568	854	507	761		46	1330	2000	1160	1740	1000	1510
584	878	522	784	465	699		48	1310	1970	1140	1710	982	1480
538	809	481	723	429	644		50	1290	1940	1120	1680	960	1440

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3170	4760	2870	4320	2600	3910
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2550	3830	2310	3470	2090	3140
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
542	813	479	719	431	646
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
859	1290	772	1160	692	1040

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.3	75.1	12.2	68.4	12.1	62.6
Area, in.^{2}					
75.6		68.5		62.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
3400	1290	3010	1150	2660	1030
r_y, in.					
4.13		4.10		4.07	
r_x/r_y					
1.62		1.62		1.61	

Note: Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W14

F_y = 70 ksi
F_u = 90 ksi

W14×						Shape		W14×					
193		176		159		lb/ft		193		176		159	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
2380	3580	2170	3260	1960	2940	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	1240	1860	1120	1680	1000	1510
2310	3460	2100	3160	1890	2850		6	1240	1860	1120	1680	1000	1510
2280	3420	2080	3120	1870	2810		7	1240	1860	1120	1680	1000	1510
2250	3380	2050	3080	1850	2770		8	1240	1860	1120	1680	1000	1510
2210	3330	2020	3030	1820	2730		9	1240	1860	1120	1680	1000	1510
2180	3270	1980	2980	1790	2680		10	1240	1860	1120	1680	1000	1510
2140	3210	1940	2920	1750	2630		11	1240	1860	1120	1680	1000	1510
2090	3140	1900	2860	1710	2580		12	1240	1860	1120	1680	1000	1510
2050	3070	1860	2800	1680	2520		13	1230	1850	1110	1660	992	1490
2000	3000	1820	2730	1630	2460		14	1220	1830	1100	1650	981	1470
1940	2920	1770	2660	1590	2390		15	1210	1820	1090	1630	971	1460
1890	2840	1720	2580	1550	2320		16	1200	1800	1080	1620	961	1440
1840	2760	1670	2510	1500	2250		17	1190	1790	1070	1600	950	1430
1780	2670	1620	2430	1450	2180		18	1180	1770	1060	1590	940	1410
1720	2590	1560	2350	1400	2110		19	1170	1750	1040	1570	930	1400
1660	2500	1510	2270	1350	2040		20	1160	1740	1030	1550	920	1380
1540	2320	1400	2100	1250	1880		22	1140	1710	1010	1520	899	1350
1420	2130	1280	1930	1150	1730		24	1110	1670	993	1490	878	1320
1300	1950	1170	1760	1050	1580		26	1090	1640	972	1460	858	1290
1180	1770	1060	1600	951	1430		28	1070	1610	951	1430	837	1260
1060	1590	955	1440	854	1280		30	1050	1580	930	1400	817	1230
949	1430	853	1280	762	1140		32	1030	1550	909	1370	796	1200
841	1260	756	1140	675	1010		34	1010	1520	889	1340	775	1170
750	1130	674	1010	602	905		36	987	1480	868	1300	755	1130
673	1010	605	909	540	812		38	966	1450	847	1270	734	1100
608	914	546	821	487	733		40	945	1420	826	1240	714	1070
551	829	495	744	442	665		42	924	1390	805	1210	693	1040
502	755	451	678	403	605		44	902	1360	785	1180	672	1010
460	691	413	621	369	554		46	881	1320	764	1150	652	980
422	634	379	570	339	509		48	860	1290	743	1120	631	949
389	585	350	525	312	469		50	839	1260	722	1090	607	912

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2380	3580	2170	3260	1960	2940
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1920	2880	1750	2620	1580	2360
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
386	579	353	530	313	469
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
629	945	569	856	510	767

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.1	57.6	12.0	53.4	11.9	49.0
Area, in.^{2}					
56.8		51.8		46.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
2400	931	2140	838	1900	748
r_y, in.					
4.05		4.02		4.00	
r_x/r_y					
1.60		1.60		1.60	

Note: Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W14

F_y = 70 ksi
F_u = 90 ksi

W14×						Shape		W14×					
145		132		120		lb/ft		145		132		120^f	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
1790	2690	1630	2440	1480	2220	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	908	1370	817	1230	739	1110
1730	2600	1570	2350	1420	2140		6	908	1370	817	1230	739	1110
1710	2570	1550	2320	1410	2110		7	908	1370	817	1230	739	1110
1690	2530	1520	2290	1380	2080		8	908	1370	817	1230	739	1110
1660	2490	1490	2250	1360	2040		9	908	1370	817	1230	739	1110
1630	2450	1470	2200	1330	2000		10	908	1370	817	1230	739	1110
1600	2400	1430	2150	1300	1960		11	908	1370	817	1230	739	1110
1570	2350	1400	2100	1270	1910		12	907	1360	810	1220	732	1100
1530	2300	1360	2050	1240	1860		13	897	1350	799	1200	722	1090
1490	2240	1330	1990	1200	1810		14	887	1330	789	1190	712	1070
1450	2180	1290	1930	1170	1750		15	877	1320	779	1170	702	1060
1410	2120	1250	1870	1130	1700		16	867	1300	769	1160	693	1040
1370	2060	1200	1810	1090	1640		17	857	1290	759	1140	683	1030
1320	1990	1160	1740	1050	1580		18	846	1270	749	1130	673	1010
1280	1920	1120	1680	1010	1520		19	836	1260	739	1110	663	996
1230	1850	1070	1610	971	1460		20	826	1240	729	1100	653	981
1140	1710	982	1480	888	1340		22	806	1210	709	1060	633	952
1050	1570	892	1340	806	1210		24	786	1180	688	1030	613	922
954	1430	804	1210	726	1090		26	766	1150	668	1000	593	892
863	1300	718	1080	648	973		28	745	1120	648	974	574	862
775	1160	636	956	573	861		30	725	1090	628	944	554	832
689	1040	559	840	503	756		32	705	1060	608	913	534	803
611	918	495	744	446	670		34	685	1030	587	883	514	773
545	819	442	664	398	598		36	665	999	567	853	494	743
489	735	397	596	357	536		38	644	969	547	822	474	713
441	663	358	538	322	484		40	624	938	527	792	450	677
400	602	325	488	292	439		42	604	908	504	758	424	638
365	548	296	445	266	400		44	584	878	478	718	402	604
334	501	271	407	244	366		46	562	845	454	682	381	573
306	461	249	374	224	336		48	535	804	432	650	363	545
282	424	229	344	206	310		50	511	768	413	620	346	520

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1790	2690	1630	2440	1480	2220
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1440	2160	1310	1960	1190	1790
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
282	423	265	398	240	359
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
465	698	395	593	355	534

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.9	45.7	11.2	41.6	11.3	39.0
Area, in.^{2}					
42.7		38.8		35.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1710	677	1530	548	1380	495
r_y, in.					
3.98		3.76		3.74	
r_x/r_y					
1.59		1.67		1.67	

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

Note: Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W14

F_y = 70 ksi
F_u = 90 ksi

W14×						Shape	W14×					
109		99		90		lb/ft	109^f		99^f		90^f	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1340	2020	1220	1830	1110	1670	0	656	986	576	866	510	766
1290	1940	1170	1760	1070	1610	6	656	986	576	866	510	766
1270	1910	1160	1740	1050	1580	7	656	986	576	866	510	766
1250	1880	1140	1710	1040	1560	8	656	986	576	866	510	766
1230	1850	1120	1680	1020	1530	9	656	986	576	866	510	766
1210	1810	1100	1650	997	1500	10	656	986	576	866	510	766
1180	1770	1070	1610	975	1470	11	656	986	576	866	510	766
1150	1730	1050	1570	951	1430	12	656	986	576	866	510	766
1120	1690	1020	1530	926	1390	13	653	981	576	866	510	766
1090	1640	989	1490	899	1350	14	643	966	576	866	510	766
1060	1590	959	1440	872	1310	15	633	952	568	853	510	766
1020	1540	927	1390	843	1270	16	624	938	559	839	504	757
988	1480	895	1350	814	1220	17	614	923	549	825	495	743
952	1430	862	1300	784	1180	18	604	909	540	812	486	730
915	1380	829	1250	753	1130	19	595	894	531	798	477	716
878	1320	795	1190	722	1090	20	585	880	521	784	467	703
803	1210	726	1090	660	991	22	566	851	503	756	449	675
729	1100	658	989	597	898	24	547	822	484	728	431	648
655	985	591	889	536	806	26	527	793	466	700	413	621
585	879	527	792	478	718	28	508	764	447	672	395	594
516	776	465	698	421	632	30	489	735	428	644	377	567
454	682	408	614	370	556	32	470	706	410	616	359	540
402	604	362	544	328	492	34	450	677	391	588	336	505
359	539	323	485	292	439	36	431	648	367	551	310	467
322	484	290	435	262	394	38	406	611	342	513	289	434
290	437	261	393	237	356	40	381	573	320	481	270	406
263	396	237	356	215	323	42	359	539	301	452	253	381
240	361	216	325	196	294	44	339	510	284	427	239	359
220	330	198	297	179	269	46	321	483	269	404	226	339
202	303	181	273	164	247	48	306	459	255	384	214	322
186	279	167	251	151	228	50	291	438	243	365	204	306

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1340	2020	1220	1830	1110	1670
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1080	1620	982	1470	894	1340
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
210	315	193	289	172	259
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
313	471	272	409	236	355

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.7	36.8	14.1	34.8	15.3	33.0
Area, in.^{2}					
32.0		29.1		26.5	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1240	447	1110	402	999	362
r_y, in.					
3.73		3.71		3.70	
r_x/r_y					
1.67		1.66		1.66	

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

Note: Confirm ASTM A913 material availability before specifying.

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W14

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W14× 82		W14× 74		W14× 68		Shape lb/ft	W14× 82		W14× 74		W14× 68	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1010	1510	914	1370	838	1260	0	486	730	440	662	402	604
923	1390	838	1260	768	1150	6	486	730	440	662	402	604
895	1340	813	1220	744	1120	7	486	730	440	662	402	604
863	1300	784	1180	717	1080	8	479	720	434	652	395	594
828	1250	753	1130	688	1030	9	469	705	424	637	386	579
792	1190	719	1080	657	988	10	459	689	414	622	376	565
753	1130	684	1030	624	938	11	448	674	404	607	366	550
712	1070	647	973	590	887	12	438	658	394	592	356	535
671	1010	609	916	555	835	13	427	642	384	576	347	521
629	945	571	859	520	782	14	417	627	373	561	337	506
587	882	533	801	485	728	15	407	611	363	546	327	491
545	819	495	744	449	675	16	396	595	353	531	317	477
503	756	457	687	415	623	17	386	580	343	516	307	462
463	696	420	632	381	572	18	375	564	333	500	298	448
423	637	385	578	348	523	19	365	549	323	485	288	433
385	579	350	526	316	475	20	355	533	313	470	278	418
318	478	289	435	261	392	22	334	502	292	440	259	389
267	402	243	365	219	330	24	313	470	271	408	233	351
228	343	207	311	187	281	26	289	434	244	367	210	315
197	295	179	268	161	242	28	263	396	222	334	190	286
171	257	156	234	140	211	30	242	364	204	306	174	262
150	226	137	205	123	185	32	224	337	188	283	161	242
133	200	121	182	109	164	34	209	313	175	263	149	224
119	179	108	162	97.5	147	36	195	293	164	246	139	209
107	160	96.9	146	87.5	131	38	183	276	154	231	131	196
96.3	145	87.5	131	79.0	119	40	173	260	145	218	123	185
						42	164	246	137	206	116	175
						44	156	234	130	195	110	166
						46	148	223	124	186	105	157
						48	141	212	118	177	99.9	150
						50	135	203	113	169	95.4	143

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1010	1510	914	1370	838	1260
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
810	1220	736	1100	675	1010
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
204	306	179	268	163	244
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
156	235	141	213	129	194

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.40	25.2	7.40	23.8	7.34	22.7
Area, in.^{2}					
24.0		21.8		20.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
881	148	795	134	722	121
r_y, in.					
2.48		2.48		2.46	
r_x/r_y					
2.44		2.44		2.44	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W14

F_y = 70 ksi
F_u = 90 ksi

W14×						Shape	W14×					
61^c		53^c		48^c		lb/ft	61^f		53		48	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
750	1130	652	979	580	871	0	356	535	304	457	274	412
687	1030	566	851	508	764	6	356	535	302	453	271	407
665	1000	538	808	485	728	7	356	535	292	439	262	393
641	964	506	761	456	686	8	350	526	282	424	252	379
615	924	473	711	426	640	9	341	512	272	409	243	366
587	882	438	659	395	593	10	332	498	263	395	234	352
557	838	403	606	362	545	11	322	485	253	380	225	338
527	792	368	553	330	496	12	313	471	243	366	215	324
495	745	333	500	299	449	13	304	457	233	351	206	310
464	697	299	449	268	402	14	295	443	224	336	197	296
432	649	266	400	238	358	15	286	430	214	322	187	282
400	601	234	352	210	315	16	277	416	204	307	178	268
369	555	208	312	186	279	17	267	402	194	292	167	251
339	509	185	278	166	249	18	258	388	182	273	153	231
309	465	166	250	149	224	19	249	374	168	253	142	213
280	421	150	226	134	202	20	240	361	157	236	132	198
232	348	124	186	111	167	22	219	329	138	207	116	174
195	293	104	157	93.2	140	24	194	291	123	185	103	155
166	249	88.8	133	79.4	119	26	173	261	111	167	92.7	139
143	215	76.6	115	68.5	103	28	157	236	101	153	84.3	127
125	187	66.7	100	59.7	89.7	30	143	215	93.3	140	77.4	116
110	165	58.6	88.1			32	132	198	86.4	130	71.5	107
97.0	146					34	122	184	80.4	121	66.5	99.9
86.5	130					36	114	171	75.3	113	62.1	93.4
77.7	117					38	107	160	70.8	106	58.3	87.6
70.1	105					40	100	151	66.8	100	55.0	82.6
						42	94.6	142	63.2	95.0	52.0	78.1
						44	89.5	135	60.0	90.2	49.3	74.1
						46	85.0	128	57.1	85.9	46.9	70.5
						48	81.0	122	54.5	82.0	44.8	67.3
						50	77.3	116	52.2	78.4	42.8	64.3

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
750	1130	654	983	591	888
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
604	906	527	790	476	714
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
146	219	144	216	131	197
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
114	172	76.8	116	68.5	103

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.33	21.6	5.73	17.4	5.70	16.7
Area, in.^{2}					
17.9		15.6		14.1	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
640	107	541	57.7	484	51.4
r_y, in.					
2.45		1.92		1.91	
r_x/r_y					
2.44		3.07		3.06	

[c] Shape is slender for compression with F_y = 70 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W14

F_y = 70 ksi
F_u = 90 ksi

W14×						Shape	W14×					
43^c		38^c		34^c		lb/ft	43		38		34	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Design	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
507	763	441	663	385	579	0	243	365	215	323	191	287
444	667	363	545	315	474	6	240	361	202	303	178	268
423	635	338	508	293	441	7	231	348	192	289	169	254
400	601	312	469	270	406	8	223	335	182	274	160	241
375	564	284	427	246	370	9	214	322	173	260	151	227
349	525	254	382	221	333	10	205	309	163	245	142	214
321	482	223	336	196	294	11	197	296	154	231	134	201
292	438	194	292	169	254	12	188	283	144	216	125	187
263	395	166	250	145	217	13	179	270	134	202	113	171
235	354	143	215	125	187	14	171	256	120	180	100	151
209	314	125	188	109	163	15	162	243	108	162	89.9	135
184	276	110	165	95.4	143	16	153	230	97.4	146	81.2	122
163	244	97.2	146	84.5	127	17	140	210	88.9	134	73.9	111
145	218	86.7	130	75.4	113	18	128	192	81.8	123	67.7	102
130	196	77.8	117	67.7	102	19	118	177	75.6	114	62.5	93.9
117	177	70.2	106	61.1	91.8	20	109	164	70.3	106	57.9	87.1
97.1	146	58.0	87.2	50.5	75.9	22	95.5	144	61.6	92.6	50.6	76.0
81.6	123	48.8	73.3	42.4	63.8	24	84.6	127	54.8	82.4	44.8	67.4
69.5	104					26	76.0	114	49.4	74.2	40.2	60.5
59.9	90.1					28	68.9	104	44.9	67.5	36.5	54.9
52.2	78.5					30	63.1	94.8	41.2	62.0	33.4	50.2
						32	58.2	87.4	38.1	57.3	30.8	46.3
						34	54.0	81.1	35.4	53.2	28.6	43.0
						36	50.4	75.7	33.1	49.8	26.7	40.1
						38	47.2	70.9	31.1	46.7	25.0	37.6
						40	44.4	66.8	29.3	44.0	23.5	35.4
						42	42.0	63.1	27.7	41.7	22.2	33.4
						44	39.8	59.8	26.3	39.5	21.1	31.7
						46	37.8	56.8	25.0	37.6	20.0	30.1
						48	36.0	54.2	23.9	35.9	19.1	28.7
						50	34.4	51.7	22.8	34.3	18.2	27.4

Properties

Available Strength in Tensile Yielding, kips							Limiting Unbraced Lengths, ft					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		L_p	L_r	L_p	L_r	L_p	L_r
528	794	469	706	419	630		5.64	16.0	4.63	13.1	4.57	12.7
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips							Area, in.^{2}					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		12.6		11.2		10.0	
425	638	378	567	338	506		Moment of Inertia, in.^{4}					
Available Strength in Shear, kips							I_x	I_y	I_x	I_y	I_x	I_y
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$		428	45.2	385	26.7	340	23.3
117	175	122	184	112	168		r_y, in.					
Available Strength in Flexure about Y-Y Axis, kip-ft							1.89		1.55		1.53	
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$		r_x/r_y					
60.4	90.8	42.3	63.5	37.0	55.7		3.08		3.79		3.81	

[c] Shape is slender for compression with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W14

F_y = 70 ksi
F_u = 90 ksi

W14×						Shape	W14×					
30^c		26^c		22^c		lb/ft	30^f		26^v		22^v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
334	503	283	426	230	346	0	160	241	140	211	116	174
271	408	191	288	151	227	6	153	229	115	172	92.2	139
252	378	166	250	130	196	7	145	217	105	158	84.0	126
231	347	141	212	109	164	8	136	205	95.9	144	75.8	114
209	314	116	174	90.1	135	9	128	193	86.5	130	65.4	98.2
187	282	93.6	141	73.3	110	10	120	181	72.5	109	54.4	81.7
166	250	77.4	116	60.6	91.0	11	112	169	61.9	93.1	46.2	69.4
142	214	65.0	97.7	50.9	76.5	12	104	157	53.9	81.0	39.9	60.0
121	182	55.4	83.3	43.4	65.2	13	91.7	138	47.5	71.5	35.0	52.7
105	157	47.8	71.8	37.4	56.2	14	80.9	122	42.5	63.8	31.1	46.8
91.1	137	41.6	62.5	32.6	48.9	15	72.1	108	38.3	57.6	28.0	42.0
80.1	120	36.6	55.0	28.6	43.0	16	64.9	97.5	34.9	52.4	25.3	38.1
71.0	107	32.4	48.7	25.4	38.1	17	58.9	88.5	32.0	48.1	23.1	34.8
63.3	95.1	28.9	43.4			18	53.8	80.9	29.5	44.4	21.3	32.0
56.8	85.4					19	49.5	74.4	27.4	41.2	19.7	29.6
51.3	77.1					20	45.8	68.8	25.6	38.5	18.3	27.5
42.4	63.7					22	39.7	59.7	22.6	34.0	16.1	24.1
35.6	53.5					24	35.1	52.7	20.2	30.4	14.3	21.5
						26	31.3	47.1	18.3	27.5	12.9	19.4
						28	28.3	42.6	16.7	25.1	11.7	17.6
						30	25.9	38.9	15.4	23.2	10.8	16.2
						32	23.8	35.7	14.3	21.5	9.94	14.9
						34	22.0	33.1	13.3	20.0	9.25	13.9
						36	20.5	30.8	12.5	18.8	8.64	13.0
						38	19.2	28.8	11.7	17.6	8.12	12.2
						40	18.0	27.1	11.1	16.7	7.65	11.5
						42	17.0	25.5	10.5	15.8	7.24	10.9
						44	16.1	24.2	9.98	15.0	6.87	10.3
						46	15.3	22.9	9.51	14.3	6.54	9.82
						48	14.5	21.8	9.08	13.6	6.23	9.37
						50	13.9	20.8	8.69	13.1	5.96	8.96

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
371	558	322	484	272	409
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
299	448	260	389	219	329
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
104	156	89.1	134	76.9	116
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
30.0	45.1	19.4	29.1	15.3	23.0

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.06	12.2	3.22	9.02	3.10	8.60
Area, in.^{2}					
8.85		7.69		6.49	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
291	19.6	245	8.91	199	7.00
r_y, in.					
1.49		1.08		1.04	
r_x/r_y					
3.85		5.23		5.33	

[c] Shape is slender for compression with F_y = 70 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

W12

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W12×						Shape		W12×					
336^h		305^h		279^h		lb/ft		336^h		305^h		279^h	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
4150	6230	3750	5640	3430	5160	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	2110	3170	1880	2820	1680	2530
3970	5960	3590	5390	3280	4930		6	2110	3170	1880	2820	1680	2530
3900	5870	3530	5300	3220	4840		7	2110	3170	1880	2820	1680	2530
3830	5760	3460	5200	3160	4750		8	2110	3170	1880	2820	1680	2530
3750	5640	3390	5090	3090	4650		9	2110	3170	1880	2820	1680	2530
3670	5510	3310	4970	3020	4540		10	2110	3170	1880	2820	1680	2530
3570	5370	3220	4840	2940	4410		11	2100	3160	1870	2810	1670	2510
3480	5220	3130	4700	2850	4280		12	2090	3140	1860	2790	1660	2500
3370	5070	3030	4560	2760	4150		13	2080	3130	1850	2780	1650	2490
3260	4900	2930	4400	2670	4010		14	2070	3110	1840	2770	1640	2470
3150	4730	2830	4250	2570	3860		15	2060	3100	1830	2750	1640	2460
3030	4550	2720	4080	2470	3710		16	2050	3080	1820	2740	1630	2450
2910	4370	2610	3920	2360	3550		17	2040	3070	1810	2720	1620	2430
2790	4190	2490	3750	2260	3400		18	2030	3060	1800	2710	1610	2420
2660	4010	2380	3580	2150	3240		19	2020	3040	1790	2700	1600	2400
2540	3820	2270	3410	2050	3080		20	2010	3030	1780	2680	1590	2390
2290	3450	2040	3060	1840	2760		22	2000	3000	1770	2650	1570	2360
2050	3080	1820	2730	1630	2450		24	1980	2970	1750	2630	1550	2340
1810	2720	1600	2410	1440	2160		26	1960	2940	1730	2600	1540	2310
1590	2380	1390	2090	1250	1870		28	1940	2910	1710	2570	1520	2280
1380	2080	1210	1820	1090	1630		30	1920	2880	1690	2540	1500	2260
1210	1820	1070	1600	954	1430		32	1900	2860	1670	2520	1480	2230
1080	1620	945	1420	845	1270		34	1880	2830	1650	2490	1460	2200
959	1440	843	1270	754	1130		36	1860	2800	1640	2460	1450	2170
861	1290	757	1140	676	1020		38	1840	2770	1620	2430	1430	2150
777	1170	683	1030	610	917		40	1820	2740	1600	2400	1410	2120
705	1060	619	931	554	832		42	1800	2710	1580	2380	1390	2090
642	965	564	848	504	758		44	1790	2680	1560	2350	1370	2070
587	883	516	776	462	694		46	1770	2660	1540	2320	1360	2040
539	811	474	713	424	637		48	1750	2630	1520	2290	1340	2010
497	747	437	657	391	587		50	1730	2600	1510	2260	1320	1980

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4150	6230	3750	5640	3430	5160
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3340	5010	3020	4530	2760	4150
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
837	1260	744	1120	681	1020
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
957	1440	852	1280	768	1160

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.4	107	10.2	97.6	10.1	89.9
Area, in.^{2}					
98.9		89.5		81.9	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
4060	1190	3550	1050	3110	937
r_y, in.					
3.47		3.42		3.38	
r_x/r_y					
1.85		1.84		1.82	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W12

F_y = 70 ksi
F_u = 90 ksi

W-Shapes

W12× 252^h		W12× 230^h		W12× 210		Shape lb/ft	W12× 252^h		W12× 230^h		W12× 210	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
3110	4670	2840	4270	2590	3890	0	1500	2250	1350	2030	1220	1830
2960	4450	2700	4060	2470	3710	6	1500	2250	1350	2030	1220	1830
2910	4380	2660	3990	2420	3640	7	1500	2250	1350	2030	1220	1830
2850	4290	2600	3910	2370	3570	8	1500	2250	1350	2030	1220	1830
2790	4190	2540	3820	2320	3480	9	1500	2250	1350	2030	1220	1830
2720	4090	2480	3730	2260	3390	10	1490	2250	1350	2020	1210	1820
2650	3980	2410	3620	2190	3300	11	1490	2230	1340	2010	1210	1810
2570	3860	2340	3510	2130	3200	12	1480	2220	1330	2000	1200	1800
2480	3730	2260	3400	2050	3090	13	1470	2210	1320	1990	1190	1790
2400	3600	2180	3280	1980	2980	14	1460	2190	1310	1970	1180	1770
2310	3470	2100	3150	1900	2860	15	1450	2180	1300	1960	1170	1760
2210	3330	2010	3020	1820	2740	16	1440	2170	1300	1950	1160	1750
2120	3190	1920	2890	1740	2620	17	1430	2150	1290	1930	1150	1730
2020	3040	1840	2760	1660	2500	18	1420	2140	1280	1920	1150	1720
1930	2900	1750	2620	1580	2370	19	1420	2130	1270	1910	1140	1710
1830	2750	1660	2490	1500	2250	20	1410	2110	1260	1890	1130	1700
1640	2460	1480	2220	1330	2010	22	1390	2090	1240	1870	1110	1670
1450	2180	1310	1970	1180	1770	24	1370	2060	1230	1840	1090	1650
1270	1910	1140	1720	1030	1540	26	1350	2030	1210	1820	1080	1620
1100	1650	988	1480	885	1330	28	1340	2010	1190	1790	1060	1590
959	1440	860	1290	771	1160	30	1320	1980	1170	1760	1040	1570
843	1270	756	1140	678	1020	32	1300	1960	1160	1740	1030	1540
746	1120	670	1010	600	902	34	1280	1930	1140	1710	1010	1520
666	1000	597	898	535	805	36	1270	1900	1120	1690	992	1490
598	898	536	806	481	722	38	1250	1880	1110	1660	975	1470
539	811	484	727	434	652	40	1230	1850	1090	1640	958	1440
489	735	439	660	393	591	42	1210	1820	1070	1610	941	1410
446	670	400	601	358	539	44	1200	1800	1050	1580	924	1390
408	613	366	550	328	493	46	1180	1770	1040	1560	907	1360
374	563	336	505	301	453	48	1160	1740	1020	1530	890	1340
345	519	310	465	278	417	50	1140	1720	1000	1510	873	1310

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3110	4670	2840	4270	2590	3890
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2500	3750	2280	3430	2090	3130
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
604	906	545	818	486	729
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
685	1030	618	929	555	835

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.97	81.7	9.88	75.0	9.79	68.7
Area, in.^{2}					
74.1		67.7		61.8	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
2720	828	2420	742	2140	664
r_y, in.					
3.34		3.31		3.28	
r_x/r_y					
1.81		1.80		1.80	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W12

F_y = 70 ksi
F_u = 90 ksi

W-Shapes

W12×						Shape		W12×					
190		170		152		lb/ft		190		170		152	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
2350	3530	2100	3150	1870	2820	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	1090	1630	961	1440	849	1280
2230	3360	1990	2990	1780	2670		6	1090	1630	961	1440	849	1280
2190	3290	1950	2940	1750	2620		7	1090	1630	961	1440	849	1280
2150	3230	1910	2880	1710	2570		8	1090	1630	961	1440	849	1280
2100	3150	1870	2810	1670	2500		9	1090	1630	961	1440	849	1280
2040	3070	1820	2730	1620	2440		10	1080	1630	957	1440	845	1270
1980	2980	1760	2650	1570	2360		11	1080	1620	949	1430	837	1260
1920	2890	1710	2570	1520	2290		12	1070	1600	941	1410	829	1250
1850	2790	1650	2480	1470	2200		13	1060	1590	933	1400	820	1230
1790	2680	1590	2380	1410	2120		14	1050	1580	924	1390	812	1220
1710	2580	1520	2290	1350	2030		15	1040	1570	916	1380	804	1210
1640	2470	1460	2190	1290	1940		16	1030	1550	908	1360	796	1200
1570	2360	1390	2090	1230	1850		17	1030	1540	900	1350	788	1180
1490	2240	1320	1990	1170	1760		18	1020	1530	892	1340	780	1170
1420	2130	1250	1890	1110	1670		19	1010	1520	883	1330	772	1160
1340	2020	1190	1780	1050	1580		20	1000	1500	875	1320	763	1150
1190	1800	1050	1580	929	1400		22	983	1480	859	1290	747	1120
1050	1580	924	1390	813	1220		24	967	1450	842	1270	731	1100
913	1370	800	1200	702	1060		26	950	1430	826	1240	715	1070
788	1180	690	1040	606	910		28	933	1400	809	1220	698	1050
686	1030	601	904	528	793		30	916	1380	793	1190	682	1030
603	906	528	794	464	697		32	900	1350	776	1170	666	1000
534	803	468	704	411	617		34	883	1330	760	1140	649	976
476	716	418	628	366	551		36	866	1300	743	1120	633	952
428	643	375	563	329	494		38	849	1280	727	1090	617	927
386	580	338	508	297	446		40	833	1250	710	1070	601	903
350	526	307	461	269	405		42	816	1230	694	1040	584	878
319	479	280	420	245	369		44	799	1200	677	1020	568	854
292	439	256	384	224	337		46	783	1180	661	993	552	829
268	403	235	353	206	310		48	766	1150	644	969	535	805
247	371	216	325	190	285		50	749	1130	628	944	519	780

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2350	3530	2100	3150	1870	2820
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1890	2840	1690	2530	1510	2260
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
427	641	376	564	334	501
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
500	751	440	662	388	583

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.70	62.7	9.61	56.5	9.52	51.0
Area, in.^{2}					
56.0		50.0		44.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1890	589	1650	517	1430	454
r_y, in.					
3.25		3.22		3.19	
r_x/r_y					
1.79		1.78		1.77	

Note: Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W12

F_y = 70 ksi
F_u = 90 ksi

W12×						Shape lb/ft		W12×					
136		120		106				136		120		106	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
1670	2510	1480	2220	1310	1970	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	748	1120	650	977	573	861
1590	2380	1400	2100	1240	1860		6	748	1120	650	977	573	861
1560	2340	1370	2060	1210	1820		7	748	1120	650	977	573	861
1520	2290	1340	2010	1190	1780		8	748	1120	650	977	573	861
1480	2230	1310	1960	1160	1740		9	748	1120	650	977	573	861
1440	2170	1270	1910	1120	1690		10	743	1120	645	969	567	853
1400	2100	1230	1850	1090	1630		11	735	1100	637	957	560	841
1350	2030	1190	1790	1050	1580		12	727	1090	629	945	552	829
1300	1960	1140	1720	1010	1520		13	719	1080	621	933	544	818
1250	1880	1100	1650	970	1460		14	711	1070	613	921	536	806
1200	1800	1050	1580	928	1390		15	703	1060	605	910	528	794
1150	1720	1000	1510	885	1330		16	695	1040	597	898	521	782
1090	1640	955	1440	842	1270		17	687	1030	589	886	513	771
1040	1560	906	1360	798	1200		18	679	1020	582	874	505	759
982	1480	857	1290	754	1130		19	671	1010	574	862	497	747
927	1390	808	1210	711	1070		20	662	996	566	850	489	736
819	1230	712	1070	625	940		22	646	972	550	827	474	712
715	1070	620	932	544	817		24	630	947	534	803	458	689
615	925	532	800	466	700		26	614	923	519	779	443	666
530	797	459	690	402	604		28	598	899	503	756	427	642
462	695	400	601	350	526		30	582	875	487	732	412	619
406	610	352	528	308	462		32	566	851	471	709	396	595
360	541	311	468	272	410		34	550	826	456	685	381	572
321	482	278	417	243	365		36	534	802	440	661	365	549
288	433	249	375	218	328		38	518	778	424	638	347	522
260	391	225	338	197	296		40	502	754	408	614	328	493
236	354	204	307	179	268		42	485	730	391	587	311	467
215	323	186	279	163	245		44	469	705	371	558	295	444
197	295	170	256	149	224		46	453	680	354	532	281	423
181	271	156	235	137	205		48	433	650	338	508	268	404
166	250	144	216	126	189		50	414	623	324	487	257	386

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1670	2510	1480	2220	1310	1970
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1350	2020	1190	1780	1050	1580
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
296	445	260	391	220	330
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
342	515	298	448	262	394

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.43	45.8	9.34	41.3	9.28	37.3
Area, in.^{2}					
39.9		35.2		31.2	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1240	398	1070	345	933	301
r_y, in.					
3.16		3.13		3.11	
r_x/r_y					
1.77		1.76		1.76	

Note: Confirm ASTM A913 material availability before specifying.

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W12

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W12×						Shape	W12×					
96		87		79		lb/ft	96		87		79[f]	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Design	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1180	1780	1070	1610	972	1460	0	513	772	461	693	410	616
1120	1680	1010	1520	919	1380	6	513	772	461	693	410	616
1100	1650	994	1490	900	1350	7	513	772	461	693	410	616
1070	1610	971	1460	879	1320	8	513	772	461	693	410	616
1040	1570	945	1420	855	1290	9	513	772	461	693	410	616
1010	1520	918	1380	830	1250	10	508	763	455	684	409	615
981	1470	888	1330	803	1210	11	500	751	447	672	402	604
946	1420	857	1290	774	1160	12	492	740	440	661	395	593
911	1370	824	1240	744	1120	13	485	729	432	650	387	582
873	1310	790	1190	713	1070	14	477	717	425	639	380	571
835	1260	755	1130	681	1020	15	470	706	418	628	373	560
796	1200	719	1080	648	974	16	462	694	410	616	366	549
757	1140	683	1030	615	925	17	454	683	403	605	358	538
717	1080	646	972	582	875	18	447	672	395	594	351	527
677	1020	610	917	549	825	19	439	660	388	583	344	517
637	958	574	863	516	775	20	432	649	380	572	336	506
560	842	503	757	452	679	22	417	626	365	549	322	484
486	730	436	655	390	587	24	401	603	351	527	307	462
416	625	373	560	333	501	26	386	580	336	504	293	440
358	539	321	483	287	432	28	371	558	321	482	278	418
312	469	280	421	250	376	30	356	535	306	460	264	396
274	413	246	370	220	331	32	341	512	291	437	244	366
243	365	218	327	195	293	34	325	489	271	408	226	340
217	326	194	292	174	261	36	306	461	253	381	211	317
195	293	174	262	156	234	38	288	433	238	357	198	297
176	264	157	237	141	212	40	272	408	224	337	186	280
159	239	143	215	128	192	42	257	386	212	318	176	264
145	218	130	196	116	175	44	244	367	201	302	167	250
133	200	119	179	106	160	46	232	349	191	287	158	238
122	183	109	164	97.8	147	48	222	333	182	274	151	227
112	169	101	151	90.1	135	50	212	319	174	262	144	217

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1180	1780	1070	1610	972	1460
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
952	1430	864	1300	783	1170
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
196	293	180	270	163	245
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
236	354	211	317	186	279

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.22	34.7	9.16	32.3	9.92	30.3
Area, in.^{2}					
28.2		25.6		23.2	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
833	270	740	241	662	216
r_y, in.					
3.09		3.07		3.05	
r_x/r_y					
1.76		1.75		1.75	

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

Note: Confirm ASTM A913 material availability before specifying.

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W12

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W12×						Shape	W12×					
72		65		58		lb/ft	72^f		65^f		58^f	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
884	1330	801	1200	713	1070	0	363	546	317	476	301	452
835	1260	755	1140	655	984	6	363	546	317	476	301	452
818	1230	740	1110	635	955	7	363	546	317	476	301	452
799	1200	722	1090	613	922	8	363	546	317	476	298	448
777	1170	702	1060	590	886	9	363	546	317	476	291	437
754	1130	681	1020	564	848	10	363	546	317	476	284	427
729	1100	658	990	537	807	11	363	546	317	476	277	416
703	1060	634	953	509	765	12	357	536	317	476	270	405
675	1020	609	916	480	721	13	350	525	311	468	263	395
647	972	583	877	450	677	14	342	515	305	458	255	384
618	928	557	836	421	633	15	335	504	298	448	248	373
588	884	529	796	391	588	16	328	493	291	437	241	362
558	838	502	754	362	545	17	321	483	284	427	234	352
528	793	474	713	334	502	18	314	472	278	417	227	341
497	747	447	671	306	460	19	307	462	271	407	220	330
467	702	419	630	279	420	20	300	451	264	397	213	320
409	614	366	550	231	347	22	286	430	251	377	198	298
353	530	316	474	194	292	24	272	409	237	356	181	272
301	453	269	404	165	249	26	258	387	224	336	162	244
260	390	232	349	143	214	28	244	366	207	312	148	222
226	340	202	304	124	187	30	225	338	189	284	135	203
199	299	178	267	109	164	32	207	311	173	260	125	188
176	265	157	236	96.7	145	34	192	288	160	241	116	174
157	236	140	211	86.3	130	36	179	268	149	224	108	163
141	212	126	189	77.4	116	38	167	251	139	209	102	153
127	191	114	171	69.9	105	40	157	236	130	196	95.7	144
115	173	103	155			42	148	223	123	185	90.5	136
105	158	93.9	141			44	140	211	116	175	85.8	129
96.2	145	85.9	129			46	133	200	110	166	81.6	123
88.3	133	78.9	119			48	127	191	105	158	77.8	117
81.4	122	72.7	109			50	121	182	100	150	74.3	112

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
884	1330	801	1200	713	1070
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
712	1070	645	967	574	861
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
148	222	132	198	123	184
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
163	244	140	210	113	170

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.0	28.8	12.2	27.3	7.60	23.1
Area, in.^{2}					
21.1		19.1		17.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
597	195	533	174	475	107
r_y, in.					
3.04		3.02		2.51	
r_x/r_y					
1.75		1.75		2.10	

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W12

F_y = 70 ksi
F_u = 90 ksi

W12×						Shape		W12×					
53		50		45		lb/ft		53[f]		50		45	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending		ASD	LRFD	ASD	LRFD	ASD	LRFD
654	983	612	920	549	825		0	265	398	251	377	224	337
600	902	533	801	478	718		6	265	398	250	376	223	335
581	874	507	762	454	683		7	265	398	243	365	216	324
561	843	479	720	428	644		8	265	398	235	353	209	314
539	809	448	674	401	603		9	261	393	228	342	202	303
515	773	417	627	373	560		10	254	382	220	331	195	292
489	735	385	578	344	516		11	248	372	213	320	187	282
463	696	352	529	314	472		12	241	362	205	308	180	271
436	655	320	481	285	429		13	234	352	198	297	173	260
409	614	288	434	257	386		14	227	342	190	286	166	250
381	573	258	388	230	345		15	220	331	183	275	159	239
354	532	229	344	203	305		16	214	321	175	263	152	228
327	492	203	304	180	270		17	207	311	168	252	145	218
301	452	181	272	160	241		18	200	301	160	241	136	204
275	414	162	244	144	216		19	193	291	150	226	126	189
250	376	146	220	130	195		20	187	280	140	211	117	176
207	311	121	182	107	161		22	173	260	124	186	103	155
174	261	102	153	90.3	136		24	153	230	111	167	92.0	138
148	223	86.6	130	76.9	116		26	137	206	101	151	83.1	125
128	192	74.7	112	66.3	99.7		28	124	187	91.9	138	75.8	114
111	167	65.0	97.8	57.8	86.8		30	114	171	84.6	127	69.7	105
97.8	147	57.2	85.9	50.8	76.3		32	105	157	78.5	118	64.5	97.0
86.6	130						34	97.2	146	73.2	110	60.1	90.3
77.3	116						36	90.6	136	68.6	103	56.2	84.5
69.4	104						38	84.9	128	64.5	97.0	52.9	79.4
62.6	94.1						40	79.8	120	60.9	91.6	49.9	75.0
							42	75.4	113	57.7	86.8	47.2	71.0
							44	71.4	107	54.9	82.5	44.8	67.4
							46	67.9	102	52.3	78.6	42.7	64.2
							48	64.7	97.2	49.9	75.1	40.7	61.2
							50	61.7	92.8	47.8	71.8	39.0	58.6

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
654	983	612	920	549	825
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
527	790	493	739	442	663
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
117	175	126	190	113	170
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
97.5	147	74.4	112	66.4	99.8

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.51	22.0	5.85	18.4	5.82	17.5
Area, in.^{2}					
15.6		14.6		13.1	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
425	95.8	391	56.3	348	50.0
r_y, in.					
2.48		1.96		1.95	
r_x/r_y					
2.11		2.64		2.64	

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W12

F_y = 70 ksi
F_u = 90 ksi

W-Shapes

Shape	W12× 40^c		35^c		30^c		Shape lb/ft	W12× 40^f		35		30	
	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips						Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
	482	724	415	624	343	516	0	199	299	179	269	151	226
	424	637	340	511	280	420	6	198	297	168	252	140	211
	404	608	317	476	260	391	7	191	287	160	241	133	201
	382	574	290	436	239	359	8	184	277	152	229	126	190
	357	537	261	392	217	326	9	178	267	145	217	120	180
	331	498	232	349	195	292	10	171	257	137	206	113	169
	305	459	204	306	170	256	11	164	247	129	194	106	159
	279	419	176	265	147	221	12	158	237	121	183	98.7	148
	253	380	151	227	125	189	13	151	227	114	171	89.9	135
	228	342	130	196	108	163	14	144	217	103	154	79.8	120
	203	305	113	170	94.2	142	15	138	207	92.6	139	71.6	108
	180	270	99.6	150	82.8	124	16	131	197	84.3	127	64.8	97.4
	159	239	88.2	133	73.3	110	17	123	185	77.2	116	59.1	88.9
	142	213	78.7	118	65.4	98.3	18	113	170	71.3	107	54.4	81.7
	127	191	70.6	106	58.7	88.3	19	104	157	66.1	99.4	50.3	75.5
	115	173	63.7	95.8	53.0	79.7	20	97.0	146	61.7	92.7	46.7	70.2
	95.0	143	52.7	79.2	43.8	65.8	22	85.0	128	54.4	81.7	40.9	61.5
	79.8	120	44.3	66.5	36.8	55.3	24	75.6	114	48.6	73.1	36.4	54.7
	68.0	102					26	68.0	102	44.0	66.1	32.8	49.3
	58.6	88.1					28	61.9	93.0	40.2	60.4	29.8	44.8
	51.1	76.8					30	56.8	85.3	37.0	55.6	27.3	41.1
	44.9	67.5					32	52.4	78.8	34.3	51.5	25.3	38.0
							34	48.7	73.3	31.9	48.0	23.5	35.3
							36	45.6	68.5	29.9	44.9	21.9	33.0
							38	42.8	64.3	28.1	42.3	20.6	31.0
							40	40.3	60.6	26.6	39.9	19.4	29.2
							42	38.1	57.3	25.2	37.8	18.4	27.6
							44	36.2	54.3	23.9	35.9	17.4	26.2
							46	34.4	51.7	22.8	34.2	16.6	24.9
							48	32.8	49.3	21.7	32.7	15.8	23.8
							50	31.4	47.1	20.8	31.3	15.1	22.7

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
490	737	432	649	368	554
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
395	592	348	521	297	445
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
98.3	147	105	158	89.5	134
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
58.6	88.1	40.2	60.4	33.4	50.2

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.82	16.7	4.60	13.3	4.54	12.6
Area, in.^{2}					
11.7		10.3		8.79	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
307	44.1	285	24.5	238	20.3
r_y, in.					
1.94		1.54		1.52	
r_x/r_y					
2.64		3.41		3.43	

[c] Shape is slender for compression with F_y = 70 ksi.

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W12

F_y = 70 ksi
F_u = 90 ksi

W12×						Shape	W12×					
26^c		22^c		19^c		lb/ft	$26^{f,v}$		22		19^v	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
291	437	246	370	204	307	0	127	191	102	154	86.3	130
236	355	130	195	106	159	6	121	181	73.7	111	59.9	90.0
219	329	99.3	149	80.2	120	7	114	172	65.4	98.3	52.4	78.8
201	302	76.0	114	61.4	92.3	8	108	162	54.6	82.1	41.9	63.0
182	274	60.0	90.3	48.5	72.9	9	102	153	45.3	68.1	34.5	51.9
164	246	48.6	73.1	39.3	59.0	10	95.3	143	38.6	58.0	29.2	43.9
145	218	40.2	60.4	32.5	48.8	11	89.0	134	33.5	50.4	25.2	37.9
126	190	33.8	50.8	27.3	41.0	12	82.7	124	29.6	44.5	22.1	33.3
108	162	28.8	43.3	23.2	34.9	13	72.8	109	26.5	39.8	19.7	29.6
92.9	140	24.8	37.3			14	64.4	96.8	24.0	36.0	17.7	26.7
80.9	122					15	57.5	86.5	21.9	32.9	16.1	24.2
71.1	107					16	51.9	78.0	20.1	30.3	14.8	22.2
63.0	94.7					17	47.2	70.9	18.6	28.0	13.6	20.5
56.2	84.5					18	43.2	64.9	17.4	26.1	12.7	19.0
50.4	75.8					19	39.8	59.9	16.2	24.4	11.8	17.8
45.5	68.4					20	36.9	55.5	15.3	23.0	11.1	16.7
37.6	56.5					22	32.1	48.3	13.6	20.5	9.86	14.8
31.6	47.5					24	28.4	42.8	12.3	18.5	8.88	13.3
						26	25.5	38.3	11.3	16.9	8.08	12.2
						28	23.1	34.7	10.4	15.6	7.42	11.2
						30	21.1	31.8	9.60	14.4	6.86	10.3
						32	19.5	29.3	8.95	13.4	6.39	9.60
						34	18.0	27.1	8.38	12.6	5.97	8.97
						36	16.8	25.3	7.88	11.8	5.61	8.43
						38	15.8	23.7	7.44	11.2	5.29	7.95
						40	14.8	22.3	7.04	10.6	5.00	7.52
						42	14.0	21.0	6.69	10.1	4.75	7.14
						44	13.3	19.9	6.37	9.58	4.52	6.79
						46	12.6	18.9	6.08	9.14	4.31	6.48
						48	12.0	18.0	5.82	8.74	4.12	6.19
						50	11.5	17.2	5.58	8.38	3.95	5.93

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
321	482	272	408	233	351
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
258	387	219	328	188	282
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
70.6	106	89.5	134	72.1	108
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
27.5	41.4	12.8	19.2	10.4	15.6

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.00	12.2	2.53	7.40	2.45	7.05
Area, in.^{2}					
7.65		6.48		5.57	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
204	17.3	156	4.66	130	3.76
r_y, in.					
1.51		0.848		0.822	
r_x/r_y					
3.42		5.79		5.86	

[c] Shape is slender for compression with F_y = 70 ksi.

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W12–W10

F_y = 70 ksi
F_u = 90 ksi

W12×				W10×		Shape	W12×				W10×	
16[c]		14[c]		112		lb/ft	16[v]		14[f, v]		112	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
166	250	141	212	1380	2070	0	70.2	106	58.7	88.2	513	772
80.0	120	65.6	98.6	1280	1930	6	46.0	69.2	38.8	58.4	513	772
60.0	90.1	50.2	75.5	1250	1870	7	38.0	57.2	31.0	46.6	513	772
45.9	69.0	38.5	57.8	1210	1820	8	30.1	45.3	24.4	36.7	513	772
36.3	54.5	30.4	45.7	1170	1760	9	24.7	37.1	19.9	29.9	508	764
29.4	44.2	24.6	37.0	1120	1690	10	20.7	31.1	16.7	25.0	503	756
24.3	36.5	20.3	30.6	1080	1620	11	17.8	26.7	14.2	21.4	497	747
20.4	30.7	17.1	25.7	1030	1540	12	15.5	23.3	12.4	18.6	492	739
				975	1470	13	13.8	20.7	10.9	16.4	487	731
				922	1390	14	12.3	18.5	9.76	14.7	481	723
				869	1310	15	11.2	16.8	8.80	13.2	476	715
				815	1230	16	10.2	15.3	8.01	12.0	470	707
				762	1150	17	9.37	14.1	7.35	11.0	465	699
				709	1070	18	8.67	13.0	6.78	10.2	460	691
				657	988	19	8.07	12.1	6.30	9.46	454	683
				607	912	20	7.55	11.3	5.87	8.83	449	675
				510	766	22	6.68	10.0	5.18	7.79	438	659
				428	644	24	6.00	9.02	4.64	6.97	427	642
				365	548	26	5.45	8.18	4.20	6.31	417	626
				315	473	28	4.99	7.50	3.83	5.76	406	610
				274	412	30	4.60	6.92	3.53	5.31	395	594
				241	362	32	4.27	6.42	3.27	4.92	384	578
				213	321	34	3.99	6.00	3.05	4.59	374	561
				190	286	36	3.74	5.62	2.86	4.30	363	545
				171	257	38	3.52	5.30	2.69	4.04	352	529
				154	232	40	3.33	5.01	2.54	3.82	341	513
				140	210	42	3.16	4.75	2.41	3.61	331	497
				127	191	44	3.00	4.51	2.29	3.43	320	481
						46	2.86	4.30	2.18	3.27	309	464
						48	2.74	4.11	2.08	3.12	296	445
						50	2.62	3.94	1.99	2.99	284	426

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
197	297	174	262	1380	2070
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
159	238	140	211	1110	1670
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
66.4	99.8	57.0	85.7	241	361
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
7.88	11.8	6.32	9.49	242	363

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
2.31	6.64	2.61	6.41	8.00	46.2
Area, in.^{2}					
4.71		4.16		32.9	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
103	2.82	88.6	2.36	716	236
r_y, in.					
0.773		0.753		2.68	
r_x/r_y					
6.04		6.14		1.74	

[c] Shape is slender for compression with F_y = 70 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W10

F_y = 70 ksi
F_u = 90 ksi

W10× 100 P_n/Ω_c	W10× 100 $\phi_c P_n$	W10× 88 P_n/Ω_c	W10× 88 $\phi_c P_n$	W10× 77 P_n/Ω_c	W10× 77 $\phi_c P_n$	Shape lb/ft	W10× 100 M_{nx}/Ω_b	W10× 100 $\phi_b M_{nx}$	W10× 88 M_{nx}/Ω_b	W10× 88 $\phi_b M_{nx}$	W10× 77 M_{nx}/Ω_b	W10× 77 $\phi_b M_{nx}$
Available Compressive Strength, kips						Design	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1230	1850	1090	1640	951	1430	0	454	683	395	593	341	512
1140	1710	1010	1520	880	1320	6	454	683	395	593	341	512
1110	1670	982	1480	855	1290	7	454	683	395	593	341	512
1070	1610	951	1430	828	1240	8	454	682	394	592	340	511
1040	1560	917	1380	797	1200	9	448	674	389	584	335	503
996	1500	881	1320	765	1150	10	443	666	383	576	329	495
953	1430	842	1270	731	1100	11	438	658	378	568	324	487
908	1360	802	1210	695	1040	12	432	650	373	561	319	480
861	1290	760	1140	658	989	13	427	642	368	553	314	472
814	1220	718	1080	621	933	14	422	634	362	545	309	464
766	1150	675	1010	583	876	15	416	626	357	537	304	456
718	1080	632	949	544	818	16	411	618	352	529	298	449
670	1010	589	885	507	762	17	406	610	347	521	293	441
622	935	546	821	469	706	18	400	602	342	513	288	433
576	865	505	759	433	651	19	395	594	336	505	283	425
530	797	465	698	398	598	20	390	586	331	498	278	418
444	667	388	583	331	497	22	379	570	321	482	267	402
373	560	326	490	278	418	24	368	554	310	466	257	387
318	478	278	417	237	356	26	358	538	300	450	247	371
274	412	239	360	204	307	28	347	522	289	435	237	356
239	359	209	313	178	267	30	337	506	279	419	226	340
210	315	183	276	156	235	32	326	490	268	403	216	325
186	279	162	244	139	208	34	315	474	258	387	204	307
166	249	145	218	124	186	36	305	458	247	371	192	288
149	224	130	195	111	167	38	294	442	235	354	181	272
134	202	117	176	100	150	40	283	426	223	335	171	257
122	183	106	160	90.8	136	42	272	409	212	318	162	244
111	167					44	259	390	202	303	155	232
						46	248	372	192	289	147	222
						48	237	356	184	277	141	212
						50	227	341	176	265	135	203

Properties

Available Strength in Tensile Yielding, kips

W10×100 P_n/Ω_t	W10×100 $\phi_t P_n$	W10×88 P_n/Ω_t	W10×88 $\phi_t P_n$	W10×77 P_n/Ω_t	W10×77 $\phi_t P_n$
1230	1850	1090	1640	951	1430

Available Strength in Tensile Rupture (A_e = 0.75A_g), kips

W10×100 P_n/Ω_t	W10×100 $\phi_t P_n$	W10×88 P_n/Ω_t	W10×88 $\phi_t P_n$	W10×77 P_n/Ω_t	W10×77 $\phi_t P_n$
989	1480	878	1320	766	1150

Available Strength in Shear, kips

W10×100 V_n/Ω_v	W10×100 $\phi_v V_n$	W10×88 V_n/Ω_v	W10×88 $\phi_v V_n$	W10×77 V_n/Ω_v	W10×77 $\phi_v V_n$
211	317	183	274	157	236

Available Strength in Flexure about Y-Y Axis, kip-ft

W10×100 M_{ny}/Ω_b	W10×100 $\phi_b M_{ny}$	W10×88 M_{ny}/Ω_b	W10×88 $\phi_b M_{ny}$	W10×77 M_{ny}/Ω_b	W10×77 $\phi_b M_{ny}$
213	320	185	279	160	241

Limiting Unbraced Lengths, ft

W10×100 L_p	W10×100 L_r	W10×88 L_p	W10×88 L_r	W10×77 L_p	W10×77 L_r
7.91	41.8	7.85	37.2	7.76	33.1

Area, in.^{2}

W10×100	W10×88	W10×77
29.3	26.0	22.7

Moment of Inertia, in.^{4}

W10×100 I_x	W10×100 I_y	W10×88 I_x	W10×88 I_y	W10×77 I_x	W10×77 I_y
623	207	534	179	455	154

r_y, in.

W10×100	W10×88	W10×77
2.65	2.63	2.60

r_x/r_y

W10×100	W10×88	W10×77
1.74	1.73	1.73

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W10

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W10×						Shape	W10×					
68		60		54		lb/ft	68		60		54[f]	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
834	1250	742	1120	662	995	0	298	448	261	392	230	345
771	1160	685	1030	611	918	6	298	448	261	392	230	345
749	1130	665	1000	593	892	7	298	448	261	392	230	345
725	1090	643	967	573	862	8	297	446	259	389	230	345
698	1050	619	931	552	830	9	292	438	254	382	226	340
670	1010	594	892	529	795	10	286	431	249	374	221	333
639	961	566	851	504	758	11	281	423	244	367	216	325
608	914	538	809	479	720	12	276	415	239	359	212	318
575	865	509	765	453	681	13	271	408	234	352	207	311
542	815	479	720	426	641	14	266	400	229	345	202	304
509	765	449	675	399	600	15	261	393	224	337	197	296
475	714	419	630	372	560	16	256	385	219	330	192	289
442	664	389	585	346	520	17	251	377	214	322	188	282
409	615	360	541	320	480	18	246	370	210	315	183	275
377	567	331	498	294	442	19	241	362	205	307	178	267
346	521	304	457	269	405	20	236	355	200	300	173	260
288	433	252	379	223	336	22	226	339	190	285	163	246
242	364	212	318	188	282	24	216	324	180	270	154	231
206	310	181	271	160	240	26	206	309	170	255	143	215
178	267	156	234	138	207	28	195	294	159	239	130	196
155	233	136	204	120	180	30	185	279	146	220	120	180
136	205	119	179	106	159	32	172	259	136	204	111	167
121	181	106	159	93.5	141	34	161	242	127	190	103	155
108	162	94.2	142	83.4	125	36	151	227	119	178	96.7	145
96.5	145	84.5	127	74.8	112	38	142	214	112	168	91.0	137
87.1	131	76.3	115	67.6	102	40	134	202	105	159	85.8	129
79.0	119	69.2	104	61.3	92.1	42	128	192	100	150	81.3	122
						44	121	182	95.0	143	77.2	116
						46	116	174	90.5	136	73.5	110
						48	111	166	86.5	130	70.2	105
						50	106	159	82.8	124	67.1	101

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
834	1250	742	1120	662	995
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
672	1010	597	896	533	800
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
137	205	120	180	105	157
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
140	211	122	184	107	161

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.73	30.1	7.67	27.4	8.23	25.5
Area, in.^{2}					
19.9		17.7		15.8	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
394	134	341	116	303	103
r_y, in.					
2.59		2.57		2.56	
r_x/r_y					
1.71		1.71		1.71	

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W10

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W10×						Shape	W10×					
49		45		39		lb/ft	49^f		45		39	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
604	907	557	838	482	724	0	204	306	192	288	163	246
556	836	489	735	421	633	6	204	306	192	288	163	245
540	811	466	701	401	603	7	204	306	187	281	158	238
521	784	441	663	379	570	8	204	306	182	273	154	231
502	754	415	624	355	534	9	204	306	177	266	149	224
480	722	387	582	331	497	10	200	300	172	258	144	217
458	688	359	539	306	460	11	195	293	167	251	139	209
434	653	330	495	281	422	12	190	286	162	243	135	202
410	617	301	452	255	384	13	186	279	157	236	130	195
386	580	273	410	231	347	14	181	272	152	228	125	188
361	543	245	369	207	311	15	176	265	147	221	120	181
336	505	219	329	184	276	16	172	258	142	213	116	174
312	469	194	292	163	245	17	167	251	137	206	111	167
288	433	173	260	145	218	18	162	244	132	198	106	159
265	398	155	234	130	196	19	158	237	127	191	100	151
242	364	140	211	118	177	20	153	230	122	183	93.9	141
200	301	116	174	97.2	146	22	144	216	109	164	83.0	125
168	253	97.4	146	81.7	123	24	134	202	98.1	147	74.4	112
143	216	83.0	125	69.6	105	26	122	183	89.2	134	67.4	101
124	186	71.5	108	60.0	90.2	28	111	166	81.9	123	61.7	92.8
108	162	62.3	93.7	52.3	78.6	30	102	153	75.7	114	56.9	85.5
94.7	142	54.8	82.3	46.0	69.1	32	93.9	141	70.3	106	52.8	79.4
83.9	126					34	87.3	131	65.8	98.8	49.3	74.0
74.8	112					36	81.6	123	61.7	92.8	46.2	69.4
67.2	101					38	76.7	115	58.2	87.5	43.5	65.4
60.6	91.1					40	72.3	109	55.0	82.7	41.1	61.7
55.0	82.6					42	68.4	103	52.2	78.5	38.9	58.5
						44	64.9	97.5	49.7	74.7	37.0	55.6
						46	61.7	92.8	47.4	71.2	35.3	53.0
						48	58.9	88.5	45.3	68.1	33.7	50.7
						50	56.3	84.6	43.4	65.2	32.3	48.5

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
604	907	557	838	482	725
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
486	729	449	673	388	582
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
95.2	143	99.0	148	87.5	131
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
93.8	141	70.9	107	60.1	90.3

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.16	24.2	6.00	20.4	5.91	18.7
Area, in.^{2}					
14.4		13.3		11.5	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
272	93.4	248	53.4	209	45.0
r_y, in.					
2.54		2.01		1.98	
r_x/r_y					
1.71		2.15		2.16	

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W10

F_y = 70 ksi
F_u = 90 ksi

Shape W10×	33	33	30	30	26c	26c	lb/ft	33f	33f	30	30	26	26
	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips						Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
	407	612	371	557	311	468	0	130	195	128	192	109	164
	353	531	279	420	239	360	6	130	195	117	176	99.1	149
	336	505	252	379	216	324	7	130	195	111	167	93.8	141
	317	476	224	337	192	288	8	126	189	106	159	88.6	133
	296	445	196	295	167	251	9	121	182	99.8	150	83.3	125
	275	413	169	254	144	216	10	117	176	94.1	141	78.0	117
	253	381	143	215	121	182	11	113	169	88.4	133	72.7	109
	232	348	120	181	102	153	12	108	162	82.7	124	66.9	101
	210	316	102	154	86.9	131	13	104	156	75.8	114	59.4	89.3
	189	284	88.4	133	75.0	113	14	99.3	149	68.4	103	53.3	80.1
	169	253	77.0	116	65.3	98.1	15	94.9	143	62.2	93.5	48.3	72.7
	149	224	67.7	102	57.4	86.3	16	90.5	136	57.1	85.8	44.2	66.4
	132	198	59.9	90.1	50.8	76.4	17	86.0	129	52.8	79.3	40.7	61.2
	118	177	53.5	80.3	45.3	68.2	18	79.4	119	49.0	73.7	37.7	56.7
	106	159	48.0	72.1	40.7	61.2	19	73.6	111	45.8	68.9	35.1	52.8
	95.4	143	43.3	65.1	36.7	55.2	20	68.5	103	43.0	64.6	32.9	49.5
	78.8	118	35.8	53.8	30.4	45.6	22	60.2	90.5	38.3	57.6	29.2	43.9
	66.2	99.5					24	53.7	80.8	34.6	51.9	26.2	39.4
	56.4	84.8					26	48.5	72.9	31.5	47.3	23.8	35.8
	48.7	73.1					28	44.2	66.5	28.9	43.5	21.9	32.9
	42.4	63.7					30	40.6	61.1	26.8	40.3	20.2	30.3
	37.3	56.0					32	37.6	56.5	24.9	37.5	18.8	28.2
							34	35.0	52.6	23.3	35.1	17.5	26.3
							36	32.8	49.2	21.9	32.9	16.4	24.7
							38	30.8	46.3	20.7	31.1	15.5	23.3
							40	29.0	43.7	19.6	29.4	14.7	22.0
							42	27.5	41.3	18.6	27.9	13.9	20.9
							44	26.1	39.2	17.7	26.6	13.2	19.9
							46	24.9	37.4	16.9	25.4	12.6	18.9
							48	23.7	35.6	16.1	24.3	12.0	18.1
							50	22.7	34.1	15.5	23.2	11.5	17.3

Properties

Shape W10×	33	33	30	30	26	26
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	407	612	371	557	319	479
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	328	492	298	448	257	385
Available Strength in Shear, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	79.0	119	88.2	132	75.0	112
Available Strength in Flexure about Y-Y Axis, kip-ft	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	45.9	69.0	30.9	46.4	26.2	39.4

Shape W10×	33	33	30	30	26	26
Limiting Unbraced Lengths, ft	L_p	L_r	L_p	L_r	L_p	L_r
	7.06	17.1	4.09	12.6	4.06	11.9
Area, in.^{2}	9.71		8.84		7.61	
Moment of Inertia, in.^{4}	I_x	I_y	I_x	I_y	I_x	I_y
	171	36.6	170	16.7	144	14.1
r_y, in.	1.94		1.37		1.36	
r_x/r_y	2.16		3.20		3.20	

[c] Shape is slender for compression with F_y = 70 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W10

F_y = 70 ksi
F_u = 90 ksi

W10×						Shape	W10×					
22^c		19^c		17^c		lb/ft	22^f		19		17	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Design	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
260	391	226	339	197	297	0	90.1	135	75.4	113	65.3	98.2
199	300	118	177	99.5	150	6	81.1	122	56.1	84.4	47.0	70.7
181	272	91.4	137	75.9	114	7	76.4	115	50.4	75.8	41.8	62.8
160	240	70.0	105	58.1	87.3	8	71.6	108	44.0	66.2	34.9	52.5
139	208	55.3	83.1	45.9	69.0	9	66.8	100	36.8	55.3	29.0	43.6
118	178	44.8	67.3	37.2	55.9	10	62.1	93.3	31.5	47.4	24.7	37.2
99.0	149	37.0	55.7	30.7	46.2	11	57.3	86.1	27.5	41.4	21.5	32.3
83.2	125	31.1	46.8	25.8	38.8	12	50.1	75.3	24.4	36.7	19.0	28.5
70.9	107	26.5	39.9	22.0	33.1	13	44.2	66.5	21.9	33.0	17.0	25.5
61.1	91.9	22.9	34.4	19.0	28.5	14	39.5	59.3	19.9	30.0	15.4	23.1
53.3	80.0					15	35.6	53.5	18.3	27.4	14.0	21.1
46.8	70.4					16	32.4	48.7	16.8	25.3	12.9	19.4
41.5	62.3					17	29.7	44.7	15.6	23.5	12.0	18.0
37.0	55.6					18	27.4	41.2	14.6	21.9	11.1	16.7
33.2	49.9					19	25.5	38.3	13.7	20.6	10.4	15.7
30.0	45.0					20	23.8	35.7	12.9	19.4	9.81	14.7
24.8	37.2					22	21.0	31.5	11.5	17.4	8.76	13.2
						24	18.8	28.2	10.5	15.7	7.92	11.9
						26	17.0	25.5	9.57	14.4	7.23	10.9
						28	15.5	23.3	8.82	13.3	6.66	10.0
						30	14.3	21.5	8.19	12.3	6.17	9.27
						32	13.2	19.9	7.64	11.5	5.75	8.64
						34	12.3	18.5	7.16	10.8	5.39	8.09
						36	11.6	17.4	6.74	10.1	5.06	7.61
						38	10.9	16.3	6.36	9.56	4.78	7.19
						40	10.3	15.4	6.03	9.06	4.53	6.81
						42	9.73	14.6	5.73	8.61	4.30	6.46
						44	9.24	13.9	5.46	8.21	4.10	6.16
						46	8.80	13.2	5.21	7.84	3.91	5.88
						48	8.41	12.6	4.99	7.50	3.74	5.62
						50	8.05	12.1	4.78	7.19	3.58	5.39

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
272	409	236	354	209	314
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
219	329	190	285	168	253
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
68.5	103	71.4	107	67.9	102
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
21.1	31.7	11.7	17.6	9.78	14.7

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
4.12	11.1	2.61	7.79	2.52	7.41
Area, in.^{2}					
6.49		5.62		4.99	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
118	11.4	96.3	4.29	81.9	3.56
r_y, in.					
1.33		0.874		0.845	
r_x/r_y					
3.21		4.74		4.79	

[c] Shape is slender for compression with F_y = 70 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W10–W8

F_y = 70 ksi
F_u = 90 ksi

Shape	W10×				W8×			W10×				W8×	
lb/ft	15^c		12^c		67			15		12$^{f, v}$		67	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	171	257	129	193	826	1240	0	55.9	84.0	41.7	62.6	245	368
	82.3	124	62.7	94.3	734	1100	6	38.7	58.1	29.3	44.0	245	368
	61.6	92.6	46.5	69.8	703	1060	7	33.9	50.9	24.3	36.5	243	365
	47.2	70.9	35.6	53.5	669	1010	8	27.1	40.8	19.2	28.9	239	359
	37.3	56.0	28.1	42.3	633	952	9	22.4	33.7	15.7	23.7	236	354
	30.2	45.4	22.8	34.2	595	894	10	19.0	28.6	13.2	19.9	232	349
	25.0	37.5	18.8	28.3	555	835	11	16.5	24.7	11.4	17.1	229	344
	21.0	31.5	15.8	23.8	515	774	12	14.5	21.8	9.92	14.9	225	338
	17.9	26.9	13.5	20.3	474	713	13	12.9	19.4	8.79	13.2	222	333
					434	653	14	11.6	17.5	7.88	11.8	218	328
					395	593	15	10.6	15.9	7.13	10.7	215	323
					357	536	16	9.73	14.6	6.51	9.79	211	318
					320	481	17	9.00	13.5	5.99	9.00	208	312
					285	429	18	8.36	12.6	5.54	8.33	204	307
					256	385	19	7.81	11.7	5.16	7.76	201	302
					231	347	20	7.33	11.0	4.83	7.25	197	297
					191	287	22	6.54	9.82	4.27	6.43	191	286
					160	241	24	5.90	8.86	3.84	5.77	184	276
					137	205	26	5.37	8.08	3.48	5.24	177	266
					118	177	28	4.94	7.42	3.19	4.80	170	255
					103	154	30	4.57	6.87	2.94	4.43	163	245
					90.3	136	32	4.26	6.40	2.73	4.11	156	234
					79.9	120	34	3.98	5.98	2.55	3.84	149	224
							36	3.74	5.62	2.39	3.60	141	211
							38	3.53	5.31	2.26	3.39	133	200
							40	3.34	5.02	2.13	3.20	126	190
							42	3.17	4.77	2.02	3.04	120	180
							44	3.02	4.54	1.92	2.89	114	172
							46	2.88	4.33	1.83	2.75	109	164
							48	2.76	4.14	1.75	2.63	105	157
							50	2.64	3.97	1.68	2.52	100	151

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
185	278	148	223	826	1240
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
149	223	119	179	665	997
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
64.3	96.5	47.2	70.9	144	215
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
8.03	12.1	5.62	8.45	114	172

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
2.42	7.03	2.92	6.65	6.33	34.4
Area, in.^{2}					
4.41		3.54		19.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
68.9	2.89	53.8	2.18	272	88.6
r_y, in.					
0.810		0.785		2.12	
r_x/r_y					
4.88		4.97		1.75	

[c] Shape is slender for compression with F_y = 70 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 70 ksi; therefore ϕ_v = 0.90 and Ω_v = 1.67.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

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Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W8

F_y = 70 ksi
F_u = 90 ksi

W8× 58		W8× 48		W8× 40		Shape lb/ft	W8× 58		W8× 48		W8× 40	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
717	1080	591	888	490	737	0	209	314	171	257	139	209
636	955	523	786	432	649	6	209	314	171	257	139	209
608	915	500	752	412	620	7	206	310	169	253	136	205
579	870	475	714	391	588	8	203	305	165	248	133	200
547	822	448	674	368	553	9	200	300	162	243	130	195
513	771	420	632	344	517	10	196	295	159	238	126	190
478	719	391	588	319	480	11	193	290	155	233	123	185
443	666	362	544	294	443	12	189	285	152	228	120	180
407	612	332	499	270	405	13	186	279	148	223	117	176
372	560	303	456	245	368	14	182	274	145	218	114	171
338	508	275	413	221	332	15	179	269	142	213	110	166
305	458	247	371	198	298	16	176	264	138	208	107	161
272	409	220	331	176	264	17	172	259	135	203	104	156
243	365	197	295	157	236	18	169	254	132	198	101	151
218	328	176	265	141	212	19	165	249	128	193	97.5	147
197	296	159	239	127	191	20	162	243	125	188	94.3	142
163	244	132	198	105	158	22	155	233	118	178	87.9	132
137	205	111	166	88.2	133	24	148	223	112	168	79.7	120
116	175	94.2	142	75.2	113	26	141	213	105	158	72.7	109
100	151	81.2	122	64.8	97.4	28	135	202	96.6	145	66.8	100
87.5	131	70.7	106	56.5	84.9	30	128	192	89.6	135	61.8	92.9
76.9	116	62.2	93.5	49.6	74.6	32	119	180	83.6	126	57.6	86.5
68.1	102	55.1	82.8	44.0	66.1	34	112	168	78.3	118	53.9	81.0
						36	106	159	73.7	111	50.6	76.1
						38	99.7	150	69.6	105	47.8	71.8
						40	94.6	142	65.9	99.1	45.2	67.9
						42	89.9	135	62.7	94.2	42.9	64.5
						44	85.7	129	59.7	89.7	40.9	61.4
						46	81.8	123	57.0	85.7	39.0	58.6
						48	78.3	118	54.5	82.0	37.3	56.0
						50	75.1	113	52.3	78.6	35.7	53.7

Properties

W8× 58		W8× 48		W8× 40	
Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
717	1080	591	888	490	737
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
577	866	476	714	395	592
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
125	187	95.2	143	83.2	125
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
97.5	146	80.0	120	64.6	97.1

W8× 58		W8× 48		W8× 40	
Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
6.27	30.2	6.21	25.8	6.09	22.3
Area, in.^{2}					
17.1		14.1		11.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
228	75.1	184	60.9	146	49.1
r_y, in.					
2.10		2.08		2.04	
r_x/r_y					
1.74		1.74		1.73	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W8

F_y = 70 ksi
F_u = 90 ksi

W8×						Shape	W8×					
35		31		28		lb/ft	35^f		31^f		28	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
432	649	383	575	346	520	0	120	180	102	153	95.0	143
380	570	336	505	283	425	6	120	180	102	153	91.3	137
362	545	321	482	263	395	7	118	178	102	153	88.1	132
343	516	304	456	241	363	8	115	173	100	151	84.9	128
323	486	286	429	219	330	9	112	168	97.3	146	81.7	123
302	454	267	401	197	296	10	109	164	94.3	142	78.5	118
280	421	247	372	175	263	11	106	159	91.3	137	75.4	113
258	388	227	342	154	231	12	103	154	88.3	133	72.2	108
236	355	208	312	134	201	13	99.5	150	85.3	128	69.0	104
214	322	189	283	115	173	14	96.4	145	82.3	124	65.8	98.9
193	290	170	255	100	151	15	93.3	140	79.3	119	62.6	94.1
173	260	152	228	88.3	133	16	90.1	135	76.3	115	59.4	89.3
153	230	135	202	78.2	118	17	87.0	131	73.3	110	55.0	82.6
137	206	120	180	69.8	105	18	83.9	126	70.3	106	51.2	76.9
123	184	108	162	62.6	94.1	19	80.8	121	67.3	101	47.9	71.9
111	166	97.2	146	56.5	84.9	20	77.6	117	62.9	94.6	45.0	67.6
91.5	138	80.3	121	46.7	70.2	22	69.5	105	55.7	83.7	40.1	60.3
76.9	116	67.5	101	39.2	59.0	24	62.6	94.1	49.9	75.0	36.3	54.5
65.5	98.5	57.5	86.5	33.4	50.2	26	56.9	85.5	45.3	68.1	33.1	49.8
56.5	84.9	49.6	74.5			28	52.2	78.5	41.4	62.3	30.5	45.8
49.2	74.0	43.2	64.9			30	48.2	72.5	38.2	57.4	28.2	42.4
43.3	65.0	38.0	57.1			32	44.8	67.4	35.5	53.3	26.3	39.5
						34	41.9	63.0	33.1	49.8	24.6	37.0
						36	39.3	59.1	31.0	46.7	23.1	34.8
						38	37.1	55.7	29.2	43.9	21.8	32.8
						40	35.1	52.7	27.6	41.5	20.7	31.1
						42	33.3	50.0	26.2	39.3	19.6	29.5
						44	31.7	47.6	24.9	37.4	18.7	28.1
						46	30.2	45.4	23.7	35.7	17.8	26.8
						48	28.9	43.4	22.7	34.1	17.1	25.7
						50	27.6	41.5	21.7	32.6	16.4	24.6

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
432	649	383	575	346	520
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
348	521	308	462	278	418
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
70.5	106	63.8	95.8	64.3	96.5
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
55.4	83.2	46.2	69.4	35.3	53.0

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
6.48	20.4	7.53	19.0	4.84	16.0
Area, in.^{2}					
10.3		9.13		8.25	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
127	42.6	110	37.1	98.0	21.7
r_y, in.					
2.03		2.02		1.62	
r_x/r_y					
1.73		1.72		2.13	

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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W8

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W8×						Shape	W8×					
24		21		18		lb/ft	24^f		21		18^f	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
297	446	258	388	220	331	0	79.8	120	71.3	107	59.0	88.7
242	363	185	278	155	233	6	77.1	116	63.6	95.6	52.0	78.2
225	338	164	246	137	206	7	74.1	111	60.1	90.4	48.9	73.5
206	310	143	214	118	178	8	71.1	107	56.7	85.2	45.7	68.7
187	281	122	183	100	151	9	68.1	102	53.3	80.1	42.6	64.0
168	253	102	153	83.1	125	10	65.1	97.9	49.9	74.9	39.4	59.3
149	224	84.4	127	68.6	103	11	62.1	93.4	46.4	69.8	35.7	53.7
131	197	70.9	107	57.7	86.7	12	59.1	88.9	42.2	63.4	31.4	47.1
113	170	60.4	90.8	49.2	73.9	13	56.1	84.4	37.8	56.8	27.9	42.0
97.7	147	52.1	78.3	42.4	63.7	14	53.1	79.9	34.2	51.4	25.2	37.8
85.1	128	45.4	68.2	36.9	55.5	15	49.6	74.5	31.2	46.9	22.9	34.4
74.8	112	39.9	59.9	32.4	48.8	16	45.4	68.2	28.7	43.2	21.0	31.5
66.3	99.6	35.3	53.1	28.7	43.2	17	41.9	63.0	26.6	40.0	19.4	29.1
59.1	88.9	31.5	47.4	25.6	38.5	18	38.9	58.4	24.8	37.3	18.0	27.1
53.1	79.8	28.3	42.5	23.0	34.6	19	36.3	54.5	23.2	34.9	16.8	25.3
47.9	72.0	25.5	38.4	20.8	31.2	20	34.0	51.1	21.8	32.8	15.8	23.7
39.6	59.5					22	30.3	45.5	19.5	29.3	14.0	21.1
33.3	50.0					24	27.2	41.0	17.6	26.5	12.6	19.0
28.3	42.6					26	24.8	37.3	16.1	24.2	11.5	17.3
						28	22.8	34.2	14.8	22.3	10.6	15.9
						30	21.1	31.6	13.7	20.6	9.79	14.7
						32	19.6	29.4	12.8	19.2	9.11	13.7
						34	18.3	27.5	12.0	18.0	8.52	12.8
						36	17.2	25.8	11.3	17.0	8.00	12.0
						38	16.2	24.4	10.6	16.0	7.55	11.3
						40	15.3	23.1	10.1	15.2	7.14	10.7
						42	14.6	21.9	9.58	14.4	6.78	10.2
						44	13.9	20.8	9.12	13.7	6.45	9.69
						46	13.2	19.9	8.71	13.1	6.15	9.25
						48	12.6	19.0	8.33	12.5	5.88	8.84
						50	12.1	18.2	7.98	12.0	5.64	8.47

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
297	446	258	388	220	331
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
239	358	208	312	178	266
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
54.4	81.6	58.0	86.9	52.4	78.6
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
29.4	44.3	19.9	29.9	16.1	24.2

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.11	14.7	3.76	11.6	3.79	10.7
Area, in.^{2}					
7.08		6.16		5.26	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
82.7	18.3	75.3	9.77	61.9	7.97
r_y, in.					
1.61		1.26		1.23	
r_x/r_y					
2.12		2.77		2.79	

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

W8

F_y = 70 ksi
F_u = 90 ksi

W8×						Shape		W8×					
15		13		10^c		lb/ft		15		13^f		10^f	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
186	280	161	242	115	172	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	47.5	71.4	39.7	59.7	29.2	43.9
93.2	140	76.3	115	58.6	88.1		6	35.7	53.7	28.8	43.3	21.7	32.6
72.6	109	58.1	87.4	44.6	67.0		7	32.3	48.5	25.7	38.6	19.0	28.5
55.6	83.5	44.5	66.9	34.1	51.3		8	28.7	43.2	21.7	32.6	15.1	22.7
43.9	66.0	35.2	52.9	27.0	40.5		9	24.1	36.2	18.1	27.2	12.5	18.7
35.6	53.5	28.5	42.8	21.9	32.8		10	20.7	31.2	15.5	23.3	10.5	15.8
29.4	44.2	23.5	35.4	18.1	27.1		11	18.2	27.3	13.5	20.3	9.11	13.7
24.7	37.1	19.8	29.7	15.2	22.8		12	16.2	24.3	12.0	18.0	8.00	12.0
21.0	31.6	16.9	25.3	12.9	19.4		13	14.6	21.9	10.7	16.1	7.12	10.7
18.1	27.3	14.5	21.8	11.1	16.8		14	13.3	20.0	9.75	14.6	6.41	9.64
							15	12.2	18.3	8.92	13.4	5.83	8.76
							16	11.3	16.9	8.23	12.4	5.35	8.03
							17	10.5	15.7	7.63	11.5	4.93	7.42
							18	9.79	14.7	7.12	10.7	4.58	6.89
							19	9.19	13.8	6.67	10.0	4.28	6.43
							20	8.67	13.0	6.28	9.44	4.01	6.03
							22	7.78	11.7	5.63	8.46	3.57	5.36
							24	7.06	10.6	5.10	7.66	3.22	4.83
							26	6.47	9.72	4.66	7.00	2.93	4.40
							28	5.97	8.97	4.29	6.45	2.69	4.04
							30	5.54	8.33	3.98	5.99	2.49	3.74
							32	5.17	7.78	3.72	5.58	2.31	3.48
							34	4.85	7.29	3.48	5.23	2.16	3.25
							36	4.57	6.87	3.28	4.92	2.03	3.06
							38	4.32	6.49	3.09	4.65	1.92	2.88
							40	4.09	6.15	2.93	4.41	1.81	2.73
							42	3.89	5.85	2.79	4.19	1.72	2.59
							44	3.71	5.57	2.65	3.99	1.64	2.46
							46	3.54	5.32	2.53	3.81	1.56	2.35
							48	3.39	5.10	2.42	3.64	1.49	2.25
							50	3.25	4.89	2.32	3.49	1.43	2.15

Properties

Available Strength in Tensile Yielding, kips							Limiting Unbraced Lengths, ft					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		L_p	L_r	L_p	L_r	L_p	L_r
186	280	161	242	124	186		2.62	7.98	2.56	7.46	3.17	6.98
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips							Area, in.^{2}					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		4.44		3.84		2.96	
150	225	130	194	99.9	150		Moment of Inertia, in.^{4}					
Available Strength in Shear, kips							I_x	I_y	I_x	I_y	I_x	I_y
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$		48.0	3.41	39.6	2.73	30.8	2.09
55.6	83.5	51.5	77.2	37.6	56.3		r_y, in.					
Available Strength in Flexure about Y-Y Axis, kip-ft							0.876		0.843		0.841	
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$		r_x/r_y					
9.33	14.0	7.48	11.2	5.32	8.00		3.76		3.81		3.83	

[c] Shape is slender for compression with F_y = 70 ksi.

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

W6

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W6×						Shape	W6×					
25		20		15^c		lb/ft	25		20^f		15^f	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
308	462	246	370	185	278	0	66.0	99.2	51.6	77.5	33.6	50.4
245	368	194	292	144	217	6	63.2	95.0	49.6	74.5	33.6	50.4
225	338	178	268	132	198	7	61.3	92.1	47.7	71.7	33.4	50.2
205	307	162	243	119	178	8	59.3	89.2	45.8	68.9	31.8	47.8
184	276	145	218	105	158	9	57.4	86.3	44.0	66.1	30.1	45.3
163	244	128	192	92.1	138	10	55.5	83.4	42.1	63.3	28.5	42.9
142	214	111	167	79.5	119	11	53.6	80.5	40.2	60.5	26.9	40.4
123	185	95.7	144	67.5	101	12	51.6	77.6	38.4	57.7	25.3	38.0
105	157	81.6	123	57.5	86.5	13	49.7	74.7	36.5	54.9	23.6	35.4
90.3	136	70.3	106	49.6	74.6	14	47.8	71.8	34.6	52.1	21.2	31.9
78.7	118	61.3	92.1	43.2	64.9	15	45.8	68.9	32.8	49.3	19.3	29
69.1	104	53.9	80.9	38.0	57.1	16	43.9	66.0	30.2	45.4	17.6	26.5
61.2	92.1	47.7	71.7	33.6	50.6	17	42.0	63.1	28.0	42.1	16.3	24.5
54.6	82.1	42.5	64.0	30.0	45.1	18	39.7	59.7	26.1	39.3	15.1	22.7
49.0	73.7	38.2	57.4	26.9	40.5	19	37.4	56.2	24.5	36.8	14.1	21.2
44.3	66.5	34.5	51.8	24.3	36.5	20	35.3	53.0	23.1	34.7	13.2	19.9
36.6	55.0	28.5	42.8	20.1	30.2	22	31.7	47.6	20.6	31.0	11.7	17.7
30.7	46.2	23.9	36.0	16.9	25.4	24	28.8	43.3	18.7	28.1	10.6	15.9
						26	26.4	39.7	17.1	25.7	9.62	14.5
						28	24.4	36.7	15.8	23.7	8.83	13.3
						30	22.7	34.1	14.6	22.0	8.17	12.3
						32	21.2	31.8	13.6	20.5	7.59	11.4
						34	19.9	29.9	12.8	19.2	7.10	10.7
						36	18.7	28.1	12.0	18.1	6.67	10.0
						38	17.7	26.6	11.4	17.1	6.29	9.45
						40	16.8	25.2	10.8	16.2	5.95	8.94
						42	15.9	24.0	10.2	15.4	5.64	8.48
						44	15.2	22.8	9.73	14.6	5.37	8.07
						46	14.5	21.8	9.30	14.0	5.12	7.70
						48	13.9	20.9	8.89	13.4	4.90	7.36
						50	13.3	20.0	8.53	12.8	4.69	7.05

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
308	462	246	370	186	279
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
248	372	198	297	150	224
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
57.2	85.7	45.1	67.7	38.6	57.9
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
29.9	44.9	23.0	34.5	13.9	20.9

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
4.54	17.6	4.91	15.0	6.90	12.9
Area, in.^{2}					
7.34		5.87		4.43	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
53.4	17.1	41.4	13.3	29.1	9.32
r_y, in.					
1.52		1.50		1.45	
r_x/r_y					
1.78		1.77		1.77	

[c] Shape is slender for compression with F_y = 70 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

W6

Table IV-6B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 70 ksi
F_u = 90 ksi

Design: Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending

Shape	W6×						W6×					
lb/ft	16		12		9		16		12		9^f	
	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Design	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	199	299	149	224	112	169	40.9	61.4	29.0	43.6	20.8	31.3
6	113	169	79.3	119	58.8	88.3	34.4	51.7	22.9	34.5	16.5	24.8
7	91.8	138	63.2	94.9	46.5	69.9	32.3	48.6	21.1	31.7	14.9	22.4
8	72.3	109	48.8	73.3	35.8	53.8	30.3	45.5	19.2	28.9	13.1	19.6
9	57.1	85.8	38.6	57.9	28.3	42.5	28.2	42.4	17.1	25.7	10.9	16.4
10	46.3	69.5	31.2	46.9	22.9	34.4	26.1	39.3	14.8	22.3	9.36	14.1
11	38.2	57.5	25.8	38.8	18.9	28.5	23.8	35.7	13.1	19.7	8.17	12.3
12	32.1	48.3	21.7	32.6	15.9	23.9	21.4	32.2	11.7	17.6	7.25	10.9
13	27.4	41.1	18.5	27.8	13.6	20.4	19.5	29.4	10.6	15.9	6.52	9.80
14	23.6	35.5	15.9	23.9	11.7	17.6	17.9	27.0	9.70	14.6	5.92	8.90
15	20.6	30.9	13.9	20.9	10.2	15.3	16.6	24.9	8.94	13.4	5.42	8.15
16	18.1	27.2					15.4	23.2	8.29	12.5	5.01	7.52
17							14.5	21.7	7.73	11.6	4.65	6.98
18							13.6	20.4	7.24	10.9	4.34	6.52
19							12.8	19.2	6.82	10.2	4.07	6.12
20							12.1	18.2	6.44	9.68	3.83	5.76
22							10.9	16.5	5.80	8.71	3.43	5.16
24							9.99	15.0	5.28	7.93	3.11	4.68
26							9.19	13.8	4.84	7.28	2.85	4.28
28							8.50	12.8	4.48	6.73	2.63	3.95
30							7.92	11.9	4.16	6.26	2.44	3.66
32							7.41	11.1	3.89	5.85	2.27	3.42
34							6.96	10.5	3.65	5.49	2.13	3.20
36							6.57	9.87	3.44	5.17	2.00	3.01
38							6.21	9.34	3.25	4.89	1.89	2.85
40							5.90	8.86	3.09	4.64	1.79	2.70
42							5.61	8.43	2.94	4.41	1.71	2.56
44							5.35	8.04	2.80	4.21	1.62	2.44
46							5.12	7.69	2.67	4.02	1.55	2.33
48							4.90	7.36	2.56	3.85	1.48	2.23
50							4.70	7.07	2.46	3.69	1.42	2.14

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
199	299	149	224	112	169
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
160	240	120	180	90.5	136
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
45.7	68.6	38.8	58.2	28.1	42.1
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
11.8	17.8	8.10	12.2	5.64	8.47

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
2.89	10.6	2.74	8.71	3.28	7.80
Area, in.^{2}					
4.74		3.55		2.68	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
32.1	4.43	22.1	2.99	16.4	2.20
r_y, in.					
0.967		0.918		0.905	
r_x/r_y					
2.69		2.71		2.73	

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W6–W5

F_y = 70 ksi
F_u = 90 ksi

W6×		W5×				Shape	W6×		W5×			
8.5		19		16		lb/ft	8.5^f		19		16	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
106	159	233	350	197	297	0	18.6	28.0	40.5	60.9	33.6	50.6
54.1	81.2	169	253	141	212	6	14.9	22.5	37.9	57.0	31.0	46.6
42.4	63.8	150	225	125	188	7	13.4	20.2	36.7	55.2	29.9	44.9
32.6	48.9	131	197	109	164	8	11.5	17.3	35.5	53.3	28.7	43.1
25.7	38.7	112	169	93.1	140	9	9.63	14.5	34.3	51.5	27.5	41.4
20.8	31.3	94.8	142	78.0	117	10	8.23	12.4	33.1	49.7	26.3	39.6
17.2	25.9	78.6	118	64.5	97.0	11	7.18	10.8	31.9	47.9	25.2	37.8
14.5	21.7	66.0	99.2	54.2	81.5	12	6.36	9.56	30.7	46.1	24.0	36.1
12.3	18.5	56.3	84.6	46.2	69.4	13	5.71	8.58	29.5	44.3	22.8	34.3
10.6	16.0	48.5	72.9	39.8	59.9	14	5.18	7.78	28.3	42.5	21.7	32.6
		42.3	63.5	34.7	52.1	15	4.74	7.12	27.1	40.7	20.3	30.6
		37.1	55.8	30.5	45.8	16	4.37	6.56	25.9	38.9	18.9	28.4
		32.9	49.5	27.0	40.6	17	4.05	6.09	24.6	36.9	17.7	26.5
		29.3	44.1	24.1	36.2	18	3.78	5.68	23.1	34.7	16.6	24.9
		26.3	39.6	21.6	32.5	19	3.54	5.32	21.8	32.7	15.6	23.5
		23.8	35.7	19.5	29.3	20	3.33	5.01	20.6	31.0	14.8	22.2
						22	2.98	4.49	18.6	28.0	13.3	20.0
						24	2.70	4.06	17.0	25.6	12.1	18.3
						26	2.47	3.71	15.6	23.5	11.2	16.8
						28	2.28	3.42	14.5	21.8	10.3	15.5
						30	2.11	3.17	13.5	20.3	9.61	14.4
						32	1.97	2.96	12.6	19.0	8.99	13.5
						34	1.85	2.77	11.9	17.8	8.44	12.7
						36	1.74	2.61	11.2	16.8	7.96	12.0
						38	1.64	2.46	10.6	15.9	7.53	11.3
						40	1.55	2.34	10.0	15.1	7.14	10.7
						42	1.48	2.22	9.56	14.4	6.80	10.2
						44	1.41	2.11	9.12	13.7	6.48	9.74
						46	1.34	2.02	8.72	13.1	6.19	9.31
						48	1.28	1.93	8.35	12.5	5.93	8.92
						50	1.23	1.85	8.01	12.0	5.69	8.55

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
106	159	233	350	197	297
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
85.1	128	188	281	159	238
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
27.8	41.6	38.9	58.4	33.7	50.5
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
4.89	7.35	19.3	29.0	16.0	24.0

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
3.59	7.62	3.82	16.8	3.76	14.7
Area, in.^{2}					
2.52		5.56		4.71	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
14.9	1.99	26.3	9.13	21.4	7.51
r_y, in.					
0.890		1.28		1.26	
r_x/r_y					
2.73		1.70		1.69	

[f] Shape exceeds compact limit for flexure with F_y = 70 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A913 material availability before specifying.

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W4

Table IV-6B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 70 ksi
F_u = 90 ksi

W4×		Shape		W4×	
13		lb/ft		13	
P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips				Available Flexural Strength, kip-ft	
ASD	LRFD			ASD	LRFD
161	241	Effective Length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	21.9	33.0
94.4	142		6	19.6	29.4
78.0	117		7	18.8	28.3
62.5	93.9		8	18.0	27.1
49.4	74.2		9	17.2	25.9
40.0	60.1		10	16.5	24.7
33.0	49.7		11	15.7	23.6
27.8	41.7		12	14.9	22.4
23.7	35.6		13	14.1	21.2
20.4	30.7		14	13.3	20.0
17.8	26.7		15	12.4	18.6
15.6	23.5		16	11.6	17.4
			17	10.8	16.3
			18	10.2	15.3
			19	9.64	14.5
			20	9.14	13.7
			22	8.28	12.4
			24	7.57	11.4
			26	6.97	10.5
			28	6.46	9.72
			30	6.03	9.06
			32	5.64	8.48
			34	5.31	7.98
			36	5.01	7.53
			38	4.74	7.13
			40	4.50	6.77
			42	4.29	6.44
			44	4.09	6.15
			46	3.91	5.88
			48	3.75	5.63
			50	3.59	5.40

Properties

Available Strength in Tensile Yielding, kips	
P_n/Ω_t	$\phi_t P_n$
161	241
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips	
P_n/Ω_t	$\phi_t P_n$
129	194
Available Strength in Shear, kips	
V_n/Ω_v	$\phi_v V_n$
32.6	48.9
Available Strength in Flexure about Y-Y Axis, kip-ft	
M_{ny}/Ω_b	$\phi_b M_{ny}$
10.2	15.3

Limiting Unbraced Lengths, ft	
L_p	L_r
2.99	14.0
Area, in.^{2}	
3.83	
Moment of Inertia, in.^{4}	
I_x	I_y
11.3	3.86
r_y, in.	
1.00	
r_x/r_y	
1.72	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A913 material availability before specifying.

Table IV-7A
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS24–HSS20

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS24x12x						HSS20x12x			
		¾^{a}		⅝$^{a, c}$		½$^{a, c}$		¾		⅝^{a}	
t_{des}, in.		0.750		0.625		0.500		0.750		0.625	
lb/ft		171		144		117		151		127	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	1510	2260	1250	1880	915	1380	1330	1990	1120	1680
	1	1510	2260	1250	1880	915	1370	1330	1990	1120	1680
	2	1500	2260	1250	1880	914	1370	1320	1990	1120	1680
	3	1500	2250	1250	1880	912	1370	1320	1990	1120	1680
	4	1500	2250	1250	1870	910	1370	1320	1980	1110	1670
	5	1490	2240	1240	1870	908	1360	1310	1970	1110	1660
	6	1480	2230	1240	1860	904	1360	1310	1960	1100	1660
	7	1470	2220	1230	1850	901	1350	1300	1950	1100	1650
	8	1470	2200	1230	1840	896	1350	1290	1940	1090	1640
	9	1450	2190	1220	1830	892	1340	1280	1920	1080	1620
	10	1440	2170	1210	1820	886	1330	1270	1910	1070	1610
	11	1430	2150	1200	1810	880	1320	1260	1890	1060	1600
	12	1420	2130	1190	1800	874	1310	1240	1870	1050	1580
	13	1400	2110	1180	1780	867	1300	1230	1850	1040	1560
	14	1390	2080	1170	1760	860	1290	1220	1830	1030	1550
	15	1370	2060	1160	1740	852	1280	1200	1800	1020	1530
	16	1350	2030	1140	1710	843	1270	1180	1780	1000	1510
	17	1330	2000	1130	1690	834	1250	1170	1750	987	1480
	18	1310	1970	1110	1670	825	1240	1150	1730	973	1460
	19	1290	1940	1090	1640	816	1230	1130	1700	957	1440
	20	1270	1910	1070	1610	805	1210	1110	1670	941	1410
	22	1230	1840	1040	1560	784	1180	1070	1610	907	1360
	24	1180	1770	998	1500	761	1140	1030	1540	872	1310
	26	1130	1700	957	1440	737	1110	982	1480	835	1250
	28	1080	1620	915	1380	712	1070	936	1410	796	1200
	30	1030	1540	872	1310	686	1030	889	1340	757	1140
	32	973	1460	828	1240	659	991	842	1270	717	1080
	34	920	1380	783	1180	631	949	794	1190	677	1020
	36	867	1300	739	1110	603	907	746	1120	637	958
	38	814	1220	694	1040	570	857	699	1050	598	898
	40	761	1140	651	978	535	804	652	980	558	839
	42	710	1070	607	913	500	752	606	911	520	781
	44	660	992	565	850	466	700	562	844	482	725
	46	611	918	524	788	433	651	518	779	446	670
	48	563	846	484	728	401	602	476	715	410	616
	50	519	780	446	671	369	555	439	659	378	568
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		1510	2260	1270	1910	1030	1550	1330	1990	1120	1680
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		1230	1840	1030	1550	839	1260	1080	1620	913	1370
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		587	883	496	746	404	608	480	721	406	611
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		263	395	227	341	189	284	263	395	227	341
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		953	1430	811	1220	664	998	716	1080	611	919
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		572	859	433	651	317	477	504	758	418	628
Properties											
Area, in.^{2}		50.3		42.4		34.4		44.3		37.4	
r_y, in.		4.97		5.02		5.07		4.87		4.92	
r_x/r_y		1.71		1.71		1.71		1.49		1.49	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS20

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS20x12x						HSS20x8x			
		½[a, c]		⅜[a, b, c]		5⁄16[a, b, c]		⅝[a]		½[a, c]	
t_{des}, in.		0.500		0.375		0.313		0.625		0.500	
lb/ft		103		78.5		65.9		110		89.7	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	884	1330	589	885	450	677	970	1460	764	1150
	1	883	1330	588	884	450	677	969	1460	763	1150
	2	883	1330	588	883	450	676	966	1450	762	1150
	3	881	1320	587	882	449	675	962	1450	759	1140
	4	879	1320	585	880	448	674	955	1440	755	1140
	5	876	1320	584	877	447	672	947	1420	750	1130
	6	873	1310	582	874	446	670	937	1410	745	1120
	7	869	1310	579	870	444	668	926	1390	738	1110
	8	865	1300	576	866	443	665	913	1370	730	1100
	9	860	1290	573	861	441	662	898	1350	721	1080
	10	854	1280	569	856	438	659	882	1330	711	1070
	11	848	1270	565	850	436	655	864	1300	700	1050
	12	841	1260	561	843	433	651	845	1270	689	1040
	13	834	1250	557	837	430	647	825	1240	676	1020
	14	826	1240	552	829	427	642	804	1210	659	991
	15	818	1230	546	821	423	636	782	1180	642	964
	16	809	1220	541	813	419	630	760	1140	623	937
	17	800	1200	535	804	414	623	736	1110	605	909
	18	791	1190	529	795	410	616	712	1070	585	880
	19	780	1170	522	785	405	609	687	1030	566	850
	20	768	1150	516	775	400	601	662	995	546	820
	22	741	1110	502	754	389	585	611	918	505	758
	24	712	1070	487	731	377	567	560	841	463	696
	26	682	1030	471	708	365	549	509	764	422	635
	28	652	979	454	683	353	530	459	689	382	574
	30	620	932	437	657	340	511	411	617	343	516
	32	588	884	419	630	326	490	364	547	306	459
	34	556	836	401	603	313	470	322	485	271	407
	36	524	787	383	576	299	449	288	432	241	363
	38	492	739	365	548	285	428	258	388	217	326
	40	460	692	346	520	270	406	233	350	196	294
	42	429	645	328	492	256	385	211	318	177	267
	44	399	599	308	463	242	364	193	289	162	243
	46	369	555	286	429	228	343	176	265	148	222
	48	340	511	264	396	215	322	162	243	136	204
	50	314	471	243	365	201	302	149	224	125	188
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		910	1370	692	1040	581	873	970	1460	790	1190
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		741	1110	562	843	475	712	790	1180	644	965
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		332	500	255	383	210	315	406	611	332	500
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		189	284	147	221	125	188	138	207	117	176
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		501	754	367	551	265	398	492	739	404	608
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		298	448	199	300	153	231	250	376	178	267
Properties											
Area, in.^{2}		30.4		23.1		19.4		32.4		26.4	
r_y, in.		4.97		5.02		5.05		3.32		3.37	
r_x/r_y		1.48		1.48		1.48		2.07		2.06	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

HSS20

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS20x8x				HSS20x4x					
		3/8[a, c]		5/16[a, b, c]		1/2[a, c]		3/8[a, c]		5/16[a, b, c]	
t_{des}, in.		0.375		0.313		0.500		0.375		0.313	
lb/ft		68.3		57.4		76.1		58.1		48.9	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	499	750	381	572	644	968	409	615	306	460
	1	498	749	380	572	642	966	408	613	305	458
	2	497	748	380	571	637	958	405	608	303	455
	3	496	745	378	569	628	944	400	601	299	450
	4	493	742	377	566	616	926	393	591	294	442
	5	490	737	374	563	601	903	384	578	288	433
	6	487	732	372	559	582	875	374	562	280	422
	7	483	725	369	554	556	836	362	544	272	409
	8	478	718	365	548	525	789	349	524	262	394
	9	472	710	361	542	492	740	334	502	252	378
	10	466	701	356	536	458	688	318	478	240	361
	11	460	691	351	528	422	635	302	454	228	343
	12	453	680	346	520	387	581	285	428	216	325
	13	445	669	340	512	352	528	267	401	203	306
	14	437	657	334	503	317	477	249	374	190	286
	15	429	644	328	493	284	427	230	345	177	266
	16	420	631	322	483	252	378	206	309	164	246
	17	410	617	315	473	223	335	183	275	151	226
	18	401	602	307	462	199	299	163	245	138	208
	19	391	587	300	451	178	268	146	220	126	189
	20	381	572	292	439	161	242	132	198	114	171
	22	359	540	277	416	133	200	109	164	94.0	141
	24	338	507	260	391	112	168	91.7	138	79.0	119
	26	315	474	244	366	95.3	143	78.1	117	67.3	101
	28	292	440	227	341			67.4	101	58.0	87.2
	30	269	404	210	315						
	32	241	362	193	290						
	34	214	321	176	265						
	36	190	286	161	243						
	38	171	257	146	220						
	40	154	232	132	198						
	42	140	210	120	180						
	44	127	192	109	164						
	46	117	175	99.8	150						
	48	107	161	91.7	138						
	50	98.7	148	84.5	127						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		602	905	506	761	671	1010	512	770	431	648
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		491	736	413	619	546	819	416	624	351	527
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		255	383	210	315	332	500	255	383	210	315
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		92.7	139	79.4	119	44.9	67.5	38.8	58.3	34.4	51.7
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		312	469	263	395	307	461	238	357	200	301
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		119	178	90.9	137	72.7	109	49.4	74.3	37.8	56.9
Properties											
Area, in.^{2}		20.1		16.9		22.4		17.1		14.4	
r_y, in.		3.43		3.46		1.66		1.72		1.74	
r_x/r_y		2.04		2.04		3.80		3.72		3.70	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS20–HSS18

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape		HSS20x4x		HSS18x6x							
		¼$^{a, b, c}$		⅝		½^{a}		⅜$^{a, c}$		5⁄16$^{a, c}$	
t_{des}, in.		0.250		0.625		0.500		0.375		0.313	
lb/ft		39.4		93.3		76.1		58.1		48.9	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	212	319	820	1230	671	1010	445	669	338	509
	1	212	319	819	1230	670	1010	445	668	338	508
	2	210	316	815	1220	666	1000	443	666	337	506
	3	208	313	808	1210	661	993	440	662	335	504
	4	205	308	798	1200	653	982	437	656	332	499
	5	200	301	785	1180	643	967	432	649	329	494
	6	195	294	771	1160	632	950	427	641	325	488
	7	190	285	753	1130	618	929	420	631	320	481
	8	183	275	734	1100	603	907	413	620	315	473
	9	176	265	713	1070	586	881	404	608	309	464
	10	169	253	689	1040	568	854	395	594	302	454
	11	161	241	665	999	549	825	386	579	295	443
	12	152	229	639	960	528	794	375	564	287	431
	13	144	216	611	919	507	762	364	547	279	419
	14	135	203	583	877	485	728	352	530	270	406
	15	126	190	555	834	462	694	340	512	262	393
	16	117	177	525	790	439	659	328	493	252	379
	17	109	163	496	746	415	624	315	473	243	365
	18	100	151	467	702	392	589	302	454	233	351
	19	92.8	139	438	658	369	554	288	433	223	336
	20	86.3	130	409	615	346	519	271	407	214	321
	22	75.2	113	353	531	301	452	237	356	194	291
	24	65.9	99.0	300	452	258	387	204	307	174	261
	26	56.1	84.3	256	385	220	330	174	262	150	226
	28	48.4	72.7	221	332	189	285	150	226	130	195
	30			192	289	165	248	131	197	113	170
	32			169	254	145	218	115	173	99.2	149
	34			150	225	128	193	102	153	87.9	132
	36			134	201	115	172	91.0	137	78.4	118
	38			120	180	103	155	81.6	123	70.4	106
	40			108	163	92.8	139	73.7	111	63.5	95.4
	42					84.2	127	66.8	100	57.6	86.6
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		347	522	820	1230	671	1010	512	770	431	648
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		283	424	670	1000	546	819	416	624	351	527
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		129	193	362	543	296	446	228	342	192	289
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		29.2	43.9	92.7	139	80.8	122	65.7	98.8	56.9	85.5
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		154	232	359	540	297	446	230	346	195	293
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		26.8	40.3	161	242	121	182	80.3	121	61.5	92.5
Properties											
Area, in.^{2}		11.6		27.4		22.4		17.1		14.4	
r_y, in.		1.77		2.46		2.52		2.57		2.60	
r_x/r_y		3.67		2.43		2.40		2.38		2.37	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

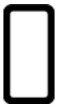
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.



HSS18–HSS16

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS18x6x		HSS16x12x							
		¼[a, b, c]		¾		⅝		½[a]		⅜[a, b, c]	
t_{des}, in.		0.250		0.750		0.625		0.500		0.375	
lb/ft		39.4		130		110		89.7		68.3	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	240	361	1150	1720	970	1460	790	1190	569	855
	1	240	360	1150	1720	970	1460	790	1190	569	855
	2	239	359	1140	1720	968	1460	789	1190	568	854
	3	238	357	1140	1720	966	1450	787	1180	567	852
	4	236	354	1140	1710	963	1450	785	1180	566	850
	5	233	351	1130	1700	959	1440	782	1170	564	847
	6	231	346	1130	1690	954	1430	778	1170	561	844
	7	227	341	1120	1680	948	1430	773	1160	559	840
	8	223	336	1110	1670	942	1420	768	1150	556	835
	9	219	330	1100	1660	935	1400	762	1150	553	830
	10	215	323	1090	1640	927	1390	756	1140	549	825
	11	210	315	1080	1630	918	1380	749	1130	545	819
	12	204	307	1070	1610	908	1360	741	1110	540	812
	13	199	299	1060	1590	898	1350	733	1100	535	805
	14	193	290	1050	1570	887	1330	724	1090	530	797
	15	187	281	1030	1550	875	1310	714	1070	525	789
	16	180	271	1020	1530	863	1300	705	1060	519	780
	17	174	261	1000	1500	850	1280	694	1040	513	771
	18	167	251	985	1480	836	1260	683	1030	506	761
	19	160	241	968	1450	822	1240	672	1010	500	751
	20	154	231	950	1430	807	1210	660	993	493	740
	22	140	210	913	1370	777	1170	636	956	478	718
	24	126	190	874	1310	745	1120	610	917	462	695
	26	113	169	834	1250	711	1070	583	877	446	670
	28	101	151	793	1190	677	1020	556	835	427	641
	30	90.7	136	751	1130	642	965	527	793	406	610
	32	81.8	123	708	1060	606	911	499	750	384	577
	34	72.4	109	666	1000	571	858	470	707	362	545
	36	64.6	97.1	623	937	535	804	441	663	341	512
	38	58.0	87.2	581	874	500	752	413	621	319	480
	40	52.3	78.7	540	812	466	700	385	579	298	448
	42	47.5	71.4	500	751	432	649	358	538	278	417
	44			461	693	399	600	331	498	257	387
	46			423	635	367	551	305	459	238	358
	48			388	583	337	506	280	421	219	329
	50			358	538	310	467	258	388	201	303
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		347	522	1150	1720	970	1460	790	1190	602	905
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		283	424	933	1400	790	1180	644	965	491	736
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		134	201	372	559	317	476	260	392	201	302
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		47.2	70.9	263	395	227	341	189	284	147	221
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		154	231	511	769	437	656	359	540	265	399
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		43.9	65.9	419	630	359	540	287	431	186	280
Properties											
Area, in.^{2}		11.6		38.3		32.4		26.4		20.1	
r_y, in.		2.63		4.73		4.79		4.84		4.90	
r_x/r_y		2.36		1.25		1.25		1.25		1.24	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.
[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.
Confirm ASTM A1085 material availability before specifying.

HSS16

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS16x12x		HSS16x8x							
		5/16$^{a, b, c}$		5/8		1/2^{a}		3/8$^{a, c}$		5/16$^{a, c}$	
t_{des}, in.		0.313		0.625		0.500		0.375		0.313	
lb/ft		57.4		93.3		76.1		58.1		48.9	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	440	661	820	1230	671	1010	479	720	370	556
	1	439	660	820	1230	670	1010	479	719	370	556
	2	439	660	817	1230	668	1000	478	718	369	554
	3	438	659	813	1220	665	999	476	715	368	552
	4	438	658	807	1210	660	993	473	712	366	550
	5	436	656	800	1200	655	984	470	707	363	546
	6	435	654	791	1190	648	974	467	701	361	542
	7	433	651	781	1170	640	961	462	695	357	537
	8	432	649	770	1160	630	948	457	687	353	531
	9	429	645	757	1140	620	932	451	678	349	525
	10	427	642	743	1120	609	915	445	669	345	518
	11	424	638	727	1090	597	897	438	659	339	510
	12	422	634	711	1070	583	877	431	648	334	502
	13	419	629	693	1040	570	856	423	636	328	493
	14	415	624	675	1010	555	834	415	624	322	484
	15	411	618	656	985	540	811	406	610	315	474
	16	406	611	636	955	524	787	397	596	308	464
	17	402	604	615	924	507	762	387	582	301	453
	18	397	596	594	893	490	737	377	567	294	442
	19	392	589	572	860	473	711	366	550	286	430
	20	386	581	551	828	456	685	353	530	278	418
	22	375	564	506	761	420	631	326	490	262	394
	24	363	546	462	694	384	578	299	450	245	369
	26	351	527	418	629	349	524	273	410	228	343
	28	338	507	375	564	314	472	246	370	210	316
	30	324	487	334	503	281	422	221	332	189	284
	32	310	466	295	443	249	374	197	296	169	254
	34	296	445	261	393	220	331	174	262	149	225
	36	282	423	233	350	196	295	155	234	133	200
	38	267	401	209	314	176	265	140	210	120	180
	40	253	380	189	284	159	239	126	189	108	162
	42	236	354	171	257	144	217	114	172	97.9	147
	44	219	329	156	235	132	198	104	156	89.2	134
	46	202	304	143	215	120	181	95.2	143	81.6	123
	48	186	280	131	197	111	166	87.5	131	75.0	113
	50	171	258	121	182	102	153	80.6	121	69.1	104
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		506	761	820	1230	671	1010	512	770	431	648
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		413	619	670	1000	546	819	416	624	351	527
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		170	255	317	476	260	392	201	302	170	255
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		125	188	138	207	117	176	92.7	139	79.4	119
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		195	292	342	514	282	424	219	329	185	279
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		145	218	210	315	169	255	111	167	85.8	129
Properties											
Area, in.^{2}		16.9		27.4		22.4		17.1		14.4	
r_y, in.		4.93		3.25		3.30		3.36		3.39	
r_x/r_y		1.24		1.73		1.72		1.71		1.71	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

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Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS16

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS16x8x		HSS16x4x							
		¼[a, b, c]		⅝		½[a]		⅜[a, c]		5⁄16[a, c]	
t_{des}, in.		0.250		0.625		0.500		0.375		0.313	
lb/ft		39.4		76.3		62.5		47.9		40.4	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	267	401	671	1010	551	828	389	585	295	443
	1	267	401	668	1000	549	825	388	583	294	442
	2	266	400	660	991	542	815	385	579	292	439
	3	265	399	646	971	532	799	380	571	288	433
	4	264	397	627	943	517	778	373	560	283	426
	5	262	394	604	908	500	751	364	547	277	416
	6	260	391	577	868	478	719	353	530	269	404
	7	258	388	547	822	455	683	341	512	260	391
	8	255	384	514	772	429	644	327	491	250	376
	9	252	379	479	719	401	603	312	469	240	360
	10	249	374	442	665	372	560	292	439	228	343
	11	245	369	405	609	343	516	270	406	216	325
	12	241	363	368	553	314	471	248	373	203	306
	13	237	356	332	499	284	427	226	340	190	286
	14	233	350	296	446	256	384	205	308	177	266
	15	228	343	263	395	228	343	184	277	160	240
	16	223	335	231	347	202	303	164	247	143	215
	17	218	328	205	307	179	269	145	219	127	191
	18	213	320	182	274	159	240	130	195	113	170
	19	207	312	164	246	143	215	116	175	102	153
	20	202	303	148	222	129	194	105	158	91.9	138
	22	190	286	122	184	107	160	86.8	131	75.9	114
	24	179	268	103	154	89.7	135	73.0	110	63.8	95.9
	26	166	250	87.4	131	76.4	115	62.2	93.5	54.4	81.7
	28	154	232					53.6	80.6	46.9	70.4
	30	142	214								
	32	130	196								
	34	119	178								
	36	108	163								
	38	97.5	147								
	40	88.0	132								
	42	79.8	120								
	44	72.7	109								
	46	66.5	100								
	48	61.1	91.8								
	50	56.3	84.6								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		347	522	671	1010	551	828	422	635	356	536
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		283	424	546	819	449	673	345	517	290	435
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		134	201	317	476	260	392	201	302	170	255
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		65.1	97.9	47.8	71.9	44.9	67.5	38.8	58.3	34.4	51.7
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		143	215	246	369	205	308	160	241	136	205
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		61.7	92.7	85.3	128	70.1	105	46.0	69.2	35.7	53.7
Properties											
Area, in.^{2}		11.6		22.4		18.4		14.1		11.9	
r_y, in.		3.41		1.59		1.64		1.69		1.72	
r_x/r_y		1.71		3.17		3.12		3.08		3.05	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.
[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.
Confirm ASTM A1085 material availability before specifying.

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Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS16

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS16x4x				HSS14x10x					
		¼[a, b, c]		3⁄16[a, b, c]		⅝		½		⅜[a, c]	
t_{des}, in.		0.250		0.188		0.625		0.500		0.375	
lb/ft		32.6		24.7		93.3		76.1		58.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	207	311	132	199	820	1230	671	1010	510	766
	1	206	310	132	198	820	1230	670	1010	510	766
	2	205	308	131	197	818	1230	669	1010	509	765
	3	202	304	130	195	815	1230	667	1000	508	763
	4	199	299	127	191	812	1220	664	997	506	760
	5	195	292	125	187	807	1210	660	992	503	757
	6	189	285	122	183	801	1200	655	985	500	752
	7	184	276	118	177	794	1190	649	976	496	746
	8	177	266	114	171	786	1180	643	967	492	739
	9	170	255	109	164	777	1170	636	956	486	731
	10	162	244	105	157	767	1150	628	944	481	722
	11	154	231	99.5	150	757	1140	620	931	474	713
	12	146	219	94.2	142	745	1120	610	917	467	703
	13	137	206	88.9	134	733	1100	600	902	460	691
	14	128	192	83.4	125	720	1080	590	887	452	680
	15	119	179	77.9	117	706	1060	579	870	444	667
	16	110	166	72.4	109	691	1040	567	852	435	654
	17	101	152	66.9	101	676	1020	555	834	426	641
	18	93.0	140	61.7	92.7	661	993	542	815	417	627
	19	84.9	128	57.1	85.8	645	969	529	796	407	612
	20	76.6	115	53.0	79.7	628	944	516	776	398	597
	22	63.3	95.2	46.1	69.4	594	892	489	734	377	567
	24	53.2	80.0	40.6	61.0	558	839	460	691	356	535
	26	45.3	68.2	35.7	53.6	522	785	431	647	334	502
	28	39.1	58.8	30.8	46.2	486	730	401	603	312	469
	30					450	676	372	559	290	435
	32					414	622	343	516	268	403
	34					379	570	315	473	246	370
	36					345	519	287	431	226	339
	38					312	469	260	391	205	309
	40					282	423	235	353	186	279
	42					256	384	213	320	168	253
	44					233	350	194	292	153	231
	46					213	320	178	267	140	211
	48					196	294	163	245	129	194
	50					180	271	150	226	119	179
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		287	432	218	328	820	1230	671	1010	512	770
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		234	351	178	267	670	1000	546	819	416	624
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		134	201	67.9	102	272	408	225	338	174	261
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		29.2	43.9	23.2	34.9	183	274	153	230	120	180
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		110	166	79.0	119	317	476	262	394	203	306
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		25.5	38.4	16.6	25.0	252	379	209	314	140	211
Properties											
Area, in.^{2}		9.59		7.29		27.4		22.4		17.1	
r_y, in.		1.75		1.78		3.97		4.01		4.08	
r_x/r_y		3.03		3.00		1.30		1.30		1.29	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.
[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.
Confirm ASTM A1085 material availability before specifying.

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Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS14

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS14x10x				HSS14x6x					
		5/16$^{a, b, c}$		1/4$^{a, b, c}$		5/8		1/2		3/8$^{a, c}$	
t_{des}, in.		0.313		0.250		0.625		0.500		0.375	
lb/ft		48.9		39.4		76.3		62.5		47.9	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	399	600	285	429	671	1010	551	828	420	631
	1	399	600	285	429	669	1010	550	827	420	631
	2	399	599	285	428	666	1000	547	822	418	628
	3	398	598	284	427	660	992	542	815	415	624
	4	396	595	284	426	651	979	536	805	411	618
	5	394	593	283	425	641	963	527	793	405	609
	6	392	589	281	423	628	944	517	778	398	597
	7	390	586	280	421	614	922	506	760	389	585
	8	387	581	278	418	597	898	493	741	379	570
	9	383	576	276	415	579	870	478	719	369	554
	10	380	571	274	412	559	841	463	696	357	537
	11	376	564	272	408	539	809	446	671	345	518
	12	371	558	269	405	517	776	429	644	332	499
	13	367	551	267	401	494	742	411	617	318	478
	14	362	543	264	396	470	707	392	589	304	457
	15	356	535	261	392	446	670	372	560	290	436
	16	351	527	257	387	422	634	353	530	275	414
	17	345	518	254	381	397	597	333	501	260	391
	18	339	509	249	375	373	560	314	471	246	369
	19	332	500	245	368	349	524	294	442	231	347
	20	326	490	240	361	325	488	275	413	216	325
	22	312	469	230	346	279	419	237	357	188	283
	24	298	447	220	331	236	354	202	303	161	242
	26	282	424	209	314	201	302	172	258	137	206
	28	264	397	198	298	173	260	148	223	118	178
	30	245	369	187	281	151	227	129	194	103	155
	32	227	341	176	264	133	199	114	171	90.5	136
	34	209	314	165	247	117	177	101	151	80.2	121
	36	191	288	153	230	105	157	89.7	135	71.5	108
	38	175	262	142	214	94.0	141	80.5	121	64.2	96.5
	40	158	237	129	194	84.9	128	72.6	109	58.0	87.1
	42	143	215	117	176						
	44	131	196	107	160						
	46	119	179	97.6	147						
	48	110	165	89.6	135						
	50	101	152	82.6	124						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		431	648	347	522	671	1010	551	828	422	635
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		351	527	283	424	546	819	449	673	345	517
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		147	221	119	180	272	408	225	338	174	261
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		102	153	83.1	125	92.7	139	80.8	122	65.7	98.8
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		165	249	114	172	235	353	195	294	152	229
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		109	165	79.9	120	128	192	107	161	73.2	110
Properties											
Area, in.^{2}		14.4		11.6		22.4		18.4		14.1	
r_y, in.		4.10		4.13		2.41		2.46		2.51	
r_x/r_y		1.30		1.29		1.97		1.96		1.95	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

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Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS14

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape	HSS14x6x						HSS14x4x			
	5⁄16[a, c]		¼[a, c]		3⁄16[a, b, c]		⅝		½	
t_{des}, in.	0.313		0.250		0.188		0.625		0.500	
lb/ft	40.4		32.6		24.7		67.8		55.7	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y										
0	325	488	233	350	153	230	596	895	491	738
1	324	487	232	349	153	230	593	892	489	735
2	323	485	232	348	152	229	586	880	483	726
3	321	482	230	346	151	228	573	862	474	712
4	318	478	228	343	150	226	556	836	460	692
5	315	473	226	339	149	223	535	805	444	668
6	310	466	223	335	147	221	511	768	425	639
7	305	459	219	330	144	217	483	726	403	606
8	300	450	215	324	142	213	453	681	380	571
9	293	441	211	317	139	209	422	634	355	533
10	286	430	206	310	136	204	389	584	329	494
11	279	419	201	302	133	199	355	534	302	454
12	271	407	196	294	129	194	322	484	276	414
13	262	394	190	285	125	188	289	435	249	375
14	253	381	184	276	121	182	258	388	224	336
15	244	367	177	267	117	176	228	342	199	299
16	235	353	171	257	113	170	200	301	175	264
17	222	334	164	247	109	164	177	266	155	234
18	210	316	157	236	104	157	158	238	139	208
19	198	297	150	226	100	150	142	213	124	187
20	185	279	143	215	95.5	143	128	192	112	169
22	162	243	129	194	86.5	130	106	159	92.8	140
24	139	209	115	172	77.6	117	88.9	134	78.0	117
26	119	178	97.8	147	68.8	103	75.7	114	66.5	99.9
28	102	154	84.3	127	61.4	92.3				
30	89.0	134	73.5	110	55.2	83.0				
32	78.3	118	64.6	97.0	49.9	75.0				
34	69.3	104	57.2	86.0	44.5	66.9				
36	61.8	92.9	51.0	76.7	39.7	59.7				
38	55.5	83.4	45.8	68.8	35.6	53.5				
40	50.1	75.3	41.3	62.1	32.1	48.3				
42	45.4	68.3	37.5	56.3	29.2	43.8				
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	356	536	287	432	218	328	596	896	491	738
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	290	435	234	351	178	267	484	726	400	600
Available Strength in Shear about X-X Axis, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	147	221	119	180	75.2	113	272	408	225	338
Available Strength in Shear about Y-Y Axis, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	56.9	85.5	47.2	70.9	36.7	55.2	47.8	71.9	44.9	67.5
Available Strength in Flexure about X-X Axis, kip-ft	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	129	195	106	159	76.8	115	193	290	162	243
Available Strength in Flexure about Y-Y Axis, kip-ft	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	57.2	86.0	41.3	62.1	27.2	40.9	74.9	113	63.6	95.6
Properties										
Area, in.^{2}	11.9		9.59		7.29		19.9		16.4	
r_y, in.	2.54		2.57		2.60		1.57		1.62	
r_x/r_y	1.94		1.93		1.92		2.83		2.79	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

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Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS14–HSS12

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS14x4x								HSS12x10x	
		⅜[a, c]		5⁄16[a, c]		¼[a, c]		3⁄16[a, b, c]		½	
t_{des}, in.		0.375		0.313		0.250		0.188		0.500	
lb/ft		42.8		36.1		29.2		22.2		69.3	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	375	564	286	429	203	305	131	196	611	918
	1	374	562	285	428	202	304	130	196	610	917
	2	371	558	283	425	201	302	129	194	609	916
	3	365	548	279	419	198	298	128	192	607	912
	4	355	534	274	411	195	293	126	189	604	908
	5	344	517	267	402	190	286	123	185	601	903
	6	330	496	260	390	185	278	120	180	596	896
	7	314	472	251	377	179	269	116	174	591	888
	8	297	447	241	362	172	259	112	168	585	879
	9	279	419	230	346	165	248	107	161	578	869
	10	260	390	218	328	157	236	102	154	571	858
	11	240	361	205	309	149	224	97.2	146	563	846
	12	220	331	189	284	140	211	91.9	138	554	833
	13	201	302	173	260	132	198	86.4	130	545	819
	14	182	273	157	236	123	184	80.9	122	535	804
	15	163	245	141	212	114	171	75.3	113	524	788
	16	145	218	126	190	105	157	69.8	105	513	772
	17	128	193	112	168	92.9	140	64.2	96.5	502	755
	18	115	172	99.9	150	82.8	124	59.0	88.7	490	737
	19	103	155	89.6	135	74.3	112	54.5	81.9	478	719
	20	92.8	139	80.9	122	67.1	101	50.5	75.9	466	700
	22	76.7	115	66.8	100	55.4	83.3	43.7	65.7	440	661
	24	64.4	96.9	56.2	84.4	46.6	70.0	36.7	55.2	413	621
	26	54.9	82.5	47.9	71.9	39.7	59.7	31.3	47.0	386	580
	28	47.3	71.2	41.3	62.0	34.2	51.4	27.0	40.5	359	539
	30									332	499
	32									305	458
	34									279	419
	36									254	381
	38									229	344
	40									207	311
	42									187	282
	44									171	257
	46									156	235
	48									143	216
	50									132	199
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		377	567	317	477	257	387	196	294	611	918
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		307	461	258	388	209	314	160	239	497	746
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		174	261	147	221	119	180	75.2	113	189	284
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		38.8	58.3	34.4	51.7	29.2	43.9	23.2	34.9	153	230
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		127	191	108	163	88.3	133	65.3	98.2	209	315
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		43.6	65.5	34.2	51.4	24.7	37.2	16.2	24.4	185	278
Properties											
Area, in.^{2}		12.6		10.6		8.59		6.54		20.4	
r_y, in.		1.68		1.71		1.73		1.76		3.94	
r_x/r_y		2.74		2.73		2.71		2.69		1.15	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

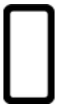
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.



HSS12

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS12x10x						HSS12x8x			
		⅜[a]		5⁄16[a, b, c]		¼[a, b, c]		⅝		½	
t_{des}, in.		0.375		0.313		0.250		0.625		0.500	
lb/ft		53.0		44.6		36.0		76.3		62.5	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	467	702	387	582	280	421	671	1010	551	828
	1	467	702	387	581	280	421	670	1010	550	827
	2	466	700	386	580	280	420	668	1000	549	825
	3	464	698	385	579	279	419	664	998	546	820
	4	462	695	384	577	278	418	659	991	542	814
	5	459	691	382	574	277	416	653	981	537	807
	6	456	686	380	570	276	414	645	970	531	798
	7	452	680	377	567	274	412	636	957	524	787
	8	448	673	374	562	272	410	626	941	516	775
	9	443	666	370	557	270	407	615	924	507	762
	10	437	657	367	551	268	403	603	906	497	747
	11	431	648	363	545	266	400	589	886	486	731
	12	425	639	357	537	263	396	575	864	475	714
	13	418	628	352	528	260	391	560	842	463	696
	14	411	617	345	519	257	387	544	818	450	677
	15	403	605	339	509	254	382	527	793	437	657
	16	395	593	332	499	251	377	510	767	423	636
	17	386	580	325	489	247	371	493	740	409	615
	18	377	567	318	478	242	364	475	713	395	593
	19	368	554	310	466	238	357	456	686	380	571
	20	359	540	303	455	233	350	438	658	365	549
	22	340	511	287	431	223	335	400	601	335	503
	24	320	481	270	406	212	319	363	545	305	458
	26	299	450	253	380	201	302	326	490	275	413
	28	279	419	236	355	190	285	290	436	246	370
	30	258	388	219	329	178	268	256	385	218	328
	32	238	358	202	304	164	247	225	338	192	289
	34	218	328	185	279	151	227	199	300	170	256
	36	199	299	169	254	138	208	178	267	152	228
	38	180	271	154	231	126	189	160	240	136	205
	40	163	245	139	209	113	170	144	217	123	185
	42	148	222	126	189	103	155	131	196	111	168
	44	135	202	115	172	93.7	141	119	179	102	153
	46	123	185	105	158	85.8	129	109	164	92.9	140
	48	113	170	96.4	145	78.8	118	100	150	85.4	128
	50	104	157	88.8	134	72.6	109	92.2	139	78.7	118
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		467	702	392	590	317	477	671	1010	551	828
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		380	570	319	479	258	388	546	819	449	673
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		147	221	125	188	101	153	227	341	189	284
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		120	180	102	153	83.1	125	138	207	117	176
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		163	245	133	199	92.4	139	217	327	181	272
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		138	208	104	156	76.4	115	164	246	136	205
Properties											
Area, in.^{2}		15.6		13.1		10.6		22.4		18.4	
r_y, in.		4.00		4.03		4.05		3.14		3.20	
r_x/r_y		1.15		1.15		1.15		1.38		1.37	

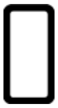
[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.



HSS12

Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape		HSS12x8x								HSS12x6x	
		3⁄8[a]		5⁄16[a, c]		1⁄4[a, b, c]		3⁄16[a, b, c]		5⁄8	
t_{des}, in.		0.375		0.313		0.250		0.188		0.625	
lb/ft		47.9		40.4		32.6		24.7		67.8	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	422	634	351	528	257	387	165	248	596	895
	1	422	634	351	527	257	386	165	248	595	894
	2	420	632	350	526	257	386	165	248	591	889
	3	418	629	349	524	256	384	164	247	586	881
	4	415	624	347	521	254	382	164	246	578	869
	5	412	619	344	517	252	379	163	245	569	855
	6	407	612	341	513	250	376	162	243	557	837
	7	402	604	338	507	248	372	160	241	544	817
	8	396	595	334	501	245	368	159	239	528	794
	9	389	585	329	495	242	363	157	237	512	769
	10	382	574	323	486	238	358	156	234	494	742
	11	374	562	316	476	234	352	154	231	475	714
	12	366	550	309	465	230	346	152	228	455	684
	13	357	536	302	454	226	340	149	224	434	652
	14	347	522	294	442	221	333	147	221	413	620
	15	337	507	286	430	216	325	144	217	391	587
	16	327	492	277	417	211	317	141	213	369	554
	17	316	476	269	404	206	309	139	208	347	521
	18	306	459	259	390	200	301	136	204	325	488
	19	295	443	250	376	195	293	132	198	303	455
	20	283	426	241	362	189	284	128	193	281	423
	22	261	392	222	333	177	266	120	181	240	361
	24	238	357	203	305	165	247	112	169	203	304
	26	215	323	184	276	150	225	104	156	173	259
	28	193	290	165	249	135	203	95.9	144	149	224
	30	172	259	148	222	121	182	87.8	132	130	195
	32	152	228	131	196	107	161	79.8	120	114	171
	34	134	202	116	174	94.9	143	72.3	109	101	152
	36	120	180	103	155	84.6	127	65.5	98.4	90.0	135
	38	108	162	92.5	139	75.9	114	58.8	88.4	80.8	121
	40	97.2	146	83.5	126	68.5	103	53.1	79.7		
	42	88.1	132	75.8	114	62.2	93.4	48.1	72.3		
	44	80.3	121	69.0	104	56.6	85.1	43.8	65.9		
	46	73.5	110	63.2	94.9	51.8	77.9	40.1	60.3		
	48	67.5	101	58.0	87.2	47.6	71.5	36.8	55.4		
	50	62.2	93.5	53.5	80.3	43.9	65.9	34.0	51.0		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		422	635	356	536	287	432	218	328	596	896
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		345	517	290	435	234	351	178	267	484	726
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		147	221	125	188	101	153	75.3	113	227	341
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		92.7	139	79.4	119	65.1	97.9	50.3	75.5	92.7	139
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		141	212	120	180	93.3	140	59.5	89.4	182	273
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		103	155	77.9	117	57.1	85.8	38.4	57.6	111	167
Properties											
Area, in.^{2}		14.1		11.9		9.59		7.29		19.9	
r_y, in.		3.25		3.28		3.31		3.34		2.37	
r_x/r_y		1.37		1.37		1.37		1.36		1.74	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

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Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS12

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS12x6x									
		½		⅜[a]		5⁄16[a, c]		¼[a, c]		3⁄16[a, b, c]	
t_{des}, in.		0.500		0.375		0.313		0.250		0.188	
lb/ft		55.7		42.8		36.1		29.2		22.2	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	491	738	377	567	312	469	227	342	151	227
	1	490	737	377	566	312	469	227	341	151	227
	2	487	733	375	563	311	467	226	340	150	226
	3	483	726	371	558	308	464	225	338	149	224
	4	477	717	367	552	306	459	223	335	148	222
	5	469	706	361	543	302	454	220	331	146	220
	6	460	692	355	533	298	447	217	326	144	217
	7	450	676	347	521	292	439	214	321	142	214
	8	438	658	338	508	285	429	210	315	139	210
	9	424	638	328	494	277	417	205	308	137	205
	10	410	617	318	478	269	404	200	301	133	200
	11	395	594	307	461	259	390	195	293	130	195
	12	379	570	295	443	249	375	189	284	126	190
	13	362	545	282	425	239	360	183	275	122	184
	14	345	519	270	405	229	344	177	266	118	178
	15	328	492	257	386	218	328	170	256	114	172
	16	310	466	243	366	207	311	164	246	110	165
	17	292	439	230	346	196	294	157	236	105	159
	18	274	412	217	326	185	278	150	225	101	152
	19	257	386	203	306	174	261	142	213	96.4	145
	20	239	360	190	286	163	244	133	200	91.9	138
	22	206	309	165	248	141	212	116	174	82.7	124
	24	174	262	140	211	121	182	99.6	150	73.7	111
	26	148	223	120	180	103	155	84.9	128	65.0	97.7
	28	128	192	103	155	88.9	134	73.2	110	57.1	85.8
	30	111	167	89.9	135	77.5	116	63.8	95.8	49.7	74.7
	32	97.9	147	79.0	119	68.1	102	56.0	84.2	43.7	65.7
	34	86.7	130	70.0	105	60.3	90.6	49.6	74.6	38.7	58.2
	36	77.4	116	62.4	93.8	53.8	80.8	44.3	66.6	34.5	51.9
	38	69.4	104	56.0	84.2	48.3	72.6	39.7	59.7	31.0	46.6
	40	62.7	94.2	50.6	76.0	43.6	65.5	35.9	53.9	28.0	42.0
	42							32.5	48.9	25.4	38.1
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		491	738	377	567	317	477	257	387	196	294
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		400	600	307	461	258	388	209	314	160	239
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		189	284	147	221	125	188	101	153	75.3	113
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		80.8	122	65.7	98.8	56.9	85.5	47.2	70.9	36.7	55.2
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		152	228	119	179	102	153	82.8	125	60.6	91.1
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		93.3	140	70.9	107	54.0	81.2	39.6	59.5	26.3	39.6
Properties											
Area, in.^{2}		16.4		12.6		10.6		8.59		6.54	
r_y, in.		2.42		2.48		2.51		2.53		2.56	
r_x/r_y		1.73		1.71		1.71		1.71		1.71	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

HSS12

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS12x4x									
		⅝		½		⅜[a]		5⁄16[a, c]		¼[a, c]	
t_{des}, in.		0.625		0.500		0.375		0.313		0.250	
lb/ft		59.3		48.9		37.7		31.8		25.8	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	521	783	431	648	332	499	275	414	197	297
	1	519	780	429	645	331	498	275	413	197	296
	2	512	769	424	637	327	492	272	409	195	294
	3	501	753	415	624	321	483	268	403	193	290
	4	486	730	404	607	313	470	263	396	189	284
	5	467	702	389	585	302	454	256	385	185	278
	6	445	669	372	559	290	435	246	369	179	270
	7	420	632	352	530	276	414	234	352	173	260
	8	394	591	331	498	260	391	222	333	166	250
	9	365	549	309	464	244	367	208	313	159	239
	10	336	505	286	429	227	341	194	292	151	227
	11	307	461	262	394	209	315	180	270	142	214
	12	277	417	238	358	192	288	165	248	134	201
	13	248	373	215	323	174	262	150	226	124	186
	14	221	332	193	289	157	236	136	205	112	169
	15	194	291	171	257	141	211	122	184	101	152
	16	170	256	150	226	125	187	109	164	90.4	136
	17	151	227	133	200	110	166	96.7	145	80.2	120
	18	135	202	119	178	98.5	148	86.2	130	71.5	107
	19	121	182	107	160	88.4	133	77.4	116	64.2	96.5
	20	109	164	96.2	145	79.8	120	69.8	105	57.9	87.0
	22	90.2	136	79.5	119	66.0	99.1	57.7	86.7	47.9	71.9
	24	75.8	114	66.8	100	55.4	83.3	48.5	72.9	40.2	60.4
	26			56.9	85.6	47.2	71.0	41.3	62.1	34.3	51.5
	28							35.6	53.6	29.5	44.4
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		521	783	431	648	332	500	281	422	227	342
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		426	639	351	527	271	406	228	343	185	277
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		227	341	189	284	147	221	125	188	101	153
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		47.8	71.9	44.9	67.5	38.8	58.3	34.4	51.7	29.2	43.9
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		146	220	123	185	97.3	146	83.3	125	68.1	102
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		64.4	96.8	54.9	82.5	42.3	63.6	32.4	48.6	23.7	35.6
Properties											
Area, in.^{2}		17.4		14.4		11.1		9.37		7.59	
r_y, in.		1.55		1.60		1.66		1.69		1.71	
r_x/r_y		2.48		2.45		2.41		2.39		2.39	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

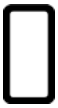
HSS12

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS12x4x		HSS12x3½x				HSS12x3x			
		3⁄16[a, b, c]		3⁄8[a]		5⁄16[a, c]		5⁄16[a, c]		¼[a, c]	
t_{des}, in.		0.188		0.375		0.313		0.313		0.250	
lb/ft		19.6		36.4		30.8		29.7		24.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	129	194	320	481	266	400	257	386	182	274
	1	128	193	319	479	265	398	256	384	182	273
	2	128	192	314	472	262	394	252	378	179	269
	3	126	189	306	460	257	387	246	369	175	263
	4	124	186	296	444	251	377	236	354	169	255
	5	121	182	283	425	240	361	222	334	162	244
	6	117	177	268	402	228	342	206	310	154	232
	7	114	171	251	377	214	321	189	285	145	218
	8	109	164	233	349	199	298	171	258	135	203
	9	105	157	214	321	183	275	153	230	125	187
	10	99.8	150	194	292	167	250	135	203	112	168
	11	94.5	142	175	263	150	226	117	176	97.5	147
	12	89.1	134	156	234	134	202	101	151	84.1	126
	13	83.6	126	137	207	119	179	85.8	129	71.7	108
	14	77.9	117	120	180	104	157	74.0	111	61.9	93.0
	15	72.3	109	104	157	90.8	137	64.4	96.9	53.9	81.0
	16	66.7	100	91.7	138	79.8	120	56.6	85.1	47.4	71.2
	17	61.0	91.7	81.3	122	70.7	106	50.2	75.4	42.0	63.1
	18	55.9	84.0	72.5	109	63.1	94.8	44.8	67.3	37.4	56.2
	19	50.8	76.3	65.0	97.8	56.6	85.1	40.2	60.4	33.6	50.5
	20	45.8	68.9	58.7	88.2	51.1	76.8	36.2	54.5	30.3	45.6
	22	37.9	56.9	48.5	72.9	42.2	63.5				
	24	31.8	47.8	40.8	61.3	35.5	53.3				
	26	27.1	40.8								
	28	23.4	35.1								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		174	261	320	482	271	408	262	394	212	319
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		141	212	261	391	221	332	213	320	173	259
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		75.3	113	147	221	125	188	125	188	101	153
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		23.2	34.9	32.1	48.2	28.8	43.3	23.2	34.8	20.2	30.4
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		52.1	78.2	91.8	138	78.8	119	74.1	111	60.9	91.5
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		15.7	23.5	35.8	53.8	27.3	41.0	22.3	33.5	16.5	24.7
Properties											
Area, in.^{2}		5.80		10.7		9.06		8.75		7.09	
r_y, in.		1.74		1.45		1.47		1.26		1.28	
r_x/r_y		2.37		2.71		2.70		3.08		3.06	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.
[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.
Confirm ASTM A1085 material availability before specifying.



HSS12–HSS10

Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape		HSS12x3x		HSS12x2x						HSS10x8x	
		3/16$^{a, b, c}$		5/16$^{a, c}$		1/4$^{a, c}$		3/16$^{a, b, c}$		5/8	
t_{des}, in.		0.188		0.313		0.250		0.188		0.625	
lb/ft		18.4		27.6		22.4		17.1		67.8	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	117	176	238	358	167	252	106	159	596	895
	1	117	175	235	353	166	249	105	157	595	895
	2	115	173	227	341	161	241	102	153	593	892
	3	113	169	210	316	152	229	97.1	146	590	887
	4	109	164	188	283	142	213	90.8	136	585	880
	5	105	158	163	245	129	193	83.3	125	579	871
	6	100	151	136	205	114	171	74.9	113	572	860
	7	94.6	142	111	166	94.5	142	66.0	99.2	564	848
	8	88.6	133	86.9	131	75.3	113	56.9	85.5	555	834
	9	82.3	124	68.7	103	59.5	89.4	48.1	72.3	544	818
	10	75.6	114	55.6	83.6	48.2	72.4	39.4	59.2	533	801
	11	68.9	104	46.0	69.1	39.8	59.9	32.5	48.9	520	782
	12	62.2	93.4	38.6	58.0	33.5	50.3	27.3	41.1	507	762
	13	55.4	83.3	32.9	49.5	28.5	42.9	23.3	35.0	493	741
	14	49.4	74.3					20.1	30.2	479	719
	15	43.1	64.7							463	696
	16	37.9	56.9							448	673
	17	33.5	50.4							431	648
	18	29.9	45.0							415	624
	19	26.8	40.3							398	598
	20	24.2	36.4							381	573
	22									347	521
	24									313	471
	26									280	421
	28									248	373
	30									218	327
	32									191	287
	34									169	255
	36									151	227
	38									136	204
	40									122	184
	42									111	167
	44									101	152
	46									92.5	139
	48									85.0	128
	50									78.3	118
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		162	243	243	365	197	297	151	226	596	896
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		132	198	198	297	161	241	123	184	484	726
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		75.3	113	125	188	101	153	75.3	113	183	274
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		16.5	24.8	11.9	17.9	11.2	16.9	9.73	14.6	138	207
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		46.6	70.0	65.1	97.9	53.4	80.3	41.1	61.7	164	247
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		10.9	16.4	12.9	19.4	9.69	14.6	6.53	9.82	141	212
Properties											
Area, in.^{2}		5.41		8.12		6.59		5.03		19.9	
r_y, in.		1.31		0.810		0.837		0.866		3.07	
r_x/r_y		3.02		4.57		4.47		4.38		1.19	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

HSS10

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS10x8x									
		½		⅜		5⁄16[a]		¼[a, b, c]		3⁄16[a, b, c]	
t_{des}, in.		0.500		0.375		0.313		0.250		0.188	
lb/ft		55.7		42.8		36.1		29.2		22.2	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	491	738	377	567	317	477	250	375	162	243
	1	490	737	377	566	317	477	249	375	162	243
	2	489	735	376	565	316	475	249	374	162	243
	3	486	731	374	562	314	473	248	372	161	242
	4	483	725	371	558	312	469	246	370	160	241
	5	478	718	367	552	309	465	245	368	159	240
	6	472	710	363	546	306	460	242	364	158	238
	7	466	700	358	539	302	454	240	360	157	236
	8	458	689	353	530	297	447	237	356	155	234
	9	450	676	347	521	292	439	233	351	154	231
	10	441	662	340	511	287	431	230	345	152	228
	11	431	647	332	499	281	422	226	339	150	225
	12	420	632	324	488	274	412	221	333	148	222
	13	409	615	316	475	267	402	217	326	145	218
	14	397	597	307	462	260	391	211	318	143	215
	15	385	579	298	448	253	380	205	308	140	211
	16	372	560	288	434	245	368	199	299	137	206
	17	359	540	279	419	237	356	192	289	134	202
	18	346	520	269	404	228	343	186	279	131	197
	19	332	499	258	388	220	331	179	269	127	191
	20	319	479	248	373	211	318	172	259	123	185
	22	291	437	227	341	194	292	158	238	115	173
	24	263	396	206	310	177	266	144	217	107	161
	26	236	355	186	279	160	240	131	196	98.6	148
	28	210	316	166	249	143	215	117	176	90.2	136
	30	185	278	147	221	127	191	104	157	80.7	121
	32	163	245	129	194	112	168	91.9	138	71.3	107
	34	144	217	114	172	99.2	149	81.4	122	63.1	94.9
	36	129	193	102	153	88.5	133	72.6	109	56.3	84.7
	38	115	173	91.5	138	79.4	119	65.2	98.0	50.6	76.0
	40	104	157	82.6	124	71.7	108	58.8	88.4	45.6	68.6
	42	94.5	142	74.9	113	65.0	97.7	53.4	80.2	41.4	62.2
	44	86.1	129	68.3	103	59.3	89.1	48.6	73.1	37.7	56.7
	46	78.8	118	62.5	93.9	54.2	81.5	44.5	66.9	34.5	51.8
	48	72.3	109	57.4	86.2	49.8	74.8	40.9	61.4	31.7	47.6
	50	66.7	100	52.9	79.5	45.9	69.0	37.6	56.6	29.2	43.9
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		491	738	377	567	317	477	257	387	196	294
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		400	600	307	461	258	388	209	314	160	239
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		153	230	120	180	102	153	83.1	125	63.8	95.8
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		117	176	92.7	139	79.4	119	65.1	97.9	50.3	75.5
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		137	207	108	162	91.8	138	71.8	108	46.2	69.4
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		118	177	92.3	139	76.2	115	54.0	81.2	36.5	54.8
Properties											
Area, in.^{2}		16.4		12.6		10.6		8.59		6.54	
r_y, in.		3.12		3.17		3.22		3.24		3.27	
r_x/r_y		1.19		1.19		1.18		1.19		1.18	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.
[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS10

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape	HSS10x6x									
	5⁄8		1⁄2		3⁄8		5⁄16[a]		1⁄4[a, c]	
t_{des}, in.	0.625		0.500		0.375		0.313		0.250	
lb/ft	59.3		48.9		37.7		31.8		25.8	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y: 0	521	783	431	648	332	499	281	422	220	330
1	520	781	430	647	332	499	280	421	219	330
2	517	777	428	643	330	496	279	419	219	328
3	512	769	424	637	327	492	276	415	217	326
4	505	759	418	629	323	485	273	410	215	323
5	496	746	411	618	318	478	269	404	212	319
6	486	730	403	606	312	468	264	396	209	314
7	473	711	393	591	305	458	258	387	205	309
8	460	691	382	575	296	446	251	377	201	303
9	445	668	370	557	288	432	244	366	197	296
10	428	644	357	537	278	418	236	354	192	288
11	411	618	344	517	268	403	227	342	185	278
12	393	591	329	495	257	386	218	328	178	267
13	374	563	314	472	246	370	209	314	171	256
14	355	534	299	449	234	352	199	300	163	245
15	335	504	283	425	223	334	190	285	155	233
16	316	475	267	401	211	316	180	270	147	221
17	296	445	251	377	199	298	170	255	139	209
18	276	415	235	353	186	280	160	240	131	197
19	257	386	219	329	175	262	150	225	123	185
20	238	358	204	306	163	245	140	210	115	173
22	202	304	174	262	140	211	121	182	99.9	150
24	170	255	147	220	119	179	103	154	85.3	128
26	145	217	125	188	101	152	87.6	132	72.7	109
28	125	187	108	162	87.3	131	75.5	113	62.7	94.2
30	109	163	93.8	141	76.0	114	65.8	98.8	54.6	82.0
32	95.5	143	82.4	124	66.8	100	57.8	86.9	48.0	72.1
34	84.6	127	73.0	110	59.2	89.0	51.2	77.0	42.5	63.9
36	75.4	113	65.1	97.9	52.8	79.3	45.7	68.6	37.9	57.0
38	67.7	102	58.5	87.9	47.4	71.2	41.0	61.6	34.0	51.1
40					42.8	64.3	37.0	55.6	30.7	46.1
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	521	783	431	648	332	500	281	422	227	342
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	426	639	351	527	271	406	228	343	185	277
Available Strength in Shear about X-X Axis, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	183	274	153	230	120	180	102	153	83.1	125
Available Strength in Shear about Y-Y Axis, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	92.7	139	80.8	122	65.7	98.8	56.9	85.5	47.2	70.9
Available Strength in Flexure about X-X Axis, kip-ft	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	135	203	114	171	89.6	135	76.6	115	62.6	94.1
Available Strength in Flexure about Y-Y Axis, kip-ft	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	94.1	141	79.6	120	62.9	94.5	52.2	78.4	37.3	56.1
Properties										
Area, in.^{2}	17.4		14.4		11.1		9.37		7.59	
r_y, in.	2.32		2.37		2.43		2.46		2.49	
r_x/r_y	1.50		1.50		1.49		1.48		1.48	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

HSS10

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS10x6x		HSS10x5x							
		3⁄16[a, b, c]		3⁄8		5⁄16[a]		1⁄4[a, c]		3⁄16[a, c]	
t_{des}, in.		0.188		0.375		0.313		0.250		0.188	
lb/ft		19.6		35.1		29.7		24.1		18.4	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	147	221	308	463	262	394	205	308	136	205
	1	147	221	308	462	261	393	204	307	136	204
	2	147	220	305	459	259	390	203	305	135	203
	3	146	219	301	453	256	385	201	302	134	202
	4	144	217	296	445	252	378	199	298	132	199
	5	143	214	289	435	246	370	195	293	130	196
	6	141	211	282	423	240	360	191	287	128	192
	7	138	208	272	409	232	349	186	280	125	187
	8	135	204	262	394	224	336	181	272	121	182
	9	132	199	251	378	214	322	175	262	118	177
	10	129	194	239	360	204	307	167	251	114	171
	11	126	189	227	341	194	292	159	238	109	164
	12	122	183	214	322	183	275	150	225	105	158
	13	118	177	201	302	172	259	141	212	100	151
	14	114	171	188	282	161	242	132	199	95.5	143
	15	109	165	175	262	150	225	123	185	90.5	136
	16	105	158	161	243	139	209	115	172	85.5	129
	17	101	151	148	223	128	192	106	159	80.4	121
	18	95.9	144	136	204	117	176	97.2	146	75.4	113
	19	91.3	137	124	186	107	161	88.9	134	69.5	105
	20	86.7	130	112	168	96.9	146	80.8	121	63.5	95.4
	22	77.1	116	92.4	139	80.1	120	66.8	100	52.4	78.8
	24	66.0	99.2	77.7	117	67.3	101	56.1	84.4	44.1	66.2
	26	56.2	84.5	66.2	99.5	57.3	86.2	47.8	71.9	37.5	56.4
	28	48.5	72.9	57.1	85.8	49.4	74.3	41.2	62.0	32.4	48.7
	30	42.2	63.5	49.7	74.7	43.1	64.7	35.9	54.0	28.2	42.4
	32	37.1	55.8	43.7	65.7	37.8	56.9	31.6	47.4	24.8	37.3
	34	32.9	49.4	38.7	58.2	33.5	50.4	28.0	42.0	22.0	33.0
	36	29.3	44.1								
	38	26.3	39.6								
	40	23.8	35.7								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		173	260	308	464	262	394	212	319	162	243
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		141	212	251	377	213	320	173	259	132	198
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		63.8	95.8	120	180	102	153	83.1	125	63.8	95.8
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		36.7	55.2	52.3	78.6	45.7	68.6	38.2	57.4	30.0	45.1
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		46.1	69.2	80.6	121	69.1	104	56.6	85.1	43.7	65.6
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		25.0	37.6	49.7	74.6	41.1	61.7	29.7	44.6	19.8	29.8
Properties											
Area, in.^{2}		5.78		10.3		8.75		7.09		5.41	
r_y, in.		2.51		2.04		2.06		2.09		2.12	
r_x/r_y		1.48		1.73		1.72		1.72		1.71	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

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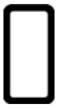


HSS10

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape		HSS10x4x									
		⅝		½		⅜		5/16[a]		¼[a, c]	
t_{des}, in.		0.625		0.500		0.375		0.313		0.250	
lb/ft		50.8		42.1		32.6		27.6		22.4	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	446	670	371	558	287	431	243	365	190	285
	1	444	667	370	556	286	429	242	364	189	284
	2	438	659	365	549	282	424	239	360	188	282
	3	428	644	357	537	277	416	235	353	185	278
	4	415	624	347	522	269	405	229	344	181	272
	5	399	599	334	502	260	390	221	332	177	266
	6	379	570	319	479	249	374	212	318	171	257
	7	358	538	302	454	236	355	202	303	165	248
	8	335	503	283	426	223	335	190	286	156	234
	9	310	466	264	397	208	313	178	268	146	220
	10	285	428	244	366	193	290	166	249	136	205
	11	259	389	223	335	178	267	153	230	126	190
	12	233	351	202	304	162	244	140	211	116	174
	13	209	314	182	274	147	221	127	192	106	159
	14	185	278	162	244	132	198	115	173	95.8	144
	15	162	243	144	216	118	177	103	155	86.1	129
	16	142	214	126	190	104	156	91.2	137	76.7	115
	17	126	189	112	168	91.9	138	80.8	121	68.0	102
	18	112	169	99.7	150	82.0	123	72.1	108	60.6	91.1
	19	101	152	89.5	135	73.6	111	64.7	97.2	54.4	81.8
	20	91.0	137	80.8	121	66.4	99.8	58.4	87.8	49.1	73.8
	22	75.2	113	66.8	100	54.9	82.5	48.3	72.5	40.6	61.0
	24	63.2	95.0	56.1	84.3	46.1	69.3	40.5	60.9	34.1	51.3
	26			47.8	71.8	39.3	59.1	34.5	51.9	29.1	43.7
	28									25.1	37.7
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		446	671	371	558	287	431	243	365	197	297
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		364	546	302	453	234	351	198	297	161	241
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		183	274	153	230	120	180	102	153	83.1	125
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		47.8	71.9	44.9	67.5	38.8	58.3	34.4	51.7	29.2	43.9
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		106	159	90.1	135	71.6	108	61.4	92.3	50.4	75.8
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		53.9	81.0	46.2	69.4	37.2	55.9	30.9	46.5	22.4	33.6
Properties											
Area, in.^{2}		14.9		12.4		9.58		8.12		6.59	
r_y, in.		1.53		1.58		1.63		1.66		1.69	
r_x/r_y		2.12		2.09		2.08		2.07		2.05	

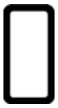
[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.



HSS10

Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape		HSS10x4x		HSS10x3½x							
		3/16[a, c]		½		3/8		5/16[a]		¼[a, c]	
t_{des}, in.		0.188		0.500		0.375		0.313		0.250	
lb/ft		17.1		40.3		31.3		26.5		21.6	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	125	188	356	535	275	414	234	351	182	274
	1	125	187	354	533	274	412	233	350	182	273
	2	124	186	348	524	270	406	229	344	180	270
	3	122	183	339	509	263	395	224	336	176	265
	4	120	180	326	490	254	381	216	324	172	258
	5	117	176	310	465	242	364	206	310	166	250
	6	113	170	291	438	229	344	195	293	160	240
	7	110	165	271	407	214	322	183	275	150	225
	8	105	158	249	374	198	298	170	255	140	210
	9	101	151	226	340	182	273	156	234	129	193
	10	95.5	144	203	306	165	247	142	213	117	176
	11	90.2	136	181	272	148	222	128	192	106	159
	12	84.7	127	159	239	131	197	114	171	95.0	143
	13	79.1	119	138	207	115	173	100	151	84.2	127
	14	73.5	110	119	179	100	151	87.4	131	74.0	111
	15	67.6	102	104	156	87.3	131	76.2	114	64.4	96.8
	16	60.6	91.0	91.1	137	76.7	115	67.0	101	56.6	85.1
	17	53.7	80.8	80.7	121	67.9	102	59.3	89.1	50.2	75.4
	18	47.9	72.1	72.0	108	60.6	91.1	52.9	79.5	44.7	67.2
	19	43.0	64.7	64.6	97.1	54.4	81.8	47.5	71.4	40.2	60.4
	20	38.8	58.4	58.3	87.6	49.1	73.8	42.8	64.4	36.2	54.5
	22	32.1	48.2	48.2	72.4	40.6	61.0	35.4	53.2	29.9	45.0
	24	27.0	40.5					29.8	44.7	25.2	37.8
	26	23.0	34.5								
	28	19.8	29.8								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		151	226	356	536	275	414	234	351	190	285
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		123	184	290	435	224	336	190	286	155	232
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		63.8	95.8	153	230	120	180	102	153	83.1	125
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		23.2	34.9	35.9	54.0	32.1	48.2	28.8	43.3	24.7	37.1
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		38.9	58.5	84.1	126	67.1	101	57.6	86.6	47.4	71.3
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		14.9	22.4	38.7	58.1	31.2	46.9	26.1	39.2	18.9	28.4
Properties											
Area, in.^{2}		5.03		11.9		9.20		7.81		6.34	
r_y, in.		1.72		1.37		1.43		1.45		1.48	
r_x/r_y		2.04		2.36		2.33		2.32		2.30	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

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Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape		HSS10x3½x		HSS10x3x							
		3/16$^{a, c}$		3/8		5/16^{a}		1/4$^{a, c}$		3/16$^{a, c}$	
t_{des}, in.		0.188		0.375		0.313		0.250		0.188	
lb/ft		16.4		30.0		25.5		20.7		15.8	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	119	179	264	397	224	337	175	263	114	171
	1	119	179	262	395	223	335	174	261	113	170
	2	118	177	257	386	218	328	171	258	112	168
	3	116	174	248	372	211	317	167	251	109	164
	4	113	170	236	354	201	302	162	243	106	159
	5	109	165	221	332	189	284	154	232	102	153
	6	105	158	204	307	175	263	144	217	96.6	145
	7	101	151	186	279	160	241	132	199	91.1	137
	8	95.7	144	167	251	145	217	120	180	85.0	128
	9	90.3	136	148	222	129	194	107	162	78.6	118
	10	84.5	127	129	194	113	170	94.9	143	71.9	108
	11	78.5	118	111	166	97.9	147	82.8	124	65.1	97.9
	12	72.4	109	93.7	141	83.5	125	71.2	107	56.9	85.5
	13	66.3	99.6	79.8	120	71.1	107	60.7	91.2	48.6	73.1
	14	58.6	88.1	68.8	103	61.3	92.2	52.3	78.6	41.9	63.0
	15	51.2	76.9	60.0	90.1	53.4	80.3	45.6	68.5	36.5	54.9
	16	45.0	67.6	52.7	79.2	47.0	70.6	40.1	60.2	32.1	48.3
	17	39.9	59.9	46.7	70.2	41.6	62.5	35.5	53.3	28.4	42.8
	18	35.6	53.4	41.6	62.6	37.1	55.8	31.6	47.6	25.4	38.1
	19	31.9	48.0	37.4	56.2	33.3	50.0	28.4	42.7	22.8	34.2
	20	28.8	43.3	33.7	50.7	30.1	45.2	25.6	38.5	20.6	30.9
	22	23.8	35.8								
	24	20.0	30.1								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		145	218	264	397	224	337	182	274	140	210
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		118	177	215	323	183	274	149	223	114	171
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		63.8	95.8	120	180	102	153	83.1	125	63.8	95.8
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		19.9	29.8	25.3	38.1	23.2	34.8	20.2	30.4	16.5	24.8
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		36.7	55.1	62.6	94.1	53.9	81.0	44.4	66.8	34.4	51.8
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		12.6	18.9	25.7	38.6	21.5	32.3	15.5	23.3	10.4	15.6
Properties											
Area, in.^{2}		4.84		8.83		7.49		6.09		4.66	
r_y, in.		1.51		1.21		1.24		1.27		1.30	
r_x/r_y		2.28		2.68		2.65		2.62		2.59	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

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Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS10–HSS9

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS10x2x								HSS9x7x	
		⅜		5⁄16[a]		¼[a, c]		3⁄16[a, c]		⅝	
t_{des}, in.		0.375		0.313		0.250		0.188		0.625	
lb/ft		27.5		23.3		19.0		14.5		59.3	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	242	364	206	309	160	240	102	154	521	783
	1	238	357	202	304	158	238	101	152	520	782
	2	226	339	193	290	153	230	98.4	148	518	778
	3	207	311	178	267	144	217	93.6	141	514	773
	4	183	275	158	238	131	197	87.3	131	509	765
	5	156	235	137	206	114	172	79.7	120	502	754
	6	129	194	114	172	96.5	145	71.2	107	494	742
	7	103	154	92.4	139	79.1	119	62.2	93.4	484	728
	8	79.4	119	72.2	109	62.8	94.4	51.3	77.1	474	712
	9	62.7	94.2	57.1	85.8	49.6	74.6	40.6	61.0	462	694
	10	50.8	76.3	46.2	69.5	40.2	60.4	32.9	49.4	449	675
	11	42.0	63.1	38.2	57.4	33.2	49.9	27.2	40.9	435	654
	12	35.3	53.0	32.1	48.3	27.9	42.0	22.8	34.3	420	632
	13			27.4	41.1	23.8	35.7	19.5	29.2	405	609
	14							16.8	25.2	389	585
	15									373	560
	16									356	535
	17									339	509
	18									322	483
	19									304	458
	20									287	432
	22									254	381
	24									221	332
	26									190	286
	28									164	246
	30									143	215
	32									125	189
	34									111	167
	36									99.2	149
	38									89.0	134
	40									80.3	121
	42									72.9	109
	44									66.4	99.8
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		242	364	206	309	167	252	128	193	521	783
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		197	295	167	251	136	204	104	156	426	639
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		120	180	102	153	83.1	125	63.8	95.8	160	241
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		11.8	17.7	11.9	17.9	11.2	16.9	9.73	14.6	115	173
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		53.6	80.6	46.4	69.8	38.4	57.8	29.7	44.6	127	191
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		15.1	22.7	12.7	19.2	9.12	13.7	6.21	9.33	107	161

Properties

	⅜	5⁄16	¼	3⁄16	⅝
Area, in.^{2}	8.08	6.87	5.59	4.28	17.4
r_y, in.	0.776	0.803	0.830	0.858	2.66
r_x/r_y	3.93	3.86	3.80	3.73	1.22

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS9

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS9x7x									
		½		⅜		5⁄16		¼[a]		3⁄16[a, b, c]	
t_{des}, in.		0.500		0.375		0.313		0.250		0.188	
lb/ft		48.9		37.7		31.8		25.8		19.6	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	431	648	332	499	281	422	227	342	156	235
	1	431	647	332	499	280	421	227	341	156	234
	2	429	644	331	497	279	419	226	340	156	234
	3	426	640	328	493	277	417	225	338	155	233
	4	421	633	325	489	275	413	223	334	154	231
	5	416	625	321	483	271	408	220	331	152	229
	6	409	615	316	475	267	402	217	326	151	226
	7	402	604	311	467	263	395	213	320	149	223
	8	393	591	304	458	257	387	209	314	146	220
	9	384	577	297	447	252	378	204	307	144	216
	10	374	561	290	435	245	369	199	299	141	212
	11	362	545	281	423	238	358	194	291	138	207
	12	351	527	273	410	231	348	188	283	134	202
	13	338	509	264	396	224	336	182	274	131	197
	14	326	489	254	382	216	324	176	264	127	191
	15	312	469	244	367	207	312	169	254	123	186
	16	299	449	234	352	199	299	162	244	119	180
	17	285	428	224	336	190	286	155	234	115	173
	18	271	407	213	320	182	273	148	223	111	167
	19	257	386	203	304	173	260	141	212	107	161
	20	243	365	192	289	164	246	134	202	103	154
	22	215	324	171	257	146	220	120	181	92.8	139
	24	189	284	151	227	129	195	107	160	82.4	124
	26	163	245	131	198	113	170	93.4	140	72.5	109
	28	141	212	113	170	97.8	147	80.9	122	62.9	94.6
	30	123	184	98.8	148	85.2	128	70.5	106	54.8	82.4
	32	108	162	86.8	130	74.9	113	62.0	93.1	48.2	72.4
	34	95.5	144	76.9	116	66.3	99.7	54.9	82.5	42.7	64.2
	36	85.2	128	68.6	103	59.2	88.9	49.0	73.6	38.1	57.2
	38	76.4	115	61.6	92.5	53.1	79.8	43.9	66.0	34.2	51.4
	40	69.0	104	55.6	83.5	47.9	72.0	39.7	59.6	30.8	46.4
	42	62.6	94.1	50.4	75.7	43.5	65.3	36.0	54.1	28.0	42.0
	44	57.0	85.7	45.9	69.0	39.6	59.5	32.8	49.3	25.5	38.3
	46			42.0	63.1	36.2	54.5	30.0	45.1	23.3	35.1
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		431	648	332	500	281	422	227	342	173	260
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		351	527	271	406	228	343	185	277	141	212
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		135	203	106	160	90.6	136	74.1	111	57.0	85.7
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		98.8	149	79.2	119	68.1	102	56.1	84.4	43.5	65.4
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		107	161	84.3	127	72.1	108	58.9	88.5	38.2	57.4
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		90.1	135	70.9	107	60.6	91.1	44.3	66.5	29.7	44.7
Properties											
Area, in.^{2}		14.4		11.1		9.37		7.59		5.78	
r_y, in.		2.71		2.77		2.80		2.83		2.86	
r_x/r_y		1.22		1.22		1.21		1.21		1.21	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS9

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape	HSS9x5x									
	5⁄8		1⁄2		3⁄8		5⁄16		1⁄4[a]	
t_{des}, in.	0.625		0.500		0.375		0.313		0.250	
lb/ft	50.8		42.1		32.6		27.6		22.4	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y: 0	446	670	371	558	287	431	243	365	197	297
1	445	669	370	556	286	430	242	364	197	296
2	441	663	367	552	284	427	241	362	195	294
3	435	653	362	544	280	421	238	357	193	290
4	426	640	355	534	275	414	233	351	190	285
5	415	624	347	521	269	404	228	343	186	279
6	402	604	336	506	261	393	222	334	181	271
7	387	582	325	488	253	380	215	323	175	263
8	371	557	312	468	243	365	207	311	169	253
9	353	531	297	447	233	350	198	298	162	243
10	334	502	282	424	222	333	189	284	154	232
11	315	473	266	401	210	315	179	269	147	220
12	294	442	250	376	198	297	169	254	139	208
13	274	412	234	351	185	279	159	238	130	196
14	253	381	217	326	173	260	148	223	122	183
15	233	350	200	301	161	241	138	207	114	171
16	213	320	184	277	148	223	127	191	105	158
17	194	291	168	253	136	205	117	176	97.0	146
18	175	263	153	230	124	187	107	161	89.0	134
19	157	236	138	207	113	170	97.5	147	81.3	122
20	142	213	124	187	102	153	88.2	133	73.7	111
22	117	176	103	154	84.3	127	72.9	110	60.9	91.5
24	98.5	148	86.3	130	70.8	106	61.2	92.0	51.2	76.9
26	83.9	126	73.6	111	60.4	90.7	52.2	78.4	43.6	65.5
28	72.4	109	63.4	95.3	52.0	78.2	45.0	67.6	37.6	56.5
30	63.0	94.8	55.2	83.0	45.3	68.1	39.2	58.9	32.7	49.2
32			48.6	73.0	39.8	59.9	34.4	51.8	28.8	43.3
34							30.5	45.9	25.5	38.3
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	446	671	371	558	287	431	243	365	197	297
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	364	546	302	453	234	351	198	297	161	241
Available Strength in Shear about X-X Axis, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	160	241	135	203	106	160	90.6	136	74.1	111
Available Strength in Shear about Y-Y Axis, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	70.3	106	62.9	94.5	52.3	78.6	45.7	68.6	38.2	57.4
Available Strength in Flexure about X-X Axis, kip-ft	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	101	152	85.8	129	68.1	102	58.4	87.8	48.2	72.4
Available Strength in Flexure about Y-Y Axis, kip-ft	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	66.4	99.8	56.6	85.1	45.2	67.9	38.9	58.5	28.7	43.2
Properties										
Area, in.^{2}	14.9		12.4		9.58		8.12		6.59	
r_y, in.	1.91		1.96		2.02		2.04		2.07	
r_x/r_y	1.60		1.59		1.58		1.58		1.58	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

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HSS9

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS9x5x		HSS9x3x							
		3⁄16[a, c]		½		⅜		5⁄16		¼[a]	
t_{des}, in.		0.188		0.500		0.375		0.313		0.250	
lb/ft		17.1		35.2		27.5		23.3		19.0	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	134	201	311	468	242	364	206	309	167	252
	1	134	201	309	464	240	361	204	307	166	250
	2	133	200	302	453	235	353	200	301	163	245
	3	132	198	290	436	227	340	193	290	158	237
	4	130	195	274	412	215	323	184	277	151	226
	5	128	192	255	384	202	303	173	260	142	213
	6	125	188	234	351	186	279	160	241	132	198
	7	122	184	211	317	169	254	146	220	121	182
	8	119	179	187	281	152	228	132	198	109	165
	9	115	173	163	246	134	201	117	176	97.8	147
	10	111	167	140	211	116	175	103	154	86.2	130
	11	107	160	119	178	99.9	150	88.6	133	75.0	113
	12	102	154	99.7	150	84.3	127	75.3	113	64.3	96.7
	13	97.4	146	84.9	128	71.9	108	64.2	96.5	54.8	82.4
	14	92.5	139	73.2	110	62.0	93.1	55.4	83.2	47.3	71.0
	15	87.5	132	63.8	95.9	54.0	81.1	48.2	72.5	41.2	61.9
	16	81.7	123	56.1	84.3	47.4	71.3	42.4	63.7	36.2	54.4
	17	75.5	114	49.7	74.7	42.0	63.2	37.5	56.4	32.1	48.2
	18	69.5	104	44.3	66.6	37.5	56.3	33.5	50.3	28.6	43.0
	19	63.6	95.6	39.8	59.8	33.6	50.6	30.1	45.2	25.7	38.6
	20	57.9	87.0			30.4	45.6	27.1	40.8	23.2	34.8
	22	47.8	71.9								
	24	40.2	60.4								
	26	34.3	51.5								
	28	29.5	44.4								
	30	25.7	38.7								
	32	22.6	34.0								
	34	20.0	30.1								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		151	226	311	468	242	364	206	309	167	252
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		123	184	254	380	197	295	167	251	136	204
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		57.0	85.7	135	203	106	160	90.6	136	74.1	111
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		30.0	45.1	26.9	40.5	25.3	38.1	23.2	34.8	20.2	30.4
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		37.2	55.9	64.6	97.1	52.1	78.4	44.9	67.5	37.2	55.9
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		19.2	28.8	28.2	42.4	23.2	34.8	20.2	30.3	15.0	22.6
Properties											
Area, in.^{2}		5.03		10.4		8.08		6.87		5.59	
r_y, in.		2.10		1.15		1.20		1.23		1.26	
r_x/r_y		1.57		2.48		2.45		2.42		2.40	

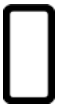
[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.



HSS9–HSS8

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS9x3x		HSS8x6x							
		3/16$^{a, c}$		5/8		1/2		3/8		5/16	
t_{des}, in.		0.188		0.625		0.500		0.375		0.313	
lb/ft		14.5		50.8		42.1		32.6		27.6	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	112	168	446	670	371	558	287	431	243	365
	1	111	167	445	669	371	557	286	430	243	365
	2	109	165	442	665	368	554	285	428	241	363
	3	107	161	438	658	365	548	282	424	239	359
	4	103	155	432	649	360	541	278	418	236	355
	5	99.1	149	424	637	353	531	274	411	232	349
	6	94.1	141	414	622	346	519	268	403	228	342
	7	88.5	133	403	606	337	506	261	393	222	334
	8	82.3	124	391	587	327	491	254	382	216	325
	9	75.9	114	377	567	316	475	246	370	209	315
	10	68.1	102	362	545	304	457	237	357	202	304
	11	59.6	89.6	347	521	292	439	228	343	195	292
	12	51.5	77.4	331	497	279	419	218	328	186	280
	13	44.0	66.1	314	472	265	399	208	313	178	268
	14	37.9	57.0	297	446	251	378	198	298	169	255
	15	33.0	49.7	279	420	237	357	187	282	161	241
	16	29.0	43.6	262	394	223	335	177	266	152	228
	17	25.7	38.7	245	368	209	314	166	250	143	214
	18	22.9	34.5	227	342	195	293	155	234	134	201
	19	20.6	31.0	211	316	181	272	145	218	125	188
	20	18.6	27.9	194	292	167	252	135	202	116	175
	22			163	245	141	213	115	173	99.6	150
	24			137	205	119	179	96.7	145	84.1	126
	26			116	175	101	152	82.4	124	71.6	108
	28			100	151	87.3	131	71.0	107	61.8	92.8
	30			87.5	131	76.1	114	61.9	93.0	53.8	80.9
	32			76.9	116	66.9	100	54.4	81.7	47.3	71.1
	34			68.1	102	59.2	89.0	48.2	72.4	41.9	62.9
	36			60.8	91.3	52.8	79.4	43.0	64.6	37.4	56.1
	38					47.4	71.3	38.6	58.0	33.5	50.4
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		128	193	446	671	371	558	287	431	243	365
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		104	156	364	546	302	453	234	351	198	297
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		57.0	85.7	138	207	117	176	92.7	139	79.4	119
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		16.5	24.8	92.7	139	80.8	122	65.7	98.8	56.9	85.5
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		28.9	43.5	94.8	143	80.3	121	63.9	96.0	54.9	82.5
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		10.1	15.1	77.3	116	65.9	99.0	52.4	78.8	44.9	67.5
Properties											
Area, in.^{2}		4.28		14.9		12.4		9.58		8.12	
r_y, in.		1.29		2.25		2.30		2.36		2.39	
r_x/r_y		2.37		1.26		1.25		1.25		1.25	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS8

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS8x6x				HSS8x4x					
		¼[a]		3⁄16[a, b, c]		⅝		½		⅜	
t_{des}, in.		0.250		0.188		0.625		0.500		0.375	
lb/ft		22.4		17.1		42.3		35.2		27.5	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	197	297	142	214	371	558	311	468	242	364
	1	197	296	142	214	370	555	310	466	241	362
	2	196	294	142	213	364	548	306	460	238	358
	3	194	292	141	211	356	535	299	450	233	350
	4	192	288	139	209	344	517	290	436	227	340
	5	189	284	137	207	330	496	279	419	218	328
	6	185	278	135	203	313	470	265	399	209	314
	7	181	272	133	200	294	442	251	377	198	297
	8	176	264	130	195	274	412	234	352	186	279
	9	171	256	127	191	253	380	217	327	173	261
	10	165	248	123	185	231	347	200	300	160	241
	11	159	239	120	180	209	314	182	273	147	221
	12	152	229	116	174	188	282	164	247	134	201
	13	146	219	112	168	167	250	147	221	121	181
	14	139	208	107	160	147	220	130	196	108	162
	15	132	198	101	153	128	192	114	172	95.9	144
	16	125	187	96.1	144	112	169	101	151	84.3	127
	17	117	176	90.7	136	99.4	149	89.1	134	74.7	112
	18	110	166	85.3	128	88.7	133	79.5	119	66.6	100
	19	103	155	79.9	120	79.6	120	71.3	107	59.8	89.9
	20	96.1	144	74.7	112	71.8	108	64.4	96.7	54.0	81.1
	22	82.6	124	64.4	96.8	59.4	89.2	53.2	79.9	44.6	67.0
	24	69.9	105	54.7	82.2	49.9	75.0	44.7	67.2	37.5	56.3
	26	59.6	89.6	46.6	70.1					31.9	48.0
	28	51.4	77.2	40.2	60.4						
	30	44.8	67.3	35.0	52.6						
	32	39.3	59.1	30.8	46.3						
	34	34.8	52.4	27.3	41.0						
	36	31.1	46.7	24.3	36.5						
	38	27.9	41.9	21.8	32.8						
	40	25.2	37.8	19.7	29.6						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		197	297	151	226	371	558	311	468	242	364
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		161	241	123	184	302	453	254	380	197	295
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		65.1	97.9	50.3	75.5	138	207	117	176	92.7	139
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		47.2	70.9	36.7	55.2	47.8	71.9	44.9	67.5	38.8	58.3
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		44.9	67.5	33.3	50.0	71.9	108	61.6	92.6	49.7	74.6
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		35.6	53.6	23.4	35.2	43.2	64.9	37.4	56.3	30.4	45.8
Properties											
Area, in.^{2}		6.59		5.03		12.4		10.4		8.08	
r_y, in.		2.42		2.45		1.49		1.54		1.60	
r_x/r_y		1.25		1.24		1.76		1.75		1.73	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

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Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS8

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape		HSS8x4x						HSS8x3x			
		5⁄16		¼[a]		3⁄16[a, c]		½		⅜	
t_{des}, in.		0.313		0.250		0.188		0.500		0.375	
lb/ft		23.3		19.0		14.5		31.8		24.9	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	206	309	167	252	120	180	280	421	219	330
	1	205	308	167	251	120	180	278	418	218	327
	2	202	304	165	248	119	178	271	408	213	320
	3	198	298	162	243	117	176	261	392	205	308
	4	193	290	157	236	115	172	246	370	195	293
	5	186	280	152	228	112	168	229	344	182	274
	6	178	268	146	219	108	162	209	315	168	252
	7	169	254	138	208	104	156	188	283	152	229
	8	159	239	131	196	99.4	149	167	251	136	205
	9	149	223	122	184	94.5	142	145	219	120	181
	10	138	207	114	171	88.2	133	125	187	104	157
	11	127	190	105	158	81.6	123	105	158	89.3	134
	12	115	173	95.9	144	74.9	113	88.2	133	75.2	113
	13	104	157	87.1	131	68.2	103	75.1	113	64.1	96.4
	14	93.7	141	78.4	118	61.7	92.7	64.8	97.4	55.3	83.1
	15	83.4	125	70.1	105	55.4	83.2	56.4	84.8	48.2	72.4
	16	73.5	110	62.1	93.3	49.3	74.0	49.6	74.5	42.3	63.6
	17	65.1	97.9	55.0	82.6	43.6	65.6	43.9	66.0	37.5	56.3
	18	58.1	87.3	49.0	73.7	38.9	58.5	39.2	58.9	33.4	50.3
	19	52.1	78.4	44.0	66.1	34.9	52.5	35.2	52.9	30.0	45.1
	20	47.0	70.7	39.7	59.7	31.5	47.4				
	22	38.9	58.4	32.8	49.3	26.1	39.2				
	24	32.7	49.1	27.6	41.5	21.9	32.9				
	26	27.8	41.8	23.5	35.3	18.7	28.0				
	28					16.1	24.2				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		206	309	167	252	128	193	280	421	219	330
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		167	251	136	204	104	156	228	342	179	268
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		79.4	119	65.1	97.9	50.3	75.5	117	176	92.7	139
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		34.4	51.7	29.2	43.9	23.2	34.9	26.9	40.5	25.3	38.1
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		42.7	64.1	35.2	52.9	27.4	41.3	52.4	78.8	42.4	63.8
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		26.2	39.4	21.0	31.6	14.0	21.0	25.2	37.9	20.7	31.2
Properties											
Area, in.^{2}		6.87		5.59		4.28		9.36		7.33	
r_y, in.		1.62		1.65		1.68		1.14		1.19	
r_x/r_y		1.73		1.72		1.71		2.24		2.22	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

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Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS8

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape		HSS8x3x						HSS8x2x			
		$\frac{5}{16}$		$\frac{1}{4}$[a]		$\frac{3}{16}$[a, c]		$\frac{3}{8}$		$\frac{5}{16}$	
t_{des}, in.		0.313		0.250		0.188		0.375		0.313	
lb/ft		21.2		17.3		13.3		22.4		19.1	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	187	281	152	229	109	163	197	296	168	253
	1	186	279	151	228	108	162	194	291	165	249
	2	182	273	148	223	106	160	183	276	157	237
	3	175	263	143	216	104	156	168	252	145	218
	4	167	251	137	206	100	151	148	222	129	193
	5	157	235	129	194	95.9	144	126	189	111	166
	6	145	218	120	180	90.7	136	103	155	92.1	138
	7	132	199	110	165	84.8	127	81.8	123	74.1	111
	8	119	179	99.0	149	76.9	116	63.0	94.6	57.6	86.6
	9	105	158	88.3	133	68.8	103	49.8	74.8	45.5	68.5
	10	92.1	138	77.7	117	60.8	91.4	40.3	60.6	36.9	55.4
	11	79.4	119	67.4	101	53.0	79.7	33.3	50.1	30.5	45.8
	12	67.3	101	57.6	86.6	45.6	68.6	28.0	42.1	25.6	38.5
	13	57.4	86.2	49.1	73.8	38.9	58.4			21.8	32.8
	14	49.5	74.3	42.4	63.7	33.5	50.3				
	15	43.1	64.8	36.9	55.5	29.2	43.9				
	16	37.9	56.9	32.4	48.7	25.6	38.5				
	17	33.5	50.4	28.7	43.2	22.7	34.1				
	18	29.9	45.0	25.6	38.5	20.3	30.5				
	19	26.9	40.4	23.0	34.6	18.2	27.3				
	20	24.2	36.4	20.8	31.2	16.4	24.7				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		187	281	152	229	117	176	197	296	168	253
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		152	228	124	186	95.2	143	161	241	137	206
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		79.4	119	65.1	97.9	50.3	75.5	92.7	139	79.4	119
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		23.2	34.8	20.2	30.4	16.5	24.8	11.8	17.7	11.9	17.9
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		36.7	55.1	30.4	45.8	23.7	35.7	35.4	53.3	30.7	46.1
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		18.1	27.2	14.6	21.9	9.69	14.6	12.1	18.1	10.7	16.1
Properties											
Area, in.^{2}		6.24		5.09		3.90		6.58		5.62	
r_y, in.		1.22		1.25		1.27		0.766		0.793	
r_x/r_y		2.20		2.18		2.17		3.22		3.17	

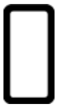
[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.



HSS8–HSS7

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS8x2x				HSS7x5x					
		¼[a]		3⁄16[a, c]		½		⅜		5⁄16	
t_{des}, in.		0.250		0.188		0.500		0.375		0.313	
lb/ft		15.6		12.0		35.2		27.5		23.3	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	137	207	97.6	147	311	468	242	364	206	309
	1	135	203	96.5	145	310	467	241	363	205	308
	2	129	194	93.4	140	308	463	239	360	203	306
	3	119	179	88.5	133	303	456	236	355	201	302
	4	107	161	82.0	123	297	446	231	348	197	296
	5	92.8	140	73.2	110	289	435	226	339	192	289
	6	78.1	117	62.3	93.7	280	421	219	329	187	281
	7	63.7	95.7	51.5	77.4	270	405	211	317	180	271
	8	50.2	75.5	41.3	62.1	258	388	203	305	173	260
	9	39.7	59.6	32.6	49.0	245	369	193	291	165	249
	10	32.1	48.3	26.4	39.7	232	349	183	276	157	236
	11	26.6	39.9	21.8	32.8	218	328	173	260	149	223
	12	22.3	33.5	18.4	27.6	204	306	162	244	140	210
	13	19.0	28.6	15.6	23.5	189	284	152	228	131	196
	14			13.5	20.3	175	263	141	211	122	183
	15					160	241	130	195	112	169
	16					146	220	119	179	103	155
	17					133	200	109	163	94.7	142
	18					120	180	98.6	148	86.2	129
	19					107	161	88.8	134	77.9	117
	20					96.9	146	80.2	121	70.3	106
	22					80.1	120	66.3	99.6	58.1	87.3
	24					67.3	101	55.7	83.7	48.8	73.4
	26					57.4	86.2	47.4	71.3	41.6	62.5
	28					49.5	74.3	40.9	61.5	35.9	53.9
	30					43.1	64.8	35.6	53.6	31.2	46.9
	32							31.3	47.1	27.5	41.3
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		137	207	106	159	311	468	242	364	206	309
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		112	168	86.1	129	254	380	197	295	167	251
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		65.1	97.9	50.3	75.5	98.8	149	79.2	119	68.1	102
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		11.2	16.9	9.73	14.6	62.9	94.5	52.3	78.6	45.7	68.6
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		25.7	38.6	20.1	30.2	57.6	86.6	46.2	69.4	39.9	60.0
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		8.69	13.1	5.78	8.69	45.4	68.3	36.4	54.8	31.4	47.3
Properties											
Area, in.^{2}		4.59		3.53		10.4		8.08		6.87	
r_y, in.		0.819		0.847		1.89		1.95		1.98	
r_x/r_y		3.13		3.07		1.31		1.30		1.30	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

HSS7

Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape		HSS7x5x				HSS7x4x					
		¼		3⁄16[a, c]		½		⅜		5⁄16	
t_{des}, in.		0.250		0.188		0.500		0.375		0.313	
lb/ft		19.0		14.5		31.8		24.9		21.2	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	167	252	128	192	280	421	219	330	187	281
	1	167	251	128	192	279	419	219	328	186	280
	2	166	249	127	190	275	414	216	324	184	276
	3	163	246	125	188	269	404	211	317	180	271
	4	161	241	123	185	261	392	205	308	175	263
	5	157	236	120	181	250	376	197	296	169	253
	6	152	229	117	176	238	357	188	283	161	242
	7	147	221	113	170	224	337	178	268	153	230
	8	142	213	109	164	209	315	167	251	144	216
	9	136	204	104	157	194	291	155	233	134	201
	10	129	194	99.5	150	178	267	143	215	124	186
	11	122	184	94.4	142	161	243	131	197	114	171
	12	115	173	89.0	134	145	219	119	178	103	155
	13	108	162	83.6	126	130	195	107	160	93.2	140
	14	100	151	78.0	117	115	172	95.0	143	83.4	125
	15	93.1	140	72.5	109	100	151	83.8	126	74.1	111
	16	85.9	129	67.1	101	88.2	133	73.7	111	65.1	97.9
	17	78.8	118	61.7	92.7	78.1	117	65.3	98.1	57.7	86.7
	18	71.9	108	56.5	84.9	69.7	105	58.2	87.5	51.5	77.3
	19	65.3	98.2	51.4	77.3	62.5	94.0	52.2	78.5	46.2	69.4
	20	58.9	88.6	46.5	69.9	56.4	84.8	47.1	70.9	41.7	62.7
	22	48.7	73.2	38.4	57.7	46.6	70.1	39.0	58.6	34.5	51.8
	24	40.9	61.5	32.3	48.5	39.2	58.9	32.7	49.2	28.9	43.5
	26	34.9	52.4	27.5	41.3			27.9	41.9	24.7	37.1
	28	30.1	45.2	23.7	35.6						
	30	26.2	39.4	20.7	31.0						
	32	23.0	34.6	18.2	27.3						
	34			16.1	24.2						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		167	252	128	193	280	421	219	330	187	281
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		136	204	104	156	228	342	179	268	152	228
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		56.1	84.4	43.5	65.4	98.8	149	79.2	119	68.1	102
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		38.2	57.4	30.0	45.1	44.9	67.5	38.8	58.3	34.4	51.7
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		32.9	49.5	25.4	38.3	49.4	74.3	39.9	60.0	34.7	52.1
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		25.9	39.0	17.6	26.5	33.2	49.9	26.9	40.5	23.4	35.1
Properties											
Area, in.^{2}		5.59		4.28		9.36		7.33		6.24	
r_y, in.		2.01		2.04		1.52		1.57		1.60	
r_x/r_y		1.30		1.29		1.57		1.56		1.55	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

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Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS7

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape		HSS7x4x				HSS7x3x					
		¼		$^3\!/_{16}$$^{a, c}$		½		⅜		$^5\!/_{16}$	
t_{des}, in.		0.250		0.188		0.500		0.375		0.313	
lb/ft		17.3		13.3		28.4		22.4		19.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	152	229	116	175	250	376	197	296	168	253
	1	152	228	116	174	248	373	196	294	167	251
	2	150	225	115	173	242	364	191	287	163	246
	3	147	221	113	170	232	349	184	277	158	237
	4	143	215	110	165	219	329	175	262	150	225
	5	138	207	106	160	203	305	163	245	140	211
	6	132	199	102	153	185	278	150	226	129	194
	7	125	189	96.8	146	166	249	136	204	118	177
	8	118	178	91.4	137	146	220	121	182	105	158
	9	111	166	85.7	129	127	191	107	160	93.1	140
	10	103	154	79.7	120	108	163	92.5	139	81.0	122
	11	94.3	142	73.5	111	90.5	136	78.9	119	69.5	104
	12	86.1	129	67.4	101	76.0	114	66.4	99.8	58.7	88.2
	13	78.0	117	61.2	92.0	64.8	97.3	56.6	85.1	50.0	75.1
	14	70.1	105	55.2	83.0	55.8	83.9	48.8	73.3	43.1	64.8
	15	62.5	93.9	49.4	74.3	48.6	73.1	42.5	63.9	37.5	56.4
	16	55.1	82.9	43.8	65.9	42.8	64.3	37.4	56.1	33.0	49.6
	17	48.8	73.4	38.8	58.3	37.9	56.9	33.1	49.7	29.2	43.9
	18	43.6	65.5	34.6	52.0	33.8	50.8	29.5	44.4	26.1	39.2
	19	39.1	58.8	31.1	46.7			26.5	39.8	23.4	35.2
	20	35.3	53.0	28.0	42.2					21.1	31.7
	22	29.2	43.8	23.2	34.8						
	24	24.5	36.8	19.5	29.3						
	26	20.9	31.4	16.6	24.9						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		152	229	117	176	250	376	197	296	168	253
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		124	186	95.2	143	204	306	161	241	137	206
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		56.1	84.4	43.5	65.4	98.8	149	79.2	119	68.1	102
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		29.2	43.9	23.2	34.9	26.9	40.5	25.3	38.1	23.2	34.8
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		28.7	43.1	22.3	33.5	41.4	62.3	33.7	50.6	29.4	44.3
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		19.4	29.2	13.2	19.9	22.1	33.2	18.3	27.5	16.0	24.0
Properties											
Area, in.^{2}		5.09		3.90		8.36		6.58		5.62	
r_y, in.		1.63		1.66		1.12		1.18		1.20	
r_x/r_y		1.55		1.54		2.01		1.97		1.98	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

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Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS7–HSS6

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS7x3x				HSS7x2x				HSS6x5x	
		¼		3⁄16[a, c]		¼		3⁄16[a, c]		½	
t_{des}, in.		0.250		0.188		0.250		0.188		0.500	
lb/ft		15.6		12.0		13.9		10.7		31.8	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	137	207	105	158	122	184	93.9	141	280	421
	1	136	205	105	157	121	181	92.9	140	279	420
	2	134	201	103	155	115	173	88.8	134	277	416
	3	129	194	99.6	150	106	159	82.5	124	273	410
	4	123	185	95.0	143	94.8	143	74.3	112	267	401
	5	115	174	89.5	135	82.1	123	64.9	97.6	259	390
	6	107	161	83.2	125	68.9	104	55.1	82.8	251	377
	7	97.7	147	76.4	115	56.0	84.2	45.4	68.2	241	362
	8	88.0	132	69.1	104	44.0	66.1	36.2	54.5	230	346
	9	78.2	118	61.8	92.8	34.8	52.2	28.6	43.0	218	328
	10	68.5	103	54.5	81.8	28.1	42.3	23.2	34.9	206	310
	11	59.2	89.0	47.4	71.2	23.3	35.0	19.2	28.8	193	290
	12	50.3	75.7	40.6	61.1	19.5	29.4	16.1	24.2	180	270
	13	42.9	64.5	34.6	52.0	16.7	25.0	13.7	20.6	167	250
	14	37.0	55.6	29.8	44.9			11.8	17.8	153	230
	15	32.2	48.4	26.0	39.1					140	211
	16	28.3	42.6	22.9	34.3					127	192
	17	25.1	37.7	20.2	30.4					115	173
	18	22.4	33.6	18.1	27.1					103	155
	19	20.1	30.2	16.2	24.4					92.6	139
	20	18.1	27.2	14.6	22.0					83.6	126
	22									69.1	104
	24									58.1	87.3
	26									49.5	74.3
	28									42.7	64.1
	30									37.2	55.8
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		137	207	106	159	122	184	94.3	142	280	421
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		112	168	86.1	129	99.8	150	76.7	115	228	342
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		56.1	84.4	43.5	65.4	56.1	84.4	43.5	65.4	80.8	122
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		20.2	30.4	16.5	24.8	11.2	16.9	9.73	14.6	62.9	94.5
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		24.4	36.7	19.1	28.7	20.2	30.4	15.9	23.9	45.2	67.9
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		13.4	20.1	9.19	13.8	7.96	12.0	5.48	8.24	39.9	60.0
Properties											
Area, in.^{2}		4.59		3.53		4.09		3.15		9.36	
r_y, in.		1.23		1.26		0.812		0.840		1.85	
r_x/r_y		1.96		1.94		2.78		2.74		1.16	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

HSS6

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS6x5x								HSS6x4x	
		3⁄8		5⁄16		1⁄4		3⁄16[a]		1⁄2	
t_{des}, in.		0.375		0.313		0.250		0.188		0.500	
lb/ft		24.9		21.2		17.3		13.3		28.4	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	219	330	187	281	152	229	117	175	250	376
	1	219	329	186	280	152	228	116	175	249	374
	2	217	326	185	278	151	227	116	174	246	369
	3	214	321	182	274	149	224	114	171	240	360
	4	210	315	179	269	146	219	112	168	232	349
	5	204	307	174	262	142	214	109	164	222	334
	6	198	297	169	254	138	208	106	160	211	317
	7	191	286	163	245	133	201	103	154	198	298
	8	182	274	156	235	128	193	98.7	148	185	278
	9	174	261	149	224	122	184	94.3	142	170	256
	10	164	247	141	212	116	175	89.7	135	156	234
	11	155	233	133	200	110	165	84.9	128	141	212
	12	145	218	125	188	103	155	79.9	120	126	190
	13	135	203	116	175	96.3	145	74.8	112	112	169
	14	125	187	108	162	89.5	135	69.7	105	98.8	149
	15	115	172	99.6	150	82.8	124	64.6	97.1	86.1	129
	16	105	158	91.3	137	76.1	114	59.5	89.5	75.7	114
	17	95.3	143	83.2	125	69.6	105	54.6	82.0	67.0	101
	18	86.1	129	75.5	113	63.3	95.1	49.8	74.8	59.8	89.9
	19	77.3	116	67.9	102	57.1	85.8	45.1	67.8	53.7	80.7
	20	69.8	105	61.3	92.1	51.5	77.5	40.7	61.2	48.4	72.8
	22	57.7	86.7	50.6	76.1	42.6	64.0	33.6	50.6	40.0	60.2
	24	48.5	72.8	42.6	64.0	35.8	53.8	28.3	42.5	33.6	50.6
	26	41.3	62.1	36.3	54.5	30.5	45.8	24.1	36.2		
	28	35.6	53.5	31.3	47.0	26.3	39.5	20.8	31.2		
	30	31.0	46.6	27.2	40.9	22.9	34.4	18.1	27.2		
	32			23.9	36.0	20.1	30.3	15.9	23.9		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		219	330	187	281	152	229	117	176	250	376
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		179	268	152	228	124	186	95.2	143	204	306
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		65.7	98.8	56.9	85.5	47.2	70.9	36.7	55.2	80.8	122
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		52.3	78.6	45.7	68.6	38.2	57.4	30.0	45.1	44.9	67.5
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		36.7	55.1	31.7	47.6	26.2	39.4	20.4	30.6	38.4	57.8
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		32.2	48.4	27.9	42.0	23.1	34.7	17.4	26.1	28.7	43.1
Properties											
Area, in.^{2}		7.33		6.24		5.09		3.90		8.36	
r_y, in.		1.91		1.94		1.97		2.00		1.49	
r_x/r_y		1.16		1.15		1.15		1.15		1.38	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

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Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS6

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS6x4x								HSS6x3x	
		⅜		5/16		¼		3/16[a]		½	
t_{des}, in.		0.375		0.313		0.250		0.188		0.500	
lb/ft		22.4		19.1		15.6		12.0		25.0	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	197	296	168	253	137	207	106	159	220	331
	1	196	295	168	252	137	206	105	158	218	328
	2	194	291	165	249	135	203	104	156	213	320
	3	189	285	162	243	132	199	102	153	204	306
	4	183	276	157	236	129	193	99.2	149	192	288
	5	176	265	151	227	124	186	95.7	144	177	266
	6	168	252	144	217	119	178	91.6	138	161	242
	7	158	238	136	205	112	169	87.0	131	144	216
	8	148	223	128	192	106	159	82.0	123	126	190
	9	138	207	119	179	98.5	148	76.7	115	109	164
	10	126	190	110	165	91.1	137	71.1	107	92.3	139
	11	115	173	100	151	83.5	126	65.4	98.3	76.8	115
	12	104	156	91.0	137	76.0	114	59.7	89.8	64.6	97.0
	13	93.0	140	81.7	123	68.6	103	54.1	81.3	55.0	82.7
	14	82.5	124	72.8	109	61.4	92.2	48.6	73.1	47.4	71.3
	15	72.4	109	64.3	96.6	54.5	81.9	43.3	65.1	41.3	62.1
	16	63.6	95.6	56.5	84.9	47.9	72.0	38.2	57.5	36.3	54.6
	17	56.4	84.7	50.0	75.2	42.4	63.8	33.9	50.9	32.2	48.3
	18	50.3	75.6	44.6	67.1	37.9	56.9	30.2	45.4	28.7	43.1
	19	45.1	67.8	40.1	60.2	34.0	51.1	27.1	40.8		
	20	40.7	61.2	36.1	54.3	30.7	46.1	24.5	36.8		
	22	33.7	50.6	29.9	44.9	25.3	38.1	20.2	30.4		
	24	28.3	42.5	25.1	37.7	21.3	32.0	17.0	25.5		
	26			21.4	32.1	18.1	27.3	14.5	21.8		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		197	296	168	253	137	207	106	159	220	331
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		161	241	137	206	112	168	86.1	129	179	269
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		65.7	98.8	56.9	85.5	47.2	70.9	36.7	55.2	80.8	122
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		38.8	58.3	34.4	51.7	29.2	43.9	23.2	34.9	26.9	40.5
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		31.2	46.9	27.2	40.9	22.6	34.0	17.6	26.5	31.4	47.3
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		23.6	35.4	20.5	30.8	17.1	25.7	12.9	19.4	18.9	28.5
Properties											
Area, in.^{2}		6.58		5.62		4.59		3.53		7.36	
r_y, in.		1.54		1.57		1.60		1.63		1.10	
r_x/r_y		1.38		1.38		1.37		1.37		1.76	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS6

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS6x3x								HSS6x2x	
		3/8		5/16		1/4		3/16[a]		3/8	
t_{des}, in.		0.375		0.313		0.250		0.188		0.375	
lb/ft		19.8		17.0		13.9		10.7		17.3	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	175	262	149	225	122	184	94.3	142	152	229
	1	173	260	148	223	122	183	93.7	141	149	224
	2	169	254	145	218	119	179	91.8	138	141	212
	3	163	245	140	210	115	173	88.7	133	128	193
	4	154	231	132	199	109	164	84.5	127	113	169
	5	144	216	124	186	102	154	79.5	119	95.1	143
	6	132	198	114	171	94.5	142	73.7	111	77.4	116
	7	119	179	103	155	86.1	129	67.4	101	60.6	91.1
	8	106	159	92.1	138	77.3	116	60.8	91.5	46.5	69.9
	9	92.6	139	81.0	122	68.4	103	54.2	81.4	36.7	55.2
	10	79.8	120	70.1	105	59.7	89.7	47.6	71.5	29.7	44.7
	11	67.7	102	59.8	89.9	51.3	77.1	41.2	61.9	24.6	37.0
	12	56.9	85.5	50.4	75.7	43.4	65.2	35.1	52.8	20.7	31.0
	13	48.5	72.8	42.9	64.5	37.0	55.6	29.9	45.0		
	14	41.8	62.8	37.0	55.6	31.9	47.9	25.8	38.8		
	15	36.4	54.7	32.2	48.4	27.8	41.8	22.5	33.8		
	16	32.0	48.1	28.3	42.6	24.4	36.7	19.7	29.7		
	17	28.3	42.6	25.1	37.7	21.6	32.5	17.5	26.3		
	18	25.3	38.0	22.4	33.6	19.3	29.0	15.6	23.5		
	19	22.7	34.1	20.1	30.2	17.3	26.0	14.0	21.0		
	20					15.6	23.5	12.6	19.0		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		175	262	149	225	122	184	94.3	142	152	229
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		142	213	122	182	99.8	150	76.7	115	124	186
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		65.7	98.8	56.9	85.5	47.2	70.9	36.7	55.2	65.7	98.8
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		25.3	38.1	23.2	34.8	20.2	30.4	16.5	24.8	11.8	17.7
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		25.9	39.0	22.8	34.2	19.0	28.6	14.9	22.4	20.8	31.2
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		15.8	23.8	13.9	20.9	11.7	17.5	8.91	13.4	9.01	13.5
Properties											
Area, in.^{2}		5.83		4.99		4.09		3.15		5.08	
r_y, in.		1.16		1.18		1.21		1.24		0.749	
r_x/r_y		1.74		1.75		1.73		1.72		2.50	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS6–HSS5

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS6x2x						HSS5x4x			
		5⁄16		¼		3⁄16[a]		½		⅜	
t_{des}, in.		0.313		0.250		0.188		0.500		0.375	
lb/ft		14.8		12.2		9.42		25.0		19.8	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	131	196	107	162	83.2	125	220	331	175	262
	1	128	193	106	159	82.0	123	219	330	174	261
	2	122	183	101	151	78.3	118	216	325	171	257
	3	111	168	92.8	139	72.5	109	211	317	167	252
	4	98.6	148	82.7	124	65.1	97.9	203	306	162	243
	5	84.2	127	71.4	107	56.7	85.3	194	292	155	233
	6	69.4	104	59.6	89.6	47.9	72.1	184	277	147	222
	7	55.3	83.1	48.2	72.4	39.3	59.1	172	259	139	209
	8	42.7	64.2	37.7	56.6	31.2	46.8	160	240	129	194
	9	33.7	50.7	29.8	44.7	24.6	37.0	147	221	119	180
	10	27.3	41.1	24.1	36.2	19.9	30.0	134	201	109	164
	11	22.6	34.0	19.9	29.9	16.5	24.8	120	181	99.1	149
	12	19.0	28.5	16.7	25.2	13.8	20.8	107	161	89.0	134
	13			14.3	21.4	11.8	17.7	94.5	142	79.2	119
	14							82.4	124	69.8	105
	15							71.8	108	60.9	91.5
	16							63.1	94.8	53.5	80.4
	17							55.9	84.0	47.4	71.2
	18							49.9	74.9	42.3	63.5
	19							44.7	67.2	37.9	57.0
	20							40.4	60.7	34.2	51.4
	22							33.4	50.2	28.3	42.5
	24							28.0	42.1	23.8	35.7
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		131	196	107	162	83.2	125	220	331	175	262
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		106	159	87.4	131	67.9	102	179	269	142	213
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		56.9	85.5	47.2	70.9	36.7	55.2	62.9	94.5	52.3	78.6
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		11.9	17.9	11.2	16.9	9.73	14.6	44.9	67.5	38.8	58.3
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		18.3	27.5	15.4	23.2	12.2	18.3	28.7	43.1	23.6	35.4
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		8.06	12.1	6.86	10.3	5.30	7.97	24.4	36.6	20.2	30.3
Properties											
Area, in.^{2}		4.36		3.59		2.78		7.36		5.83	
r_y, in.		0.775		0.802		0.829		1.45		1.50	
r_x/r_y		2.48		2.44		2.41		1.19		1.20	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

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Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS5

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape		HSS5x4x						HSS5x3x			
		5⁄16		¼		3⁄16		½		⅜	
t_{des}, in.		0.313		0.250		0.188		0.500		0.375	
lb/ft		17.0		13.9		10.7		21.6		17.3	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	149	225	122	184	94.3	142	190	286	152	229
	1	149	224	122	183	93.9	141	189	284	151	227
	2	147	221	120	181	92.8	139	184	276	147	221
	3	143	216	118	177	90.8	137	176	264	141	212
	4	139	209	114	172	88.2	133	165	248	133	200
	5	134	201	110	165	85.0	128	152	228	124	186
	6	127	191	105	158	81.2	122	138	207	113	170
	7	120	180	99.1	149	76.9	116	122	184	102	153
	8	112	168	92.8	140	72.2	109	107	161	89.7	135
	9	104	156	86.3	130	67.3	101	91.7	138	78.0	117
	10	95.3	143	79.4	119	62.2	93.5	77.2	116	66.7	100
	11	86.7	130	72.5	109	57.0	85.6	64.0	96.2	56.0	84.1
	12	78.2	117	65.7	98.7	51.8	77.8	53.8	80.8	47.0	70.7
	13	69.9	105	58.9	88.6	46.7	70.1	45.8	68.9	40.1	60.2
	14	61.9	93.0	52.4	78.8	41.7	62.7	39.5	59.4	34.5	51.9
	15	54.2	81.4	46.2	69.4	36.9	55.5	34.4	51.7	30.1	45.2
	16	47.6	71.6	40.6	61.0	32.5	48.8	30.2	45.5	26.4	39.8
	17	42.2	63.4	35.9	54.0	28.8	43.2	26.8	40.3	23.4	35.2
	18	37.6	56.6	32.1	48.2	25.7	38.6	23.9	35.9	20.9	31.4
	19	33.8	50.8	28.8	43.3	23.0	34.6				
	20	30.5	45.8	26.0	39.0	20.8	31.2				
	22	25.2	37.9	21.5	32.3	17.2	25.8				
	24	21.2	31.8	18.0	27.1	14.4	21.7				
	26			15.4	23.1	12.3	18.5				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		149	225	122	184	94.3	142	190	286	152	229
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		122	182	99.8	150	76.7	115	155	233	124	186
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		45.7	68.6	38.2	57.4	30.0	45.1	62.9	94.5	52.3	78.6
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		34.4	51.7	29.2	43.9	23.2	34.9	26.9	40.5	25.3	38.1
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		20.6	30.9	17.2	25.8	13.5	20.3	23.0	34.5	19.2	28.9
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		17.6	26.5	14.7	22.1	11.6	17.4	15.8	23.8	13.3	20.1
Properties											
Area, in.^{2}		4.99		4.09		3.15		6.36		5.08	
r_y, in.		1.53		1.56		1.59		1.08		1.13	
r_x/r_y		1.19		1.19		1.19		1.51		1.50	

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

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HSS5

Table IV-7A (continued)

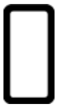
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS5x3x						HSS5x2½x			
		5⁄16		¼		3⁄16		¼		3⁄16	
t_{des}, in.		0.313		0.250		0.188		0.250		0.188	
lb/ft		14.8		12.2		9.42		11.4		8.78	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	131	196	107	162	83.2	125	100	150	77.5	117
	1	130	195	107	160	82.6	124	98.9	149	76.8	115
	2	127	190	104	157	80.9	122	95.8	144	74.5	112
	3	122	183	101	151	78.0	117	90.8	136	70.8	106
	4	115	173	95.4	143	74.2	112	84.2	127	66.0	99.1
	5	107	161	89.3	134	69.5	105	76.5	115	60.2	90.5
	6	98.5	148	82.2	124	64.2	96.6	68.0	102	53.9	81.0
	7	89.0	134	74.7	112	58.5	87.9	59.1	88.9	47.2	71.0
	8	79.1	119	66.8	100	52.5	79.0	50.4	75.7	40.6	61.0
	9	69.3	104	58.9	88.5	46.5	69.9	42.0	63.1	34.2	51.3
	10	59.7	89.7	51.1	76.8	40.5	60.9	34.2	51.5	28.1	42.3
	11	50.6	76.1	43.7	65.7	34.9	52.4	28.3	42.5	23.2	34.9
	12	42.5	63.9	36.9	55.4	29.5	44.3	23.8	35.7	19.5	29.4
	13	36.2	54.5	31.4	47.2	25.1	37.8	20.3	30.4	16.6	25.0
	14	31.2	47.0	27.1	40.7	21.7	32.6	17.5	26.3	14.4	21.6
	15	27.2	40.9	23.6	35.4	18.9	28.4	15.2	22.9	12.5	18.8
	16	23.9	36.0	20.7	31.2	16.6	24.9	13.4	20.1	11.0	16.5
	17	21.2	31.8	18.4	27.6	14.7	22.1			9.73	14.6
	18	18.9	28.4	16.4	24.6	13.1	19.7				
	19	17.0	25.5	14.7	22.1	11.8	17.7				
	20					10.6	16.0				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		131	196	107	162	83.2	125	100	150	77.5	117
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		106	159	87.4	131	67.9	102	81.6	122	63.1	94.6
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		45.7	68.6	38.2	57.4	30.0	45.1	38.2	57.4	30.0	45.1
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		23.2	34.8	20.2	30.4	16.5	24.8	15.7	23.6	13.1	19.7
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		16.9	25.4	14.2	21.4	11.2	16.9	12.7	19.2	10.1	15.2
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		11.8	17.7	9.93	14.9	7.88	11.9	7.78	11.7	6.21	9.34
Properties											
Area, in.^{2}		4.36		3.59		2.78		3.34		2.59	
r_y, in.		1.16		1.19		1.21		0.991		1.02	
r_x/r_y		1.50		1.49		1.50		1.74		1.73	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.
Confirm ASTM A1085 material availability before specifying.



HSS5–HSS4

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS5x2x								HSS4x3x	
		⅜		5⁄16		¼		3⁄16		⅜	
t_{des}, in.		0.375		0.313		0.250		0.188		0.375	
lb/ft		14.7		12.7		10.5		8.15		14.7	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	130	195	112	168	92.5	139	71.9	108	130	195
	1	127	191	110	165	91.0	137	70.7	106	128	193
	2	120	180	104	157	86.5	130	67.5	101	125	188
	3	109	164	95.1	143	79.5	119	62.3	93.7	120	180
	4	95.1	143	83.8	126	70.6	106	55.8	83.9	113	169
	5	79.9	120	71.2	107	60.7	91.2	48.4	72.7	104	156
	6	64.5	97.0	58.3	87.6	50.4	75.8	40.7	61.1	94.2	142
	7	50.1	75.3	46.1	69.2	40.5	60.8	33.1	49.8	84.0	126
	8	38.4	57.7	35.4	53.2	31.5	47.3	26.1	39.2	73.5	111
	9	30.3	45.6	28.0	42.1	24.9	37.4	20.6	31.0	63.2	95.1
	10	24.5	36.9	22.7	34.1	20.1	30.3	16.7	25.1	53.4	80.3
	11	20.3	30.5	18.7	28.2	16.6	25.0	13.8	20.7	44.4	66.7
	12	17.0	25.6	15.7	23.7	14.0	21.0	11.6	17.4	37.3	56.0
	13					11.9	17.9	9.87	14.8	31.8	47.8
	14									27.4	41.2
	15									23.9	35.9
	16									21.0	31.5
	17									18.6	27.9
	18									16.6	24.9
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		130	195	112	168	92.5	139	71.9	108	130	195
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		106	158	91.3	137	75.4	113	58.5	87.8	106	158
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		52.3	78.6	45.7	68.6	38.2	57.4	30.0	45.1	38.8	58.3
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		11.8	17.7	11.9	17.9	11.2	16.9	9.73	14.6	25.3	38.1
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		14.9	22.4	13.2	19.9	11.3	16.9	8.96	13.5	13.4	20.1
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		7.49	11.3	6.74	10.1	5.79	8.70	4.64	6.98	10.9	16.4
Properties											
Area, in.^{2}		4.33		3.74		3.09		2.40		4.33	
r_y, in.		0.737		0.762		0.790		0.816		1.09	
r_x/r_y		2.13		2.13		2.10		2.08		1.27	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.
Confirm ASTM A1085 material availability before specifying.

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Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS4

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape		HSS4x3x						HSS4x2½x			
		5⁄16		¼		3⁄16		⅜		5⁄16	
t_{des}, in.		0.313		0.250		0.188		0.375		0.313	
lb/ft		12.7		10.5		8.15		13.4		11.6	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	112	168	92.5	139	71.9	108	118	178	102	154
	1	111	167	91.8	138	71.3	107	117	176	101	152
	2	108	163	89.6	135	69.7	105	112	169	97.6	147
	3	104	156	86.1	129	67.1	101	105	159	91.9	138
	4	97.9	147	81.4	122	63.7	95.7	96.5	145	84.6	127
	5	90.8	136	75.8	114	59.5	89.4	86.1	129	75.9	114
	6	82.8	124	69.5	104	54.7	82.3	74.8	112	66.6	100
	7	74.2	112	62.6	94.1	49.6	74.6	63.4	95.3	57.0	85.6
	8	65.4	98.4	55.6	83.5	44.3	66.6	52.4	78.8	47.6	71.6
	9	56.7	85.3	48.5	73.0	38.9	58.5	42.2	63.4	38.8	58.3
	10	48.4	72.7	41.7	62.7	33.7	50.7	34.1	51.3	31.4	47.2
	11	40.5	60.8	35.3	53.0	28.8	43.3	28.2	42.4	26.0	39.0
	12	34.0	51.1	29.6	44.5	24.2	36.4	23.7	35.6	21.8	32.8
	13	29.0	43.6	25.2	37.9	20.6	31.0	20.2	30.4	18.6	27.9
	14	25.0	37.6	21.8	32.7	17.8	26.7	17.4	26.2	16.0	24.1
	15	21.8	32.7	19.0	28.5	15.5	23.3	15.2	22.8	14.0	21.0
	16	19.1	28.8	16.7	25.0	13.6	20.5				
	17	16.9	25.5	14.8	22.2	12.1	18.1				
	18	15.1	22.7	13.2	19.8	10.8	16.2				
	19			11.8	17.8	9.66	14.5				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		112	168	92.5	139	71.9	108	118	178	102	154
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		91.3	137	75.4	113	58.5	87.8	96.2	144	83.5	125
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		34.4	51.7	29.2	43.9	23.2	34.9	38.8	58.3	34.4	51.7
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		23.2	34.8	20.2	30.4	16.5	24.8	18.6	27.9	17.5	26.4
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		11.9	17.8	10.1	15.1	8.01	12.0	11.7	17.6	10.4	15.7
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		9.68	14.6	8.23	12.4	6.56	9.86	8.31	12.5	7.46	11.2
Properties											
Area, in.^{2}		3.74		3.09		2.40		3.95		3.42	
r_y, in.		1.12		1.15		1.18		0.910		0.938	
r_x/r_y		1.26		1.26		1.25		1.46		1.46	

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

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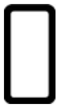
HSS4

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS4x2½x				HSS4x2x					
		¼		3/16		3/8		5/16		¼	
t_{des}, in.		0.250		0.188		0.375		0.313		0.250	
lb/ft		9.66		7.51		12.2		10.6		8.81	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	85.0	128	66.2	99.4	107	161	93.1	140	77.5	117
	1	84.1	126	65.5	98.4	105	158	91.4	137	76.2	115
	2	81.3	122	63.4	95.3	98.8	148	86.3	130	72.2	109
	3	76.8	115	60.1	90.3	89.1	134	78.5	118	66.1	99.4
	4	71.0	107	55.8	83.8	77.2	116	68.7	103	58.4	87.8
	5	64.1	96.4	50.7	76.2	64.2	96.5	57.9	87.0	49.8	74.9
	6	56.6	85.1	45.1	67.7	51.3	77.1	46.9	70.6	41.0	61.6
	7	48.9	73.5	39.2	58.9	39.2	58.9	36.7	55.1	32.6	48.9
	8	41.3	62.1	33.4	50.2	30.0	45.1	28.1	42.2	25.1	37.7
	9	34.1	51.2	27.9	41.9	23.7	35.6	22.2	33.3	19.8	29.8
	10	27.7	41.6	22.7	34.2	19.2	28.9	18.0	27.0	16.1	24.2
	11	22.9	34.4	18.8	28.3	15.9	23.9	14.9	22.3	13.3	20.0
	12	19.2	28.9	15.8	23.7			12.5	18.8	11.2	16.8
	13	16.4	24.6	13.5	20.2						
	14	14.1	21.2	11.6	17.4						
	15	12.3	18.5	10.1	15.2						
	16	10.8	16.2	8.89	13.4						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		85.0	128	66.2	99.5	107	161	93.1	140	77.5	117
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		69.2	104	54.0	80.9	87.4	131	75.7	114	63.1	94.6
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		29.2	43.9	23.2	34.9	38.8	58.3	34.4	51.7	29.2	43.9
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		15.7	23.6	13.1	19.7	11.8	17.7	11.9	17.9	11.2	16.9
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		8.88	13.4	7.11	10.7	9.98	15.0	8.98	13.5	7.71	11.6
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		6.39	9.60	5.11	7.69	5.96	8.96	5.41	8.14	4.69	7.05
Properties											
Area, in.^{2}		2.84		2.21		3.58		3.11		2.59	
r_y, in.		0.966		0.993		0.717		0.744		0.771	
r_x/r_y		1.45		1.45		1.77		1.76		1.75	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.
Confirm ASTM A1085 material availability before specifying.



HSS4–HSS3½

Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape	HSS4x2x		HSS3½x2½x							
	³⁄₁₆		⅜		⁵⁄₁₆		¼		³⁄₁₆	
t_{des}, in.	0.188		0.375		0.313		0.250		0.188	
lb/ft	6.87		12.2		10.6		8.81		6.87	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y: 0	60.5	90.9	107	161	93.1	140	77.5	117	60.5	90.9
1	59.5	89.4	106	159	92.0	138	76.6	115	59.8	89.9
2	56.6	85.1	102	153	88.6	133	74.0	111	57.9	87.0
3	52.1	78.4	95.1	143	83.3	125	69.8	105	54.8	82.3
4	46.5	69.8	86.7	130	76.3	115	64.3	96.6	50.7	76.2
5	40.0	60.2	76.9	116	68.2	103	57.9	87.0	45.9	69.0
6	33.4	50.2	66.5	99.9	59.5	89.4	50.9	76.4	40.7	61.1
7	27.0	40.5	56.0	84.1	50.6	76.1	43.7	65.6	35.2	52.9
8	21.0	31.6	45.9	68.9	42.0	63.1	36.6	55.1	29.9	44.9
9	16.6	25.0	36.6	55.0	33.9	51.0	30.0	45.1	24.8	37.2
10	13.5	20.2	29.7	44.6	27.5	41.3	24.3	36.5	20.1	30.2
11	11.1	16.7	24.5	36.8	22.7	34.1	20.1	30.2	16.6	25.0
12	9.35	14.0	20.6	31.0	19.1	28.7	16.9	25.4	14.0	21.0
13	7.96	12.0	17.6	26.4	16.3	24.4	14.4	21.6	11.9	17.9
14			15.1	22.7	14.0	21.1	12.4	18.6	10.3	15.4
15					12.2	18.4	10.8	16.2	8.94	13.4
16									7.86	11.8
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	60.5	90.9	107	161	93.1	140	77.5	117	60.5	90.9
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	49.4	74.1	87.4	131	75.7	114	63.1	94.6	49.4	74.1
Available Strength in Shear about X-X Axis, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	23.2	34.9	32.1	48.2	28.8	43.3	24.7	37.1	19.9	29.8
Available Strength in Shear about Y-Y Axis, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	9.73	14.6	18.6	27.9	17.5	26.4	15.7	23.6	13.1	19.7
Available Strength in Flexure about X-X Axis, kip-ft	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	6.21	9.34	9.33	14.0	8.38	12.6	7.19	10.8	5.79	8.70
Available Strength in Flexure about Y-Y Axis, kip-ft	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	3.79	5.70	7.34	11.0	6.59	9.90	5.69	8.55	4.57	6.86
Properties										
Area, in.^{2}	2.02		3.58		3.11		2.59		2.02	
r_y, in.	0.799		0.891		0.920		0.948		0.977	
r_x/r_y	1.74		1.31		1.30		1.31		1.30	

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

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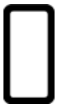


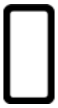
HSS3½–HSS3

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS3½x2x				HSS3½x1½x				HSS3x2½x	
		¼		3/16		¼		3/16		5/16	
t_{des}, in.		0.250		0.188		0.250		0.188		0.313	
lb/ft		7.96		6.23		7.11		5.59		9.51	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	70.1	105	55.1	82.8	62.6	94.0	49.4	74.2	83.8	126
	1	68.8	103	54.2	81.4	60.5	91.0	47.9	72.0	82.7	124
	2	65.1	97.9	51.4	77.3	54.8	82.3	43.7	65.7	79.6	120
	3	59.5	89.4	47.2	71.0	46.4	69.7	37.5	56.4	74.5	112
	4	52.3	78.7	41.9	63.0	36.7	55.2	30.3	45.5	68.0	102
	5	44.4	66.8	35.9	54.0	27.2	40.9	23.0	34.6	60.5	90.9
	6	36.3	54.6	29.7	44.7	19.1	28.8	16.5	24.8	52.4	78.7
	7	28.7	43.1	23.8	35.8	14.1	21.1	12.1	18.2	44.2	66.5
	8	22.0	33.1	18.4	27.7	10.8	16.2	9.27	13.9	36.4	54.6
	9	17.4	26.2	14.6	21.9	8.51	12.8	7.33	11.0	29.1	43.7
	10	14.1	21.2	11.8	17.7					23.6	35.4
	11	11.7	17.5	9.76	14.7					19.5	29.3
	12	9.80	14.7	8.20	12.3					16.4	24.6
	13			6.99	10.5					13.9	21.0
	14									12.0	18.1
	15									10.5	15.7
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		70.1	105	55.1	82.8	62.6	94.1	49.4	74.3	83.8	126
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		57.2	85.8	44.9	67.3	51.0	76.5	40.3	60.5	68.3	102
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		24.7	37.1	19.9	29.8	24.7	37.1	19.9	29.8	23.2	34.8
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		11.2	16.9	9.73	14.6	6.74	10.1	6.32	9.50	17.5	26.4
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		6.19	9.30	5.01	7.54	5.16	7.76	4.24	6.38	6.54	9.83
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		4.14	6.23	3.37	5.06	2.77	4.16	2.29	3.44	5.74	8.63
Properties											
Area, in.^{2}		2.34		1.84		2.09		1.65		2.80	
r_y, in.		0.760		0.784		0.562		0.587		0.898	
r_x/r_y		1.57		1.56		2.01		1.99		1.16	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.
Confirm ASTM A1085 material availability before specifying.



HSS3

Table IV-7A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS3x2½x				HSS3x2x					
		¼		3⁄16		5⁄16		¼		3⁄16	
t_{des}, in.		0.250		0.188		0.313		0.250		0.188	
lb/ft		7.96		6.23		8.45		7.11		5.59	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	70.1	105	55.1	82.8	74.6	112	62.6	94.0	49.4	74.2
	1	69.2	104	54.5	81.9	73.0	110	61.4	92.3	48.5	72.9
	2	66.7	100	52.6	79.1	68.6	103	58.0	87.1	46.0	69.2
	3	62.7	94.3	49.7	74.6	61.9	93.0	52.7	79.2	42.1	63.3
	4	57.6	86.6	45.8	68.9	53.6	80.5	46.1	69.3	37.2	55.9
	5	51.6	77.5	41.3	62.1	44.5	66.9	38.8	58.3	31.7	47.7
	6	45.1	67.7	36.4	54.7	35.4	53.3	31.4	47.2	26.1	39.2
	7	38.4	57.8	31.3	47.1	27.0	40.6	24.5	36.8	20.7	31.2
	8	32.0	48.1	26.4	39.6	20.7	31.1	18.8	28.2	16.0	24.0
	9	25.9	38.9	21.7	32.6	16.4	24.6	14.8	22.3	12.6	19.0
	10	21.0	31.5	17.6	26.4	13.2	19.9	12.0	18.1	10.2	15.4
	11	17.3	26.1	14.5	21.8	11.0	16.5	9.93	14.9	8.46	12.7
	12	14.6	21.9	12.2	18.3			8.34	12.5	7.11	10.7
	13	12.4	18.7	10.4	15.6						
	14	10.7	16.1	8.96	13.5						
	15	9.33	14.0	7.80	11.7						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		70.1	105	55.1	82.8	74.6	112	62.6	94.1	49.4	74.3
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		57.2	85.8	44.9	67.3	60.8	91.2	51.0	76.5	40.3	60.5
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		20.2	30.4	16.5	24.8	23.2	34.8	20.2	30.4	16.5	24.8
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		15.7	23.6	13.1	19.7	11.9	17.9	11.2	16.9	9.73	14.6
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		5.66	8.51	4.59	6.90	5.49	8.25	4.79	7.20	3.92	5.89
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		4.97	7.46	4.04	6.08	4.09	6.15	3.59	5.40	2.94	4.43
Properties											
Area, in.^{2}		2.34		1.84		2.49		2.09		1.65	
r_y, in.		0.927		0.956		0.714		0.742		0.771	
r_x/r_y		1.15		1.15		1.39		1.39		1.37	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.
Confirm ASTM A1085 material availability before specifying.

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Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS3–HSS2½

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS3x1½x				HSS3x1x		HSS2½x2x			
		¼		3⁄16		3⁄16		¼		3⁄16	
t_{des}, in.		0.250		0.188		0.188		0.250		0.188	
lb/ft		6.26		4.96		4.32		6.26		4.96	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	55.1	82.8	43.7	65.7	38.0	57.1	55.1	82.8	43.7	65.7
	1	53.2	80.0	42.4	63.7	35.3	53.0	54.0	81.1	42.9	64.5
	2	48.0	72.1	38.5	57.9	28.1	42.3	50.8	76.4	40.6	61.0
	3	40.4	60.7	32.9	49.5	19.3	29.0	46.0	69.1	37.0	55.6
	4	31.7	47.6	26.4	39.7	11.6	17.4	39.9	60.0	32.5	48.8
	5	23.2	34.9	19.9	29.9	7.42	11.1	33.3	50.0	27.4	41.2
	6	16.3	24.4	14.1	21.3	5.15	7.74	26.7	40.1	22.4	33.6
	7	11.9	18.0	10.4	15.6			20.5	30.8	17.6	26.4
	8	9.14	13.7	7.96	12.0			15.7	23.6	13.5	20.2
	9	7.22	10.9	6.29	9.45			12.4	18.6	10.6	16.0
	10							10.0	15.1	8.62	13.0
	11							8.30	12.5	7.12	10.7
	12							6.97	10.5	5.98	9.00
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		55.1	82.8	43.7	65.7	38.0	57.2	55.1	82.8	43.7	65.7
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		44.9	67.3	35.8	53.6	31.0	46.5	44.9	67.3	35.8	53.6
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		20.2	30.4	16.5	24.8	16.5	24.8	15.7	23.6	13.1	19.7
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		6.74	10.1	6.32	9.50	2.94	4.43	11.2	16.9	9.73	14.6
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		3.94	5.93	3.27	4.91	2.59	3.90	3.57	5.36	2.94	4.43
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		2.37	3.56	1.98	2.98	1.13	1.70	3.04	4.58	2.52	3.79
Properties											
Area, in.^{2}		1.84		1.46		1.27		1.84		1.46	
r_y, in.		0.552		0.578		0.374		0.723		0.752	
r_x/r_y		1.76		1.75		2.51		1.20		1.19	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.
Confirm ASTM A1085 material availability before specifying.

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Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS2½–HSS2

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS2½x1½x				HSS2½x1x		HSS2¼x2x		HSS2x1½x	
		¼		3⁄16		3⁄16		3⁄16		3⁄16	
t_{des}, in.		0.250		0.188		0.188		0.188		0.188	
lb/ft		5.41		4.32		3.68		4.64		3.68	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	47.6	71.5	38.0	57.1	32.3	48.6	41.0	61.6	32.3	48.6
	1	45.9	69.0	36.8	55.3	29.9	45.0	40.2	60.5	31.2	46.9
	2	41.2	61.9	33.3	50.1	23.7	35.7	38.0	57.1	28.1	42.3
	3	34.3	51.6	28.3	42.5	16.1	24.2	34.5	51.8	23.6	35.5
	4	26.6	40.0	22.5	33.8	9.59	14.4	30.1	45.3	18.5	27.8
	5	19.2	28.8	16.7	25.1	6.14	9.23	25.3	38.1	13.5	20.3
	6	13.3	20.1	11.8	17.7	4.26	6.41	20.5	30.8	9.44	14.2
	7	9.80	14.7	8.67	13.0			15.9	24.0	6.93	10.4
	8	7.51	11.3	6.64	9.97			12.2	18.3	5.31	7.98
	9			5.24	7.88			9.64	14.5	4.19	6.30
	10							7.81	11.7		
	11							6.45	9.70		
	12							5.42	8.15		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		47.6	71.6	38.0	57.2	32.3	48.6	41.0	61.7	32.3	48.6
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		38.7	58.0	31.0	46.5	26.3	39.5	33.5	50.2	26.3	39.5
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		15.7	23.6	13.1	19.7	13.1	19.7	11.4	17.2	9.73	14.6
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		6.74	10.1	6.32	9.50	2.94	4.43	9.73	14.6	6.32	9.50
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		2.87	4.31	2.41	3.62	1.87	2.81	2.52	3.79	1.67	2.52
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		1.98	2.97	1.67	2.51	0.938	1.41	2.31	3.48	1.36	2.05
Properties											
Area, in.^{2}		1.59		1.27		1.08		1.37		1.08	
r_y, in.		0.538		0.566		0.369		0.739		0.549	
r_x/r_y		1.52		1.51		2.14		1.10		1.26	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.
Confirm ASTM A1085 material availability before specifying.

Table IV-7A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS2

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape		HSS2x1x 3⁄16	
t_{des}, in.		0.188	
lb/ft		3.04	
Design		**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	26.8	40.3
	1	24.7	37.1
	2	19.3	29.0
	3	12.8	19.2
	4	7.49	11.3
	5	4.79	7.21
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$
		26.8	40.3
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$
		21.8	32.8
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$
		9.73	14.6
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$
		2.94	4.43
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$
		1.25	1.88
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$
		0.749	1.13

Properties

Area, in.^{2}	0.896
r_y, in.	0.358
r_x/r_y	1.77

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Confirm ASTM A1085 material availability before specifying.

Table IV-7B

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS24–HSS20

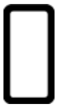
A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS24x12x						HSS20x12x			
		¾^{a}		⅝$^{a, c}$		½$^{a, c}$		¾		⅝^{a}	
t_{des}, in.		0.698		0.581		0.465		0.698		0.581	
lb/ft		171		144		117		151		127	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	1410	2120	1120	1690	819	1230	1240	1870	1050	1570
	1	1410	2120	1120	1690	819	1230	1240	1870	1050	1570
	2	1410	2120	1120	1690	818	1230	1240	1860	1050	1570
	3	1400	2110	1120	1680	817	1230	1240	1860	1040	1570
	4	1400	2110	1120	1680	815	1230	1230	1850	1040	1560
	5	1400	2100	1110	1670	813	1220	1230	1850	1040	1560
	6	1390	2090	1110	1670	810	1220	1220	1840	1030	1550
	7	1380	2080	1100	1660	807	1210	1220	1830	1030	1540
	8	1370	2060	1100	1650	803	1210	1210	1820	1020	1530
	9	1360	2050	1090	1640	799	1200	1200	1800	1010	1520
	10	1350	2030	1090	1630	794	1190	1190	1790	1000	1510
	11	1340	2010	1080	1620	789	1190	1180	1770	994	1490
	12	1330	1990	1070	1610	783	1180	1170	1750	985	1480
	13	1310	1970	1060	1600	777	1170	1150	1730	974	1460
	14	1300	1950	1050	1580	770	1160	1140	1710	963	1450
	15	1280	1930	1040	1570	763	1150	1120	1690	951	1430
	16	1260	1900	1030	1550	756	1140	1110	1670	938	1410
	17	1250	1870	1020	1540	748	1120	1090	1640	925	1390
	18	1230	1850	1010	1520	740	1110	1080	1620	911	1370
	19	1210	1820	997	1500	731	1100	1060	1590	896	1350
	20	1190	1790	985	1480	722	1090	1040	1560	881	1320
	22	1150	1730	958	1440	703	1060	1000	1510	850	1280
	24	1100	1660	929	1400	683	1030	963	1450	816	1230
	26	1060	1590	895	1350	662	995	922	1390	782	1180
	28	1010	1520	856	1290	639	961	879	1320	746	1120
	30	962	1450	815	1230	616	926	835	1250	710	1070
	32	913	1370	774	1160	592	890	790	1190	672	1010
	34	863	1300	733	1100	568	854	745	1120	635	955
	36	813	1220	691	1040	543	816	701	1050	598	898
	38	764	1150	650	977	518	778	656	986	561	843
	40	715	1070	609	916	492	740	612	921	524	788
	42	667	1000	569	855	467	702	570	856	488	734
	44	620	932	530	796	436	656	528	793	453	681
	46	574	863	492	739	405	609	488	733	419	630
	48	529	795	454	682	375	564	448	673	385	579
	50	488	733	418	629	346	520	413	620	355	534
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		1410	2120	1190	1780	961	1440	1240	1870	1050	1580
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		1100	1640	921	1380	746	1120	965	1450	814	1220
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		549	825	465	700	378	567	449	675	382	574
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		249	374	215	323	177	266	249	374	215	323
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		896	1350	758	1140	619	930	674	1010	574	863
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		509	765	391	587	287	431	474	713	371	557
Properties											
Area, in.^{2}		47.1		39.6		32.1		41.5		35.0	
r_y, in.		4.98		5.03		5.08		4.88		4.93	
r_x/r_y		1.72		1.71		1.71		1.49		1.49	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Note: Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.



HSS20

Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS20x12x						HSS20x8x			
		½[a, c]		⅜[a, b, c]		5⁄16[a, b, c]		⅝[a]		½[a, c]	
t_{des}, in.		0.465		0.349		0.291		0.581		0.465	
lb/ft		103		78.5		65.9		110		89.7	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	792	1190	528	794	397	597	907	1360	681	1020
	1	792	1190	528	794	397	597	906	1360	681	1020
	2	791	1190	528	793	397	596	904	1360	679	1020
	3	790	1190	527	792	396	596	900	1350	677	1020
	4	788	1180	526	790	396	594	894	1340	674	1010
	5	785	1180	524	788	395	593	886	1330	670	1010
	6	783	1180	522	785	393	591	877	1320	664	999
	7	779	1170	520	782	392	589	866	1300	658	990
	8	775	1170	518	778	391	587	854	1280	651	979
	9	771	1160	515	774	389	584	840	1260	644	968
	10	766	1150	511	769	387	582	825	1240	635	955
	11	760	1140	508	764	385	578	809	1220	626	941
	12	755	1130	504	758	383	575	792	1190	616	926
	13	748	1120	500	752	380	571	773	1160	605	909
	14	741	1110	496	745	377	567	754	1130	594	892
	15	734	1100	491	738	375	563	734	1100	582	874
	16	727	1090	486	731	371	558	712	1070	569	855
	17	718	1080	481	723	368	553	691	1040	556	835
	18	710	1070	475	715	365	548	668	1000	542	815
	19	701	1050	470	706	361	543	645	970	528	793
	20	692	1040	464	697	358	537	622	935	511	767
	22	672	1010	451	678	350	526	575	864	473	711
	24	652	980	438	658	341	513	527	792	435	653
	26	630	947	424	637	331	497	479	720	396	596
	28	607	912	409	615	319	480	433	651	359	540
	30	579	870	394	592	308	462	388	583	323	485
	32	550	826	378	568	296	444	345	518	288	433
	34	520	781	362	544	283	426	305	459	255	384
	36	490	736	346	520	271	407	272	409	228	342
	38	460	692	329	495	258	388	244	367	204	307
	40	431	647	313	470	245	369	221	331	184	277
	42	402	604	296	446	233	350	200	301	167	251
	44	374	562	280	421	220	331	182	274	152	229
	46	346	520	264	396	208	312	167	251	139	210
	48	319	480	247	372	195	293	153	230	128	192
	50	294	442	228	343	183	275	141	212	118	177
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		847	1270	644	968	542	815	907	1360	737	1110
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		658	987	500	750	421	631	704	1060	572	858
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		311	467	238	358	180	271	382	574	311	467
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		177	266	138	207	116	174	131	196	110	166
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		469	705	319	480	242	364	462	694	379	570
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		271	407	180	270	139	209	221	332	162	243
Properties											
Area, in.^{2}		28.3		21.5		18.1		30.3		24.6	
r_y, in.		4.99		5.04		5.07		3.34		3.39	
r_x/r_y		1.48		1.48		1.48		2.06		2.05	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Note: Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS20

A500 Gr. C

F_y = 50 ksi

F_u = 62 ksi

Shape		HSS20x8x				HSS20x4x					
		$\frac{3}{8}^{a,\,c}$		$\frac{5}{16}^{a,\,b,\,c}$		$\frac{1}{2}^{a,\,c}$		$\frac{3}{8}^{a,\,c}$		$\frac{5}{16}^{a,\,b,\,c}$	
t_{des}, in.		0.349		0.291		0.465		0.349		0.291	
lb/ft		68.3		57.4		76.1		58.1		48.9	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	445	668	340	511	571	858	364	547	271	408
	1	444	668	340	511	569	855	363	545	271	407
	2	443	667	339	510	565	848	360	542	269	404
	3	442	664	338	508	557	837	356	535	265	399
	4	440	661	337	506	547	822	350	526	261	392
	5	437	657	335	503	533	802	342	514	256	384
	6	434	652	332	499	518	778	333	501	249	374
	7	430	647	329	495	500	751	323	485	241	363
	8	426	640	326	490	480	721	311	468	233	350
	9	421	633	322	485	458	688	298	448	224	336
	10	416	625	318	479	431	648	285	428	214	322
	11	410	617	314	472	398	599	270	406	204	306
	12	404	607	309	465	366	550	255	383	193	290
	13	397	597	304	457	333	501	240	360	181	273
	14	390	586	299	449	301	453	224	336	170	256
	15	383	575	293	441	270	406	208	312	159	238
	16	375	563	287	432	241	361	192	288	147	221
	17	367	551	281	423	213	320	173	260	135	203
	18	358	538	275	413	190	286	154	232	125	187
	19	349	525	268	403	171	256	138	208	115	173
	20	340	512	262	393	154	231	125	188	107	161
	22	322	484	248	372	127	191	103	155	88.5	133
	24	302	455	233	350	107	161	86.8	130	74.4	112
	26	283	425	218	328	91.1	137	73.9	111	63.4	95.2
	28	263	395	203	306	78.5	118	63.8	95.8	54.6	82.1
	30	243	365	188	283						
	32	223	335	174	261						
	34	200	300	159	239						
	36	178	268	145	219						
	38	160	240	134	201						
	40	144	217	123	185						
	42	131	197	112	168						
	44	119	179	102	153						
	46	109	164	93.3	140						
	48	100	151	85.6	129						
	50	92.4	139	78.9	119						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		560	842	470	707	626	941	479	720	401	603
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		435	652	365	548	486	729	372	558	312	467
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		238	358	180	271	311	467	238	358	180	271
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		87.1	131	74.5	112	43.4	65.3	37.0	55.6	32.7	49.2
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		292	439	241	363	287	431	223	335	184	277
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		107	160	81.6	123	66.6	100	44.5	66.9	34.0	51.1
Properties											
Area, in.^{2}		18.7		15.7		20.9		16.0		13.4	
r_y, in.		3.44		3.47		1.68		1.73		1.75	
r_x/r_y		2.04		2.04		3.77		3.71		3.69	

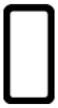
[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.



HSS20–HSS18

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS20x4x		HSS18x6x							
		$\frac{1}{4}^{a, b, c}$		$\frac{5}{8}^{a}$		$\frac{1}{2}^{a, c}$		$\frac{3}{8}^{a, c}$		$\frac{5}{16}^{a, b, c}$	
t_{des}, in.		0.233		0.581		0.465		0.349		0.291	
lb/ft		39.4		93.3		76.1		58.1		48.9	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	189	283	769	1160	609	916	399	599	302	454
	1	188	283	768	1150	609	915	398	598	302	453
	2	187	281	764	1150	606	911	397	596	301	452
	3	185	278	758	1140	602	906	394	593	299	449
	4	182	273	749	1130	597	898	391	588	297	446
	5	178	268	737	1110	590	887	387	582	293	441
	6	174	261	723	1090	582	875	382	575	290	436
	7	169	253	708	1060	573	861	376	566	286	429
	8	163	245	690	1040	562	845	370	556	281	422
	9	157	236	670	1010	548	823	363	545	275	414
	10	150	226	648	975	531	798	355	533	270	405
	11	143	215	625	940	513	771	346	520	263	396
	12	136	204	601	904	494	742	337	506	256	385
	13	128	193	576	866	474	712	327	492	249	375
	14	121	181	550	827	453	681	317	476	242	363
	15	113	170	523	787	432	650	306	460	234	352
	16	105	158	496	746	411	617	295	444	226	339
	17	97.3	146	469	705	389	585	284	427	217	327
	18	89.9	135	442	664	367	552	272	409	209	314
	19	83.3	125	415	623	346	519	260	391	200	301
	20	77.5	116	388	583	324	487	248	373	192	288
	22	67.7	102	336	505	282	424	223	335	174	261
	24	59.7	89.7	286	431	242	364	193	289	156	235
	26	52.8	79.4	244	367	207	310	164	247	139	209
	28	45.6	68.5	210	316	178	268	142	213	122	183
	30			183	276	155	233	124	186	106	159
	32			161	242	136	205	109	163	93.0	140
	34			143	215	121	182	96.2	145	82.4	124
	36			127	191	108	162	85.8	129	73.5	110
	38			114	172	96.7	145	77.0	116	66.0	99.2
	40			103	155	87.3	131	69.5	104	59.6	89.5
	42					79.2	119	63.0	94.7	54.0	81.2
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		323	486	769	1160	626	941	479	720	401	603
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		251	377	598	896	486	729	372	558	312	467
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		103	155	340	511	277	417	213	320	179	269
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		27.6	41.5	88.9	134	77.0	116	62.1	93.3	53.6	80.6
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		141	212	337	506	279	420	216	324	182	274
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		24.1	36.2	149	224	108	163	72.0	108	55.5	83.4
Properties											
Area, in.^{2}		10.8		25.7		20.9		16.0		13.4	
r_y, in.		1.78		2.48		2.53		2.58		2.61	
r_x/r_y		3.65		2.42		2.40		2.38		2.37	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.
[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS18–HSS16

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS18x6x		HSS16x12x							
		¼[a, b, c]		¾		⅝		½[a]		⅜[a, b, c]	
t_{des}, in.		0.233		0.698		0.581		0.465		0.349	
lb/ft		39.4		130		110		89.7		68.3	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	215	322	1070	1620	907	1360	737	1110	512	770
	1	214	322	1070	1610	907	1360	736	1110	512	770
	2	214	321	1070	1610	906	1360	735	1110	512	769
	3	212	319	1070	1610	903	1360	734	1100	511	768
	4	211	317	1070	1600	901	1350	731	1100	510	766
	5	209	314	1060	1600	897	1350	728	1090	508	763
	6	206	310	1060	1590	892	1340	725	1090	506	760
	7	203	305	1050	1580	887	1330	721	1080	504	757
	8	200	300	1040	1570	881	1320	716	1080	501	753
	9	196	295	1030	1560	874	1310	710	1070	498	748
	10	192	288	1030	1540	867	1300	704	1060	495	743
	11	187	282	1020	1530	858	1290	698	1050	491	738
	12	183	275	1000	1510	849	1280	691	1040	487	732
	13	178	267	993	1490	840	1260	683	1030	483	725
	14	172	259	981	1470	829	1250	675	1010	478	718
	15	167	251	968	1450	819	1230	666	1000	473	711
	16	161	242	954	1430	807	1210	657	988	468	703
	17	155	234	939	1410	795	1190	647	973	462	695
	18	149	225	924	1390	782	1180	637	958	457	687
	19	143	216	908	1370	769	1160	627	942	451	678
	20	137	206	892	1340	756	1140	616	926	445	668
	22	125	188	858	1290	727	1090	594	892	431	648
	24	113	170	822	1230	697	1050	570	856	418	628
	26	101	151	784	1180	666	1000	545	819	403	606
	28	90.2	136	746	1120	634	953	519	780	388	583
	30	81.4	122	706	1060	601	904	493	741	372	559
	32	73.8	111	667	1000	568	854	467	701	356	534
	34	67.3	101	627	942	535	804	440	661	338	508
	36	60.2	90.4	587	882	502	754	413	621	318	478
	38	54.0	81.2	548	823	469	705	387	582	298	448
	40	48.7	73.2	509	766	437	656	361	542	278	418
	42	44.2	66.4	472	709	405	609	335	504	259	389
	44			435	655	375	563	311	467	240	361
	46			400	601	344	518	287	431	222	334
	48			367	552	316	475	263	396	204	307
	50			338	508	291	438	243	365	188	283
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		323	486	1070	1620	907	1360	737	1110	560	842
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		251	377	835	1250	704	1060	572	858	435	652
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		115	173	349	524	299	449	244	367	188	283
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		44.4	66.7	249	374	215	323	177	266	138	207
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		142	213	482	724	412	619	337	506	232	349
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		39.5	59.3	394	593	337	506	254	381	169	254
Properties											
Area, in.^{2}		10.8		35.9		30.3		24.6		18.7	
r_y, in.		2.63		4.75		4.80		4.86		4.91	
r_x/r_y		2.37		1.25		1.25		1.25		1.25	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS16

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS16x12x		HSS16x8x							
		5⁄16[a, b, c]		5⁄8		1⁄2[a]		3⁄8[a, c]		5⁄16[a, c]	
t_{des}, in.		0.291		0.581		0.465		0.349		0.291	
lb/ft		57.4		93.3		76.1		58.1		48.9	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	386	580	769	1160	626	940	432	649	332	499
	1	386	580	769	1160	625	940	431	648	332	498
	2	386	580	766	1150	623	937	430	647	331	497
	3	385	579	763	1150	620	932	429	645	330	496
	4	384	578	757	1140	616	926	427	641	328	493
	5	383	576	751	1130	611	918	424	637	326	490
	6	382	574	743	1120	605	909	421	632	324	486
	7	381	572	733	1100	597	897	417	626	321	482
	8	379	570	722	1090	589	885	412	619	317	477
	9	377	567	710	1070	579	870	407	612	314	471
	10	375	564	697	1050	569	855	402	604	309	465
	11	373	561	683	1030	557	838	396	594	305	458
	12	371	557	668	1000	545	820	389	585	300	451
	13	368	553	652	979	532	800	382	574	295	443
	14	365	549	634	954	519	780	375	563	289	435
	15	362	545	617	927	505	759	367	551	283	426
	16	359	540	598	899	490	736	359	539	277	417
	17	356	535	579	870	475	714	350	526	271	407
	18	352	530	559	841	459	690	341	513	264	397
	19	349	524	539	811	443	666	332	499	257	387
	20	345	518	519	780	427	642	323	485	250	376
	22	337	506	478	718	394	592	304	456	236	355
	24	328	492	436	656	361	543	281	422	221	332
	26	316	475	395	594	328	493	256	385	206	309
	28	305	458	356	534	296	445	232	348	190	286
	30	292	440	317	477	265	398	208	313	175	263
	32	280	421	280	421	235	353	185	278	158	237
	34	267	402	248	373	208	313	164	247	140	210
	36	255	383	221	333	186	279	146	220	125	188
	38	242	363	199	299	167	250	131	197	112	168
	40	229	344	179	269	150	226	119	178	101	152
	42	216	324	163	244	136	205	108	162	91.7	138
	44	203	305	148	223	124	187	98.0	147	83.5	126
	46	189	284	136	204	114	171	89.6	135	76.4	115
	48	174	261	124	187	104	157	82.3	124	70.2	105
	50	160	240	115	172	96.2	145	75.9	114	64.7	97.2
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		470	707	769	1160	626	941	479	720	401	603
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		365	548	598	896	486	729	372	558	312	467
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		158	237	299	449	244	367	188	283	158	237
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		116	174	131	196	110	166	87.1	131	74.5	112
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		178	267	322	484	264	398	205	308	173	260
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		131	197	198	297	151	226	100	151	77.7	117
Properties											
Area, in.^{2}		15.7		25.7		20.9		16.0		13.4	
r_y, in.		4.94		3.27		3.32		3.37		3.40	
r_x/r_y		1.24		1.72		1.72		1.71		1.71	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.
[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
Note: Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS16

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS16x8x		HSS16x4x							
		¼[a, b, c]		⅝		½[a]		⅜[a, c]		5⁄16[a, c]	
t_{des}, in.		0.233		0.581		0.465		0.349		0.291	
lb/ft		39.4		76.3		62.5		47.9		40.4	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	240	360	629	945	515	774	348	523	263	395
	1	240	360	626	941	513	771	347	521	262	394
	2	239	359	618	930	507	762	344	517	260	391
	3	238	358	606	911	497	748	340	511	257	387
	4	237	356	589	885	484	728	334	501	253	380
	5	236	354	567	853	468	703	326	490	247	371
	6	234	352	542	815	448	673	317	476	240	361
	7	232	348	514	773	426	640	306	460	233	350
	8	229	345	483	726	402	604	294	442	224	337
	9	227	341	451	677	376	566	281	422	214	322
	10	224	336	417	626	350	526	267	401	204	307
	11	220	331	382	575	323	485	252	379	194	291
	12	217	326	348	523	295	443	235	354	183	275
	13	213	320	314	472	268	403	215	323	171	257
	14	209	314	281	422	241	363	195	293	160	240
	15	205	308	249	375	216	324	176	264	148	222
	16	201	302	219	329	191	287	157	236	135	203
	17	196	295	194	292	169	254	139	210	120	180
	18	191	288	173	260	151	227	124	187	107	161
	19	187	280	155	234	135	204	112	168	96.1	144
	20	182	273	140	211	122	184	101	151	86.7	130
	22	171	258	116	174	101	152	83.2	125	71.6	108
	24	161	242	97.4	146	84.9	128	69.9	105	60.2	90.5
	26	150	225	83.0	125	72.3	109	59.6	89.6	51.3	77.1
	28	139	209					51.4	77.2	44.2	66.5
	30	128	193								
	32	118	177								
	34	107	161								
	36	98.0	147								
	38	90.0	135								
	40	82.4	124								
	42	74.7	112								
	44	68.1	102								
	46	62.3	93.7								
	48	57.2	86.0								
	50	52.7	79.3								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		323	486	629	945	515	774	395	594	332	500
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		251	377	488	732	400	600	307	460	258	387
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		115	174	299	449	244	367	188	283	158	237
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		61.1	91.8	47.2	70.9	43.4	65.3	37	55.6	32.7	49.2
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		124	187	232	348	193	290	150	226	127	192
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		55.9	84.0	81.1	122	62.0	93.2	41.8	62.8	32.3	48.5
Properties											
Area, in.^{2}		10.8		21.0		17.2		13.2		11.1	
r_y, in.		3.42		1.60		1.65		1.71		1.73	
r_x/r_y		1.70		3.16		3.12		3.06		3.05	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS16

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS16x4x				HSS14x10x					
		¼$^{a, b, c}$		$\frac{3}{16}^{a, b, c}$		⅝		½^{a}		⅜$^{a, c}$	
t_{des}, in.		0.233		0.174		0.581		0.465		0.349	
lb/ft		32.6		24.7		93.3		76.1		58.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	185	278	117	175	769	1160	626	940	462	695
	1	184	277	116	175	769	1160	625	940	462	695
	2	183	275	115	173	767	1150	624	938	461	694
	3	181	272	114	171	765	1150	622	935	460	692
	4	178	267	112	169	761	1140	619	931	459	689
	5	174	261	110	165	757	1140	616	925	457	686
	6	169	255	107	161	751	1130	611	919	454	682
	7	164	247	104	156	745	1120	606	911	451	678
	8	158	238	100	151	737	1110	600	902	447	672
	9	152	229	96.2	145	729	1100	594	893	444	667
	10	145	218	92.0	138	720	1080	587	882	439	660
	11	138	208	87.6	132	710	1070	579	870	434	653
	12	131	196	83.0	125	699	1050	570	857	429	645
	13	123	185	78.3	118	688	1030	561	843	424	637
	14	115	173	73.5	110	675	1020	551	829	418	628
	15	107	161	68.6	103	663	996	541	813	412	619
	16	99.5	149	63.8	95.9	649	976	530	797	405	609
	17	91.5	138	59.0	88.7	635	954	519	781	398	599
	18	84.2	127	54.4	81.8	620	932	508	763	391	587
	19	77.7	117	50.4	75.7	605	910	496	745	382	574
	20	72.0	108	46.8	70.4	590	886	483	727	372	560
	22	59.9	90.0	40.8	61.4	558	838	458	688	353	531
	24	50.3	75.6	36.0	54.1	525	789	432	649	333	501
	26	42.9	64.4	32.0	48.1	491	738	405	608	313	470
	28	37.0	55.5	28.5	42.9	457	687	377	567	292	440
	30					423	636	350	526	272	409
	32					390	586	323	486	251	378
	34					357	536	297	446	231	348
	36					325	489	271	408	212	318
	38					294	442	247	371	193	290
	40					266	399	223	334	175	262
	42					241	362	202	303	158	238
	44					219	330	184	276	144	217
	46					201	302	168	253	132	198
	48					184	277	155	232	121	182
	50					170	255	142	214	112	168
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		268	403	202	304	769	1160	626	941	479	720
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		208	312	157	236	598	896	486	729	372	558
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		115	174	53.4	80.3	257	386	211	316	163	245
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		27.6	41.5	21.8	32.7	172	259	144	216	112	169
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		102	153	71.8	108	299	450	247	371	190	286
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		23.1	34.8	14.7	22.0	237	357	195	294	128	192
Properties											
Area, in.^{2}		8.96		6.76		25.7		20.9		16.0	
r_y, in.		1.76		1.78		3.98		4.04		4.09	
r_x/r_y		3.02		3.01		1.30		1.29		1.29	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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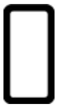

HSS14

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS14x10x				HSS14x6x					
		5⁄16[a, b, c]		1⁄4[a, b, c]		5⁄8		1⁄2[a]		3⁄8[a, c]	
t_{des}, in.		0.291		0.233		0.581		0.465		0.349	
lb/ft		48.9		39.4		76.3		62.5		47.9	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	360	542	251	377	629	945	515	774	379	569
	1	360	541	251	377	628	943	514	773	378	568
	2	360	541	250	376	624	938	511	769	377	566
	3	359	539	250	376	619	930	507	762	374	562
	4	357	537	249	375	611	918	501	753	371	557
	5	356	535	248	373	601	904	493	742	367	551
	6	354	532	247	372	590	886	484	728	362	543
	7	352	528	246	370	576	866	474	712	356	534
	8	349	524	245	368	561	843	462	694	349	524
	9	346	520	243	365	544	818	448	674	341	513
	10	343	515	241	363	526	791	434	652	333	500
	11	339	510	239	360	507	762	419	629	324	487
	12	335	504	237	356	486	731	402	605	312	469
	13	331	498	235	353	465	699	386	580	299	450
	14	327	491	232	349	443	666	368	553	286	430
	15	322	484	230	345	421	633	350	527	273	410
	16	317	476	227	341	398	599	332	499	259	390
	17	312	468	224	336	376	564	314	472	246	369
	18	306	460	221	332	353	530	296	444	232	349
	19	301	452	217	327	330	496	278	417	218	328
	20	295	443	214	322	308	463	260	390	205	308
	22	283	425	207	311	265	399	225	338	178	268
	24	270	405	199	299	225	338	192	288	153	230
	26	256	385	189	284	191	288	163	246	130	196
	28	243	365	179	270	165	248	141	212	112	169
	30	229	344	169	255	144	216	123	184	98.0	147
	32	213	319	159	239	126	190	108	162	86.1	129
	34	196	294	149	224	112	168	95.5	144	76.3	115
	36	180	270	139	209	99.9	150	85.2	128	68.1	102
	38	164	246	129	194	89.6	135	76.5	115	61.1	91.8
	40	148	223	119	179	80.9	122	69.0	104	55.1	82.8
	42	135	202	110	165					50.0	75.1
	44	123	184	99.8	150						
	46	112	169	91.3	137						
	48	103	155	83.9	126						
	50	95.0	143	77.3	116						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		401	603	323	486	629	945	515	774	395	594
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		312	467	251	377	488	732	400	600	307	460
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		137	206	111	167	257	386	211	316	163	245
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		95.5	143	77.9	117	88.9	134	77.0	116	62.1	93.3
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		144	217	104	157	221	333	184	276	143	215
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		99.4	149	72.7	109	121	182	101	151	66.6	100
Properties											
Area, in.^{2}		13.4		10.8		21.0		17.2		13.2	
r_y, in.		4.12		4.14		2.43		2.48		2.53	
r_x/r_y		1.29		1.29		1.96		1.95		1.94	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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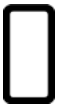

HSS14

Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS14x6x						HSS14x4x			
		5⁄16[a, c]		¼[a, c]		3⁄16[a, b, c]		⅝		½[a]	
t_{des}, in.		0.291		0.233		0.174		0.581		0.465	
lb/ft		40.4		32.6		24.7		67.8		55.7	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	291	438	209	315	136	204	560	841	458	688
	1	291	438	209	314	136	204	558	838	456	686
	2	290	436	208	313	135	204	551	828	451	678
	3	288	433	207	311	135	202	539	811	442	665
	4	286	430	205	309	134	201	524	787	430	647
	5	283	425	203	305	132	199	505	758	415	624
	6	279	419	201	301	130	196	482	724	398	598
	7	274	412	197	297	128	193	457	686	378	568
	8	269	405	194	292	126	190	429	645	357	536
	9	264	396	190	286	124	186	400	601	334	501
	10	258	387	186	279	121	182	369	555	310	465
	11	251	377	181	272	118	177	338	508	285	429
	12	244	366	176	265	115	173	307	462	261	392
	13	236	355	171	257	112	168	277	416	236	355
	14	228	343	166	249	108	162	248	372	213	320
	15	220	331	160	240	104	157	219	330	190	285
	16	212	319	154	232	101	151	193	290	168	252
	17	203	306	148	223	96.9	146	171	257	149	223
	18	194	292	142	213	93.0	140	152	229	133	199
	19	185	278	136	204	89.1	134	137	205	119	179
	20	174	261	130	195	85.1	128	123	185	107	161
	22	152	228	117	176	77.2	116	102	153	88.7	133
	24	131	197	105	157	69.3	104	85.7	129	74.6	112
	26	111	168	92.1	138	61.6	92.6	73.0	110	63.5	95.5
	28	96.1	144	79.4	119	55.0	82.7				
	30	83.7	126	69.2	104	49.5	74.4				
	32	73.6	111	60.8	91.4	44.8	67.4				
	34	65.2	98.0	53.9	80.9	40.8	61.4				
	36	58.1	87.4	48.0	72.2	37.1	55.7				
	38	52.2	78.4	43.1	64.8	33.3	50.0				
	40	47.1	70.8	38.9	58.5	30.0	45.2				
	42	42.7	64.2	35.3	53.0	27.2	41.0				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		332	500	268	403	202	304	560	842	458	689
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		258	387	208	312	157	236	435	652	356	534
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		137	206	111	167	61.4	92.2	257	386	211	316
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		53.6	80.6	44.4	66.7	34.3	51.5	47.2	70.9	43.4	65.3
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		121	182	98.8	149	65.4	98.3	182	274	152	229
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		51.8	77.9	37.4	56.2	24.1	36.3	71.1	107	60.0	90.1
Properties											
Area, in.^{2}		11.1		8.96		6.76		18.7		15.3	
r_y, in.		2.55		2.58		2.61		1.59		1.64	
r_x/r_y		1.94		1.93		1.92		2.81		2.77	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS14–HSS12

A500 Gr. C

F_y = 50 ksi

F_u = 62 ksi

Shape		HSS14x4x								HSS12x10x	
		$3/8^{a,\,c}$		$5/16^{a,\,c}$		$1/4^{a,\,c}$		$3/16^{a,\,b,\,c}$		1/2	
t_{des}, in.		0.349		0.291		0.233		0.174		0.465	
lb/ft		42.8		36.1		29.2		22.2		69.3	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	337	506	256	385	182	273	115	173	569	855
	1	336	505	255	384	181	272	115	173	568	854
	2	333	500	254	381	180	270	114	171	567	853
	3	328	494	250	376	178	267	113	169	565	850
	4	322	484	246	369	174	262	111	166	563	846
	5	314	472	240	361	171	256	108	163	559	841
	6	305	458	233	351	166	249	105	159	555	835
	7	294	442	225	339	161	241	102	154	550	827
	8	279	419	217	326	155	232	98.6	148	545	819
	9	262	394	207	311	148	223	94.6	142	539	810
	10	244	367	197	296	141	212	90.4	136	532	799
	11	226	340	186	280	134	201	85.9	129	524	788
	12	208	312	175	263	126	190	81.3	122	516	776
	13	189	285	163	245	119	178	76.5	115	508	763
	14	172	258	148	222	111	166	71.7	108	499	750
	15	154	232	133	200	103	154	66.9	101	489	735
	16	137	207	119	179	94.8	143	62.0	93.2	479	720
	17	122	183	106	159	86.8	131	57.2	86.0	469	704
	18	109	163	94.5	142	78.3	118	52.6	79.1	458	688
	19	97.4	146	84.9	128	70.3	106	48.7	73.1	446	671
	20	87.9	132	76.6	115	63.4	95.4	45.1	67.8	435	654
	22	72.7	109	63.3	95.1	52.4	78.8	39.2	58.9	411	618
	24	61.1	91.8	53.2	79.9	44.1	66.2	34.4	51.7	386	581
	26	52.0	78.2	45.3	68.1	37.5	56.4	29.3	44.1	361	543
	28	44.9	67.4	39.1	58.7	32.4	48.6	25.3	38.0	336	505
	30									311	467
	32									286	430
	34									262	393
	36									238	358
	38									215	324
	40									194	292
	42									176	265
	44									161	241
	46									147	221
	48									135	203
	50									124	187
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		353	531	297	446	240	361	181	273	569	855
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		274	412	231	346	187	280	141	211	442	663
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		163	245	137	206	111	167	61.4	92.2	177	266
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		37.0	55.6	32.7	49.2	27.6	41.5	21.8	32.7	144	216
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		119	179	101	152	82.8	125	59.6	89.6	197	296
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		39.8	59.8	31.1	46.7	22.5	33.7	14.4	21.6	174	261
Properties											
Area, in.^{2}		11.8		9.92		8.03		6.06		19.0	
r_y, in.		1.69		1.72		1.74		1.77		3.96	
r_x/r_y		2.74		2.72		2.71		2.68		1.15	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS12

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS12x10x						HSS12x8x			
		⅜[a]		5⁄16[a, b, c]		¼[a, b, c]		⅝		½	
t_{des}, in.		0.349		0.291		0.233		0.581		0.465	
lb/ft		53.0		44.6		36.0		76.3		62.5	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	437	657	351	527	247	372	629	945	515	774
	1	437	657	350	527	247	372	628	944	514	773
	2	436	655	350	526	247	371	626	941	513	771
	3	435	653	349	524	247	371	623	936	510	767
	4	433	650	348	522	246	370	618	929	507	761
	5	430	646	346	520	245	368	612	920	502	754
	6	427	642	344	517	244	366	605	910	496	746
	7	423	636	341	513	243	365	597	897	490	736
	8	419	630	339	509	241	362	588	883	482	725
	9	415	623	336	505	239	360	577	868	474	713
	10	409	615	332	499	237	357	566	850	465	699
	11	404	607	329	494	235	354	553	832	455	684
	12	398	598	325	488	233	350	540	812	445	668
	13	391	588	320	481	231	347	526	791	433	651
	14	384	578	316	474	228	343	511	769	422	634
	15	377	567	311	467	225	339	496	745	409	615
	16	370	556	306	459	222	334	480	721	396	596
	17	362	544	300	451	219	330	464	697	383	576
	18	354	531	295	443	216	325	447	672	370	556
	19	345	519	289	434	213	320	430	646	356	535
	20	336	506	282	424	209	315	412	620	342	514
	22	318	479	267	402	202	303	377	567	314	472
	24	300	451	252	379	193	290	343	515	286	430
	26	281	422	236	355	183	276	308	463	258	388
	28	262	393	220	331	173	260	275	413	231	347
	30	242	364	204	307	163	245	243	366	205	309
	32	224	336	189	284	153	229	214	321	181	272
	34	205	308	173	260	142	214	189	285	160	241
	36	187	281	158	238	130	195	169	254	143	215
	38	170	255	144	216	118	178	152	228	128	193
	40	153	230	130	195	107	161	137	206	116	174
	42	139	209	118	177	97.0	146	124	186	105	158
	44	127	190	107	161	88.4	133	113	170	95.6	144
	46	116	174	98.2	148	80.9	122	103	155	87.4	131
	48	106	160	90.2	136	74.3	112	95.0	143	80.3	121
	50	98.0	147	83.1	125	68.5	103	87.6	132	74.0	111
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		437	657	365	549	296	446	629	945	515	774
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		339	509	284	425	230	345	488	732	400	600
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		138	207	116	174	94.6	142	215	323	177	266
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		112	169	95.5	143	77.9	117	131	196	110	166
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		152	229	116	175	84.4	127	205	308	170	255
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		123	185	94.8	142	69.9	105	154	232	128	193
Properties											
Area, in.^{2}		14.6		12.2		9.90		21.0		17.2	
r_y, in.		4.01		4.04		4.07		3.16		3.21	
r_x/r_y		1.15		1.15		1.15		1.37		1.37	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.
[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
Note: Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS12

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS12x8x								HSS12x6x	
		3/8[a]		5/16[a, c]		1/4[a, b, c]		3/16[a, b, c]		5/8	
t_{des}, in.		0.349		0.291		0.233		0.174		0.581	
lb/ft		47.9		40.4		32.6		24.7		67.8	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	395	594	318	477	233	350	143	215	560	841
	1	395	593	317	477	233	350	143	215	559	840
	2	394	592	317	476	232	349	143	215	556	835
	3	392	589	315	474	231	348	143	214	551	828
	4	389	585	314	471	230	346	142	213	544	817
	5	386	580	311	468	229	344	141	212	535	804
	6	381	573	309	464	227	341	140	211	524	787
	7	377	566	306	459	224	337	139	209	512	769
	8	371	558	302	454	222	333	138	208	498	748
	9	365	548	298	448	219	329	137	205	482	725
	10	358	538	293	441	216	324	135	203	466	700
	11	351	527	289	434	212	319	134	201	448	673
	12	343	515	283	426	209	314	132	198	429	645
	13	335	503	278	418	205	308	130	195	410	616
	14	326	490	272	409	201	301	128	192	390	586
	15	317	476	266	399	196	295	126	189	370	556
	16	307	462	259	389	192	288	123	185	349	525
	17	297	447	251	377	187	281	121	182	329	494
	18	287	432	242	364	182	273	118	178	308	463
	19	277	416	234	352	177	266	116	174	288	433
	20	267	401	225	338	172	258	113	170	268	403
	22	245	369	208	312	161	242	107	161	229	345
	24	224	337	190	285	150	225	101	151	194	291
	26	203	305	172	259	139	208	93.4	140	165	248
	28	183	274	155	233	127	191	86.1	129	142	214
	30	163	245	138	208	114	171	79.0	119	124	186
	32	144	216	122	184	101	151	71.9	108	109	164
	34	127	192	108	163	89.2	134	65.2	98.0	96.4	145
	36	114	171	96.8	145	79.5	120	59.4	89.3	86.0	129
	38	102	153	86.8	131	71.4	107	54.4	81.8	77.2	116
	40	92.1	138	78.4	118	64.4	96.8	49.5	74.4		
	42	83.5	126	71.1	107	58.4	87.8	44.9	67.5		
	44	76.1	114	64.8	97.4	53.2	80.0	40.9	61.5		
	46	69.6	105	59.3	89.1	48.7	73.2	37.4	56.2		
	48	63.9	96.1	54.4	81.8	44.7	67.2	34.4	51.7		
	50	58.9	88.6	50.2	75.4	41.2	62.0	31.7	47.6		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		395	594	332	500	268	403	202	304	560	842
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		307	460	258	387	208	312	157	236	435	652
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		138	207	116	174	94.6	142	64.4	96.8	215	323
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		87.1	131	74.5	112	61.1	91.8	46.8	70.3	88.9	134
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		132	199	112	168	81.7	123	53.6	80.6	172	258
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		91.7	138	70.8	106	52.0	78.2	34.2	51.3	105	158
Properties											
Area, in.^{2}		13.2		11.1		8.96		6.76		18.7	
r_y, in.		3.27		3.29		3.32		3.35		2.39	
r_x/r_y		1.37		1.37		1.36		1.36		1.73	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.
[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

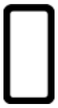
Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS12

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS12x6x									
		½		⅜^{a}		5/16$^{a, c}$		¼$^{a, c}$		3/16$^{a, b, c}$	
t_{des}, in.		0.465		0.349		0.291		0.233		0.174	
lb/ft		55.7		42.8		36.1		29.2		22.2	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	458	688	353	531	282	424	205	308	134	202
	1	457	687	353	530	282	424	205	308	134	201
	2	455	684	351	527	281	422	204	307	134	201
	3	451	678	348	523	279	419	203	305	133	199
	4	445	669	344	517	276	415	201	302	132	198
	5	438	659	339	509	273	411	199	299	130	196
	6	430	646	332	500	269	405	196	295	128	193
	7	420	631	325	489	265	398	193	290	126	190
	8	409	615	317	476	260	390	189	284	124	186
	9	397	597	308	463	254	382	185	278	121	183
	10	384	577	298	448	248	372	181	272	119	178
	11	370	556	288	432	241	362	176	265	116	174
	12	355	534	277	416	234	351	171	257	112	169
	13	340	511	265	399	224	337	166	249	109	164
	14	324	487	253	381	215	323	160	241	105	158
	15	308	462	241	362	205	307	154	232	102	153
	16	291	438	229	344	194	292	148	223	97.9	147
	17	275	413	216	325	184	276	142	214	94.0	141
	18	258	388	204	306	174	261	136	204	90.1	135
	19	242	364	191	288	163	245	130	195	86.1	129
	20	226	339	179	269	153	230	123	185	82.1	123
	22	195	293	155	233	133	200	109	164	74.0	111
	24	165	248	133	199	114	172	93.9	141	66.1	99.3
	26	141	211	113	170	97.3	146	80.0	120	58.4	87.7
	28	121	182	97.4	146	83.9	126	69.0	104	52.0	78.1
	30	106	159	84.9	128	73.1	110	60.1	90.3	46.4	69.8
	32	92.9	140	74.6	112	64.2	96.5	52.8	79.4	40.8	61.3
	34	82.2	124	66.1	99.3	56.9	85.5	46.8	70.3	36.1	54.3
	36	73.4	110	58.9	88.6	50.7	76.3	41.7	62.7	32.2	48.5
	38	65.8	99.0	52.9	79.5	45.5	68.4	37.4	56.3	28.9	43.5
	40	59.4	89.3	47.7	71.7	41.1	61.8	33.8	50.8	26.1	39.2
	42					37.3	56.0	30.7	46.1	23.7	35.6
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		458	689	353	531	297	446	240	361	181	273
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		356	534	274	412	231	346	187	280	141	211
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		177	266	138	207	116	174	94.6	142	64.4	96.8
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		77.0	116	62.1	93.3	53.6	80.6	44.4	66.7	34.3	51.5
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		143	215	112	168	95.1	143	77.6	117	51.9	78.1
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		87.8	132	63.4	95.3	49.2	74.0	35.9	54.0	23.4	35.1
Properties											
Area, in.^{2}		15.3		11.8		9.92		8.03		6.06	
r_y, in.		2.44		2.49		2.52		2.54		2.57	
r_x/r_y		1.73		1.72		1.71		1.71		1.70	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.
[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.



HSS12

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS12x4x									
		5⁄8		1⁄2		3⁄8[a]		5⁄16[a, c]		1⁄4[a, c]	
t_{des}, in.		0.581		0.465		0.349		0.291		0.233	
lb/ft		59.3		48.9		37.7		31.8		25.8	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	491	738	404	607	311	468	248	372	177	266
	1	489	735	403	605	310	466	247	371	177	266
	2	483	725	398	598	307	461	245	368	175	264
	3	472	710	390	586	301	452	241	363	173	260
	4	459	689	379	570	293	441	237	356	170	255
	5	441	663	366	550	283	426	231	347	166	250
	6	421	633	350	526	272	409	224	337	161	242
	7	398	599	332	499	259	389	216	325	156	234
	8	374	561	313	470	245	368	207	311	150	225
	9	347	522	292	439	229	345	195	293	143	215
	10	320	481	271	407	213	321	182	274	136	205
	11	293	440	249	374	197	296	169	254	129	193
	12	265	399	227	341	181	272	155	233	121	182
	13	239	359	205	308	165	247	142	213	113	170
	14	213	319	184	277	149	223	128	193	105	158
	15	188	282	164	246	133	200	116	174	95.4	143
	16	165	248	144	217	118	178	103	155	85.5	128
	17	146	219	128	192	105	157	91.4	137	75.9	114
	18	130	196	114	172	93.4	140	81.6	123	67.7	102
	19	117	176	102	154	83.9	126	73.2	110	60.7	91.3
	20	105	159	92.5	139	75.7	114	66.1	99.3	54.8	82.4
	22	87.2	131	76.4	115	62.6	94.0	54.6	82.1	45.3	68.1
	24	73.3	110	64.2	96.5	52.6	79.0	45.9	69.0	38.1	57.2
	26	62.4	93.8	54.7	82.2	44.8	67.3	39.1	58.8	32.4	48.7
	28							33.7	50.7	28.0	42.0
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		491	738	404	608	311	468	262	394	213	320
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		381	572	314	471	242	363	204	306	165	248
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		215	323	177	266	138	207	116	174	94.6	142
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		47.2	70.9	43.4	65.3	37.0	55.6	32.7	49.2	27.6	41.5
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		138	208	117	175	91.6	138	78.1	117	63.9	96.0
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		61.1	91.9	52.1	78.4	37.9	57.0	29.5	44.3	21.5	32.3
Properties											
Area, in.^{2}		16.4		13.5		10.4		8.76		7.10	
r_y, in.		1.57		1.62		1.67		1.70		1.72	
r_x/r_y		2.46		2.44		2.41		2.39		2.38	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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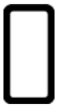


HSS12

Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS12x4x		HSS12x3½x				HSS12x3x			
		3/16[a, b, c]		3/8[a]		5/16[a, c]		5/16[a, c]		1/4[a, c]	
t_{des}, in.		0.174		0.349		0.291		0.291		0.233	
lb/ft		19.6		36.4		30.8		29.7		24.1	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	114	171	299	450	239	359	230	346	163	245
	1	113	170	298	448	238	357	229	344	162	244
	2	112	169	294	441	235	353	225	339	160	241
	3	111	167	286	430	231	347	220	331	157	236
	4	109	164	277	416	225	338	213	320	152	228
	5	107	160	265	398	218	328	204	306	146	219
	6	104	156	251	377	209	315	193	290	139	208
	7	100	151	235	353	200	300	178	267	131	196
	8	96.6	145	218	328	186	280	161	242	122	183
	9	92.6	139	201	302	172	258	144	217	113	169
	10	88.3	133	183	275	157	235	127	191	103	155
	11	83.7	126	165	248	142	213	111	167	92.3	139
	12	79.0	119	147	221	127	191	95.5	144	79.8	120
	13	74.2	111	130	195	112	169	81.4	122	68.1	102
	14	69.3	104	114	171	98.7	148	70.2	105	58.8	88.3
	15	64.4	96.7	98.9	149	86.0	129	61.1	91.9	51.2	76.9
	16	59.5	89.4	86.9	131	75.6	114	53.7	80.8	45.0	67.6
	17	54.6	82.0	77.0	116	66.9	101	47.6	71.5	39.8	59.9
	18	50.1	75.3	68.7	103	59.7	89.7	42.5	63.8	35.5	53.4
	19	46.1	69.3	61.6	92.6	53.6	80.5	38.1	57.3	31.9	47.9
	20	42.7	64.1	55.6	83.6	48.4	72.7	34.4	51.7	28.8	43.3
	22	35.5	53.3	46.0	69.1	40.0	60.1				
	24	29.8	44.8	38.6	58.1	33.6	50.5				
	26	25.4	38.2								
	28	21.9	32.9								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		161	242	299	450	253	381	245	368	199	298
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		125	187	233	349	197	295	190	285	154	231
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		64.4	96.8	138	207	116	174	116	174	94.6	142
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		21.8	32.7	30.7	46.2	27.5	41.3	22.3	33.5	19.3	28.9
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		47.8	71.9	86.6	130	73.9	111	69.6	105	57.1	85.9
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		13.9	20.9	31.9	48.0	24.8	37.3	20.4	30.6	15.0	22.6
Properties											
Area, in.^{2}		5.37		10.0		8.46		8.17		6.63	
r_y, in.		1.75		1.46		1.48		1.27		1.29	
r_x/r_y		2.36		2.70		2.69		3.07		3.05	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

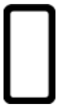
Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS12–HSS10

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS12x3x		HSS12x2x						HSS10x8x	
		$^3\!/_{16}$ [a, b, c]		$^5\!/_{16}$ [a, c]		$^1\!/_4$ [a, c]		$^3\!/_{16}$ [a, b, c]		$^5\!/_8$	
t_{des}, in.		0.174		0.291		0.233		0.174		0.581	
lb/ft		18.4		27.6		22.4		17.1		67.8	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	103	155	213	319	149	225	92.6	139	560	841
	1	103	154	210	316	148	222	91.7	138	559	841
	2	101	152	203	305	143	216	89.2	134	557	838
	3	99.2	149	192	289	136	205	85.1	128	554	833
	4	96.2	145	177	266	127	191	79.7	120	550	827
	5	92.6	139	154	231	116	174	73.3	110	545	819
	6	88.4	133	129	194	103	155	66.1	99.3	538	809
	7	83.6	126	106	159	89.6	135	58.4	87.8	530	797
	8	78.4	118	83.2	125	71.9	108	50.6	76.0	522	784
	9	72.9	110	65.8	98.8	56.8	85.3	43.0	64.6	512	770
	10	67.2	101	53.3	80.1	46.0	69.1	36.9	55.4	501	754
	11	61.4	92.2	44.0	66.2	38.0	57.1	30.6	46.0	490	736
	12	55.5	83.5	37.0	55.6	31.9	48.0	25.7	38.7	478	718
	13	49.7	74.7	31.5	47.4	27.2	40.9	21.9	33.0	465	698
	14	44.6	67.0			23.5	35.3	18.9	28.4	451	678
	15	40.2	60.4							437	657
	16	35.7	53.6							422	635
	17	31.6	47.5							407	612
	18	28.2	42.4							392	589
	19	25.3	38.0							376	565
	20	22.8	34.3							360	541
	22	18.9	28.4							328	493
	24									297	446
	26									266	399
	28									236	354
	30									207	311
	32									182	274
	34									161	242
	36									144	216
	38									129	194
	40									116	175
	42									106	159
	44									96.3	145
	46									88.1	132
	48									80.9	122
	50									74.5	112
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		150	226	227	342	185	278	140	210	560	842
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		117	175	176	265	143	215	109	163	435	652
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		64.4	96.8	116	174	94.6	142	64.4	96.8	172	259
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		15.5	23.3	11.8	17.8	10.9	16.4	9.25	13.9	131	196
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		42.6	64.1	61.1	91.9	50.1	75.4	37.6	56.5	155	233
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		9.65	14.5	11.9	17.9	8.87	13.3	5.78	8.68	133	200
Properties											
Area, in.^{2}		5.02		7.59		6.17		4.67		18.7	
r_y, in.		1.32		0.820		0.845		0.872		3.09	
r_x/r_y		3.02		4.52		4.44		4.36		1.19	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.
[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.



HSS10

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS10x8x									
		½		⅜		5⁄16[a]		¼[a, b, c]		3⁄16[a, b, c]	
t_{des}, in.		0.465		0.349		0.291		0.233		0.174	
lb/ft		55.7		42.8		36.1		29.2		22.2	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	458	688	353	531	297	446	227	341	141	212
	1	458	688	353	530	297	446	227	341	141	212
	2	456	686	352	529	296	445	226	340	141	211
	3	454	682	350	526	294	442	225	338	140	211
	4	450	677	347	522	292	439	224	336	139	210
	5	446	670	344	517	290	435	222	334	139	208
	6	441	663	340	512	286	430	220	331	138	207
	7	435	653	336	505	283	425	218	327	137	205
	8	428	643	331	497	278	418	215	323	135	203
	9	420	631	325	488	274	411	212	319	134	201
	10	412	619	319	479	268	403	209	314	132	199
	11	403	605	312	469	263	395	205	309	131	196
	12	393	590	304	457	257	386	202	303	129	194
	13	382	575	297	446	250	376	197	297	127	191
	14	372	558	288	434	243	366	193	290	125	187
	15	360	541	280	421	236	355	188	283	122	184
	16	349	524	271	407	229	344	184	276	120	180
	17	336	506	262	394	221	333	179	269	117	177
	18	324	487	253	380	214	321	174	261	115	173
	19	312	468	243	365	206	309	168	252	112	169
	20	299	449	234	351	198	297	161	243	109	164
	22	273	411	214	322	182	273	148	223	103	156
	24	248	372	195	293	165	249	135	204	96.3	145
	26	223	334	176	264	149	225	123	184	88.9	134
	28	198	298	157	236	134	201	110	165	81.5	122
	30	175	263	139	209	119	179	98.0	147	74.2	112
	32	154	231	122	184	105	158	86.5	130	66.5	99.9
	34	136	205	108	163	92.9	140	76.6	115	58.9	88.5
	36	121	183	96.7	145	82.8	125	68.3	103	52.5	78.9
	38	109	164	86.8	130	74.3	112	61.3	92.1	47.1	70.8
	40	98.4	148	78.3	118	67.1	101	55.3	83.2	42.5	63.9
	42	89.3	134	71.1	107	60.9	91.5	50.2	75.4	38.6	58.0
	44	81.3	122	64.7	97.3	55.5	83.3	45.7	68.7	35.2	52.8
	46	74.4	112	59.2	89.0	50.7	76.3	41.8	62.9	32.2	48.3
	48	68.3	103	54.4	81.8	46.6	70.0	38.4	57.8	29.5	44.4
	50	63.0	94.7	50.1	75.4	42.9	64.5	35.4	53.2	27.2	40.9
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		458	689	353	531	297	446	240	361	181	273
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		356	534	274	412	231	346	187	280	141	211
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		144	216	112	169	95.5	143	77.9	117	59.3	89.1
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		110	166	87.1	131	74.5	112	61.1	91.8	46.8	70.3
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		129	195	101	152	85.8	129	63.2	95.1	41.7	62.7
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		111	167	86.8	131	67.5	101	49.2	73.9	32.8	49.3
Properties											
Area, in.^{2}		15.3		11.8		9.92		8.03		6.06	
r_y, in.		3.14		3.19		3.22		3.25		3.28	
r_x/r_y		1.19		1.19		1.19		1.18		1.18	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.
[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
Note: Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS10

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS10x6x									
		⅝		½		⅜		5⁄16[a]		¼[a, c]	
t_{des}, in.		0.581		0.465		0.349		0.291		0.233	
lb/ft		59.3		48.9		37.7		31.8		25.8	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	491	738	404	607	311	468	262	394	199	299
	1	490	737	403	606	311	467	262	394	199	299
	2	487	732	401	603	309	465	260	391	198	297
	3	483	725	398	598	306	461	258	388	197	295
	4	476	716	392	590	303	455	255	383	195	293
	5	468	703	386	580	298	448	251	378	192	289
	6	458	689	378	568	292	439	246	370	189	285
	7	447	672	369	555	286	429	241	362	186	280
	8	434	653	359	540	278	418	235	353	182	274
	9	420	632	348	523	270	406	228	343	178	268
	10	405	609	336	505	261	392	221	332	174	261
	11	389	585	323	486	251	378	213	320	169	254
	12	372	560	310	466	241	363	205	307	164	246
	13	355	533	296	445	231	347	196	294	158	238
	14	337	506	282	423	220	331	187	281	152	229
	15	319	479	267	401	209	314	178	267	145	218
	16	300	451	252	379	198	298	169	253	138	207
	17	282	423	237	357	187	281	159	239	130	196
	18	263	396	222	334	176	264	150	225	123	184
	19	245	369	208	312	164	247	141	211	115	173
	20	228	342	193	291	153	231	132	198	108	162
	22	194	291	166	249	132	199	114	171	93.4	140
	24	163	245	140	210	112	169	96.8	146	79.8	120
	26	139	208	119	179	95.6	144	82.5	124	68.0	102
	28	120	180	103	154	82.4	124	71.2	107	58.6	88.1
	30	104	157	89.4	134	71.8	108	62.0	93.2	51.1	76.7
	32	91.5	138	78.6	118	63.1	94.9	54.5	81.9	44.9	67.4
	34	81.1	122	69.6	105	55.9	84.0	48.3	72.5	39.7	59.7
	36	72.3	109	62.1	93.3	49.9	75.0	43.0	64.7	35.5	53.3
	38	64.9	97.6	55.7	83.8	44.8	67.3	38.6	58.1	31.8	47.8
	40					40.4	60.7	34.9	52.4	28.7	43.2
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		491	738	404	608	311	468	262	394	213	320
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		381	572	314	471	242	363	204	306	165	248
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		172	259	144	216	112	169	95.5	143	77.9	117
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		88.9	134	77.0	116	62.1	93.3	53.6	80.6	44.4	66.7
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		128	192	107	161	84.3	127	71.9	108	58.9	88.5
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		89.3	134	75.1	113	59.1	88.9	46.5	69.9	33.9	51.0
Properties											
Area, in.^{2}		16.4		13.5		10.4		8.76		7.10	
r_y, in.		2.34		2.39		2.44		2.47		2.49	
r_x/r_y		1.50		1.49		1.49		1.48		1.48	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS10

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS10x6x		HSS10x5x							
		3/16$^{a, b, c}$		3/8		5/16^{a}		1/4$^{a, c}$		3/16$^{a, c}$	
t_{des}, in.		0.174		0.349		0.291		0.233		0.174	
lb/ft		19.6		35.1		29.7		24.1		18.4	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	132	198	290	435	245	368	185	278	122	183
	1	132	198	289	434	244	367	185	277	121	182
	2	131	197	287	431	242	364	184	276	121	181
	3	130	196	283	425	239	360	182	273	120	180
	4	129	194	278	418	235	353	179	270	118	178
	5	128	192	272	409	230	346	176	265	116	175
	6	126	189	265	398	224	337	173	260	114	171
	7	124	186	256	385	217	326	169	254	111	167
	8	121	182	247	371	209	314	164	246	108	163
	9	119	178	236	355	200	301	159	239	105	158
	10	116	174	225	339	191	288	153	230	102	153
	11	113	169	214	321	182	273	147	221	97.9	147
	12	109	164	202	303	172	258	141	212	93.9	141
	13	106	159	190	285	161	243	133	199	89.8	135
	14	102	154	177	266	151	227	124	187	85.5	129
	15	98.3	148	165	248	141	212	116	174	81.2	122
	16	94.4	142	152	229	130	196	108	162	76.7	115
	17	90.4	136	140	211	120	181	99.6	150	72.3	109
	18	86.4	130	129	193	110	166	91.6	138	67.8	102
	19	82.2	124	117	176	101	151	83.8	126	63.4	95.3
	20	78.1	117	106	159	91.4	137	76.3	115	59.0	88.7
	22	69.9	105	87.6	132	75.5	113	63.1	94.8	49.1	73.8
	24	61.7	92.8	73.6	111	63.4	95.3	53.0	79.6	41.3	62.0
	26	52.7	79.1	62.7	94.3	54.1	81.2	45.1	67.9	35.2	52.9
	28	45.4	68.2	54.1	81.3	46.6	70.1	38.9	58.5	30.3	45.6
	30	39.6	59.4	47.1	70.8	40.6	61.0	33.9	51.0	26.4	39.7
	32	34.8	52.2	41.4	62.3	35.7	53.6	29.8	44.8	23.2	34.9
	34	30.8	46.3	36.7	55.2	31.6	47.5	26.4	39.7	20.6	30.9
	36	27.5	41.3								
	38	24.7	37.0								
	40	22.2	33.4								
	42	20.2	30.3								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		161	242	290	435	245	368	199	298	150	226
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		125	187	225	337	190	285	154	231	117	175
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		59.3	89.1	112	169	95.5	143	77.9	117	59.3	89.1
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		34.3	51.5	49.5	74.4	43.2	64.9	36.0	54.1	28.0	42.1
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		39.7	59.7	75.8	114	64.9	97.5	53.1	79.9	40.7	61.1
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		22.4	33.7	46.7	70.1	36.8	55.3	26.9	40.5	17.7	26.6
Properties											
Area, in.^{2}		5.37		9.67		8.17		6.63		5.02	
r_y, in.		2.52		2.05		2.07		2.10		2.13	
r_x/r_y		1.48		1.72		1.72		1.71		1.70	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.
[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS10

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS10x4x									
		5⁄8		1⁄2		3⁄8		5⁄16[a]		1⁄4[a, c]	
t_{des}, in.		0.581		0.465		0.349		0.291		0.233	
lb/ft		50.8		42.1		32.6		27.6		22.4	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	419	630	347	522	269	404	227	342	171	257
	1	417	627	346	520	268	402	226	340	171	256
	2	412	619	342	513	264	397	224	336	169	254
	3	403	605	335	503	259	390	220	330	167	251
	4	390	587	325	488	252	379	214	322	164	246
	5	375	564	313	470	244	366	207	311	160	240
	6	357	537	299	449	233	351	198	298	155	233
	7	337	507	283	426	222	333	189	284	149	224
	8	315	474	266	400	209	314	178	268	143	215
	9	293	440	248	373	196	294	167	252	136	205
	10	269	404	229	344	182	273	156	234	128	193
	11	245	368	210	315	167	251	144	216	119	179
	12	221	332	191	287	153	230	132	198	109	164
	13	198	298	172	258	139	208	120	180	99.8	150
	14	176	264	154	231	125	187	108	163	90.5	136
	15	154	232	136	205	111	167	97.2	146	81.4	122
	16	135	203	120	180	98.4	148	86.3	130	72.7	109
	17	120	180	106	159	87.1	131	76.5	115	64.4	96.8
	18	107	161	94.5	142	77.7	117	68.2	102	57.4	86.3
	19	96.0	144	84.8	127	69.8	105	61.2	92.0	51.6	77.5
	20	86.6	130	76.5	115	63.0	94.6	55.2	83.0	46.5	69.9
	22	71.6	108	63.2	95.1	52.0	78.2	45.7	68.6	38.5	57.8
	24	60.2	90.4	53.1	79.9	43.7	65.7	38.4	57.7	32.3	48.6
	26			45.3	68.1	37.3	56.0	32.7	49.1	27.5	41.4
	28									23.7	35.7
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		419	630	347	522	269	404	227	342	185	278
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		326	488	270	405	209	313	176	265	143	215
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		172	259	144	216	112	169	95.5	143	77.9	117
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		47.2	70.9	43.4	65.3	37.0	55.6	32.7	49.2	27.6	41.5
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		101	151	85.1	128	67.4	101	57.6	86.6	47.4	71.3
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		51.4	77.3	43.9	66.0	34.9	52.5	27.7	41.7	20.3	30.5
Properties											
Area, in.^{2}		14.0		11.6		8.97		7.59		6.17	
r_y, in.		1.54		1.59		1.64		1.67		1.70	
r_x/r_y		2.12		2.10		2.08		2.06		2.05	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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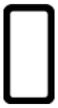
Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS10

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS10x4x				HSS10x3½x					
		3⁄16[a, c]		1⁄8[a, b, c]		½		⅜		5⁄16[a]	
t_{des}, in.		0.174		0.116		0.465		0.349		0.291	
lb/ft		17.1		11.6		40.3		31.3		26.5	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	111	167	60.9	91.6	332	499	258	388	219	328
	1	111	166	60.8	91.3	331	497	257	386	217	327
	2	110	165	60.3	90.6	325	489	253	380	214	322
	3	108	163	59.5	89.4	316	476	247	371	209	314
	4	106	160	58.4	87.8	305	458	238	358	202	304
	5	104	156	57.1	85.8	290	436	227	342	193	290
	6	101	152	55.5	83.3	273	411	215	323	183	275
	7	97.4	146	53.6	80.6	254	382	201	302	172	258
	8	93.6	141	51.6	77.5	234	352	186	280	159	239
	9	89.4	134	49.3	74.1	214	321	171	257	146	220
	10	85.0	128	47.0	70.6	193	290	155	233	133	200
	11	80.3	121	44.5	66.9	172	258	140	210	120	181
	12	75.5	113	41.9	63.0	152	228	124	187	107	161
	13	70.6	106	39.3	59.1	132	199	109	164	94.9	143
	14	65.6	98.6	36.7	55.1	114	172	95.2	143	82.9	125
	15	60.6	91.0	34.0	51.1	99.5	150	82.9	125	72.2	108
	16	55.6	83.6	31.4	47.2	87.4	131	72.9	110	63.4	95.4
	17	49.9	75.0	28.8	43.3	77.5	116	64.6	97.0	56.2	84.5
	18	44.5	66.9	26.4	39.7	69.1	104	57.6	86.5	50.1	75.3
	19	39.9	60.0	24.4	36.6	62.0	93.2	51.7	77.7	45.0	67.6
	20	36.1	54.2	22.6	33.9	56.0	84.1	46.6	70.1	40.6	61.0
	22	29.8	44.8	19.5	29.3	46.3	69.5	38.5	57.9	33.6	50.4
	24	25.0	37.6	17.1	25.7			32.4	48.7	28.2	42.4
	26	21.3	32.1	14.9	22.5						
	28	18.4	27.6	12.9	19.4						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		140	210	94.6	142	332	500	258	388	219	329
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		109	163	73.5	110	258	387	200	301	170	255
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		59.3	89.1	25.4	38.2	144	216	112	169	95.5	143
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		21.8	32.7	15.2	22.9	35.1	52.7	30.7	46.2	27.5	41.3
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		36.4	54.8	21.6	32.4	79.6	120	63.1	94.9	54.1	81.4
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		13.4	20.1	7.26	10.9	36.7	55.1	29.4	44.3	23.4	35.1
Properties											
Area, in.^{2}		4.67		3.16		11.1		8.62		7.30	
r_y, in.		1.72		1.75		1.39		1.44		1.46	
r_x/r_y		2.05		2.03		2.35		2.32		2.32	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.
[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.



HSS10

Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS10x3½x						HSS10x3x			
		¼[a, c]		3⁄16[a, c]		⅛[a, b, c]		⅜		5⁄16[a]	
t_{des}, in.		0.233		0.174		0.116		0.349		0.291	
lb/ft		21.6		16.4		11.1		30.0		25.5	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	164	246	106	159	57.3	86.2	248	372	210	315
	1	163	246	106	159	57.1	85.9	246	370	208	313
	2	162	243	105	157	56.6	85.0	241	362	204	307
	3	159	239	103	154	55.6	83.6	232	349	198	297
	4	155	233	100	151	54.4	81.7	221	332	188	283
	5	150	225	97.3	146	52.8	79.3	207	312	177	267
	6	144	216	93.7	141	50.9	76.5	192	288	165	248
	7	137	207	89.6	135	48.8	73.3	175	263	151	227
	8	130	196	85.2	128	46.4	69.8	157	237	136	205
	9	121	182	80.3	121	43.9	66.0	140	210	122	183
	10	110	166	75.3	113	41.2	62.0	122	183	107	161
	11	100	150	70.0	105	38.5	57.9	105	158	92.9	140
	12	89.7	135	64.6	97.1	35.7	53.7	89.2	134	79.4	119
	13	79.7	120	59.2	89.0	32.9	49.5	76.0	114	67.7	102
	14	70.1	105	53.8	80.9	30.1	45.3	65.6	98.5	58.3	87.7
	15	61.1	91.8	47.6	71.5	27.4	41.1	57.1	85.8	50.8	76.4
	16	53.7	80.7	41.8	62.9	24.9	37.4	50.2	75.4	44.7	67.1
	17	47.5	71.5	37.1	55.7	22.7	34.2	44.5	66.8	39.6	59.5
	18	42.4	63.7	33.1	49.7	20.9	31.4	39.7	59.6	35.3	53.0
	19	38.1	57.2	29.7	44.6	19.3	28.9	35.6	53.5	31.7	47.6
	20	34.4	51.6	26.8	40.2	17.8	26.8	32.1	48.3	28.6	43.0
	22	28.4	42.7	22.1	33.3	15.4	23.2				
	24	23.9	35.9	18.6	27.9	13.1	19.6				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		178	267	135	203	91.0	137	248	372	210	315
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		138	207	105	157	70.7	106	192	288	163	244
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		77.9	117	59.3	89.1	25.4	38.2	112	169	95.5	143
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		23.4	35.2	18.6	28.0	13.1	19.7	24.5	36.7	22.3	33.5
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		44.7	67.1	34.2	51.4	21.8	32.8	59.1	88.9	50.6	76.1
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		17.1	25.7	11.3	16.9	6.11	9.18	24.3	36.5	19.2	28.8
Properties											
Area, in.^{2}		5.93		4.50		3.04		8.27		7.01	
r_y, in.		1.49		1.51		1.54		1.22		1.25	
r_x/r_y		2.29		2.28		2.27		2.67		2.64	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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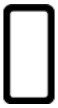

HSS10

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS10x3x						HSS10x2x			
		¼$^{a, c}$		3⁄16$^{a, c}$		⅛$^{a, b, c}$		⅜		5⁄16^{a}	
t_{des}, in.		0.233		0.174		0.116		0.349		0.291	
lb/ft		20.7		15.8		10.7		27.5		23.3	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	157	236	101	151	54.0	81.2	227	341	193	289
	1	156	235	100	150	53.8	80.9	223	335	189	285
	2	154	232	98.8	148	53.1	79.8	212	319	181	271
	3	150	226	96.6	145	52.0	78.1	195	293	167	251
	4	145	219	93.6	141	50.4	75.8	173	260	149	224
	5	139	209	89.8	135	48.5	72.9	148	223	129	194
	6	132	198	85.5	128	46.2	69.5	123	185	108	163
	7	124	186	80.6	121	43.7	65.7	98.7	148	88.0	132
	8	113	170	75.3	113	41.0	61.6	76.6	115	69.1	104
	9	101	152	69.6	105	38.1	57.2	60.5	90.9	54.6	82.1
	10	89.8	135	63.8	95.9	35.1	52.7	49.0	73.7	44.3	66.5
	11	78.4	118	57.9	87.0	32.1	48.2	40.5	60.9	36.6	55.0
	12	67.6	102	51.9	78.1	29.0	43.6	34.0	51.1	30.7	46.2
	13	57.7	86.7	45.1	67.8	26.0	39.1	29.0	43.6	26.2	39.4
	14	49.7	74.8	38.9	58.4	23.4	35.1				
	15	43.3	65.1	33.9	50.9	21.1	31.7				
	16	38.1	57.2	29.8	44.7	19.2	28.9				
	17	33.7	50.7	26.4	39.6	17.6	26.4				
	18	30.1	45.2	23.5	35.4	16.1	24.2				
	19	27.0	40.6	21.1	31.7	14.8	22.3				
	20	24.4	36.6	19.1	28.6	13.5	20.3				
	22					11.2	16.8				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		171	257	129	194	87.7	132	227	341	193	289
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		133	199	100	151	68.1	102	176	264	149	224
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		77.9	117	59.3	89.1	25.4	38.2	112	169	95.5	143
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		19.3	28.9	15.5	23.3	11.0	16.6	11.9	18.0	11.8	17.8
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		41.7	62.6	31.9	48.0	20.4	30.7	50.6	76.1	43.7	65.6
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		14.1	21.2	9.27	13.9	5.01	7.52	14.4	21.6	11.3	17.0
Properties											
Area, in.^{2}		5.70		4.32		2.93		7.58		6.43	
r_y, in.		1.28		1.30		1.33		0.787		0.812	
r_x/r_y		2.61		2.60		2.57		3.91		3.84	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.
[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS10–HSS9

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS10x2x						HSS9x7x			
		¼$^{a, c}$		3⁄16$^{a, c}$		⅛$^{a, b, c}$		⅝		½	
t_{des}, in.		0.233		0.174		0.116		0.581		0.465	
lb/ft		19.0		14.5		9.86		59.3		48.9	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	143	215	90.4	136	47.1	70.9	491	738	404	607
	1	142	213	89.5	135	46.7	70.2	490	737	404	607
	2	137	206	86.9	131	45.4	68.3	488	734	402	604
	3	130	195	82.8	124	43.4	65.2	485	728	399	600
	4	120	181	77.3	116	40.7	61.2	480	721	395	594
	5	108	162	70.7	106	37.5	56.4	473	711	390	586
	6	91.4	137	63.3	95.2	33.9	51.0	466	700	384	577
	7	75.3	113	55.5	83.5	30.1	45.2	457	687	377	567
	8	60.0	90.2	47.6	71.5	26.2	39.4	447	672	369	555
	9	47.4	71.3	38.3	57.5	22.4	33.7	436	655	360	542
	10	38.4	57.7	31.0	46.6	19.3	29.1	424	637	351	527
	11	31.7	47.7	25.6	38.5	16.9	25.4	411	618	341	512
	12	26.7	40.1	21.5	32.4	14.9	22.4	398	598	330	496
	13	22.7	34.2	18.4	27.6	13.2	19.9	383	576	318	478
	14	19.6	29.5	15.8	23.8	11.4	17.1	368	554	306	461
	15							353	531	294	442
	16							337	507	282	423
	17							321	483	269	404
	18							305	459	256	384
	19							289	435	243	365
	20							273	411	230	345
	22							242	363	204	307
	24							211	317	179	269
	26							182	273	155	234
	28							157	236	134	201
	30							137	205	117	175
	32							120	180	103	154
	34							106	160	90.8	137
	36							94.9	143	81.0	122
	38							85.1	128	72.7	109
	40							76.8	115	65.6	98.7
	42							69.7	105	59.5	89.5
	44							63.5	95.5	54.2	81.5
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		157	236	119	179	80.8	122	491	738	404	608
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		122	183	92.5	139	62.8	94.2	381	572	314	471
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		77.9	117	59.3	89.1	25.4	38.2	152	228	127	191
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		10.9	16.4	9.25	13.9	6.88	10.3	110	165	93.6	141
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		35.9	54.0	27.7	41.6	17.6	26.4	121	181	101	152
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		8.34	12.5	5.56	8.35	3.00	4.51	101	152	84.8	128
Properties											
Area, in.^{2}		5.24		3.98		2.70		16.4		13.5	
r_y, in.		0.838		0.864		0.890		2.68		2.73	
r_x/r_y		3.78		3.72		3.65		1.22		1.22	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS9

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS9x7x								HSS9x5x	
		⅜		5⁄16[a]		¼[a, b, c]		3⁄16[a, b, c]		⅝	
t_{des}, in.		0.349		0.291		0.233		0.174		0.581	
lb/ft		37.7		31.8		25.8		19.6		50.8	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	311	468	262	394	209	314	137	205	419	630
	1	311	467	262	394	208	313	137	205	418	628
	2	310	465	261	392	208	312	136	205	414	623
	3	308	462	259	389	207	311	136	204	409	614
	4	305	458	257	386	205	308	135	203	400	602
	5	301	452	254	381	203	305	134	201	390	587
	6	296	446	250	376	201	302	133	199	378	568
	7	291	438	246	369	198	297	131	197	364	548
	8	285	429	241	362	195	293	129	195	349	525
	9	279	419	235	354	191	287	128	192	333	500
	10	272	408	230	345	187	280	126	189	315	473
	11	264	397	223	335	182	273	123	185	297	446
	12	256	385	216	325	176	265	121	182	278	418
	13	247	372	209	315	170	256	118	177	259	389
	14	238	358	202	304	165	247	115	173	239	360
	15	229	344	194	292	158	238	111	167	220	331
	16	220	330	186	280	152	229	108	162	202	303
	17	210	316	178	268	146	219	104	157	184	276
	18	200	301	170	256	139	209	100	151	166	250
	19	190	286	162	244	133	199	96.6	145	149	224
	20	181	271	154	231	126	190	92.7	139	135	202
	22	161	242	138	207	113	170	84.9	128	111	167
	24	142	214	122	183	100	151	77.0	116	93.5	141
	26	124	186	106	160	88.0	132	67.8	102	79.7	120
	28	107	161	92.1	138	76.2	115	58.9	88.5	68.7	103
	30	93.2	140	80.2	121	66.4	99.8	51.3	77.1	59.9	90.0
	32	81.9	123	70.5	106	58.4	87.7	45.1	67.8	52.6	79.1
	34	72.6	109	62.5	93.9	51.7	77.7	39.9	60.0		
	36	64.7	97.3	55.7	83.7	46.1	69.3	35.6	53.5		
	38	58.1	87.3	50.0	75.1	41.4	62.2	32.0	48.1		
	40	52.4	78.8	45.1	67.8	37.4	56.2	28.9	43.4		
	42	47.6	71.5	40.9	61.5	33.9	50.9	26.2	39.3		
	44	43.3	65.1	37.3	56.1	30.9	46.4	23.8	35.8		
	46	39.6	59.6	34.1	51.3	28.2	42.5	21.8	32.8		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		311	468	262	394	213	320	161	242	419	630
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		242	363	204	306	165	248	125	187	326	488
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		99.7	150	85.0	128	69.5	104	53.0	79.7	152	228
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		74.6	112	64.1	96.3	52.7	79.3	40.5	60.9	68.1	102
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		79.3	119	67.6	102	55.4	83.2	34.6	52.0	96.1	144
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		66.6	100	55.9	84.0	39.8	59.9	26.7	40.1	63.1	94.9
Properties											
Area, in.^{2}		10.4		8.76		7.10		5.37		14.0	
r_y, in.		2.78		2.81		2.84		2.87		1.92	
r_x/r_y		1.22		1.21		1.21		1.21		1.60	

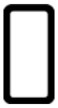
[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.



HSS9

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS9x5x									
		½		⅜		5/16[a]		1/4[a, c]		3/16[a, c]	
t_{des}, in.		0.465		0.349		0.291		0.233		0.174	
lb/ft		42.1		32.6		27.6		22.4		17.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	347	522	269	404	227	342	181	272	120	180
	1	346	521	268	403	227	341	180	271	120	180
	2	344	516	266	400	225	338	179	270	119	179
	3	339	509	262	394	222	334	178	267	118	177
	4	333	500	258	387	218	328	175	263	116	175
	5	325	488	252	379	213	321	172	259	114	172
	6	315	473	245	368	208	312	169	253	112	168
	7	304	457	237	356	201	302	164	246	109	164
	8	292	439	228	343	194	291	158	238	106	160
	9	279	419	218	328	186	279	152	228	103	155
	10	265	398	208	313	177	266	145	218	99.4	149
	11	250	376	197	296	168	252	138	207	95.5	144
	12	235	353	186	279	158	238	130	196	91.5	138
	13	220	330	174	262	149	224	122	184	87.3	131
	14	204	307	163	245	139	209	115	172	83.0	125
	15	189	284	151	227	129	194	107	161	78.5	118
	16	173	261	140	210	120	180	99.1	149	74.0	111
	17	159	238	128	193	110	166	91.4	137	69.5	104
	18	144	217	117	176	101	152	84.0	126	64.5	97.0
	19	130	196	107	160	92.0	138	76.7	115	59.1	88.8
	20	117	177	96.5	145	83.2	125	69.7	105	53.7	80.8
	22	97.1	146	79.7	120	68.8	103	57.6	86.5	44.4	66.8
	24	81.6	123	67.0	101	57.8	86.9	48.4	72.7	37.3	56.1
	26	69.5	104	57.1	85.8	49.3	74.0	41.2	62.0	31.8	47.8
	28	59.9	90.1	49.2	74.0	42.5	63.8	35.5	53.4	27.4	41.2
	30	52.2	78.5	42.9	64.4	37.0	55.6	31.0	46.5	23.9	35.9
	32	45.9	69.0	37.7	56.6	32.5	48.9	27.2	40.9	21.0	31.6
	34					28.8	43.3	24.1	36.2	18.6	27.9
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		347	522	269	404	227	342	185	278	140	210
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		270	405	209	313	176	265	143	215	109	163
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		127	191	99.7	150	85.0	128	69.5	104	53.0	79.7
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		60.1	90.4	49.5	74.4	43.2	64.9	36.0	54.1	28.0	42.1
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		81.1	122	64.1	96.4	54.9	82.5	45.2	67.9	34.4	51.8
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		53.6	80.6	42.7	64.1	35.8	53.8	25.9	38.9	17.2	25.9
Properties											
Area, in.^{2}		11.6		8.97		7.59		6.17		4.67	
r_y, in.		1.97		2.03		2.05		2.08		2.10	
r_x/r_y		1.59		1.58		1.58		1.57		1.58	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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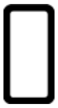

HSS9

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS9x3x									
		½		⅜		5⁄16[a]		¼[a, c]		3⁄16[a, c]	
t_{des}, in.		0.465		0.349		0.291		0.233		0.174	
lb/ft		35.2		27.5		23.3		19.0		14.5	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	292	438	227	341	193	289	153	230	99.2	149
	1	289	435	225	339	191	287	152	229	98.7	148
	2	283	425	221	331	187	282	150	225	97.3	146
	3	272	409	213	320	181	272	146	220	95.1	143
	4	258	388	202	304	173	259	141	212	92.0	138
	5	241	362	190	285	162	244	133	200	88.2	133
	6	221	332	175	263	150	226	124	186	83.8	126
	7	200	301	160	240	138	207	114	171	78.8	118
	8	178	268	143	215	124	187	103	155	73.4	110
	9	156	235	127	191	111	166	92.5	139	67.7	102
	10	135	203	111	166	97.1	146	81.7	123	61.8	92.9
	11	115	173	95.1	143	84.1	126	71.2	107	55.4	83.3
	12	96.6	145	80.4	121	71.7	108	61.3	92.1	47.9	72.0
	13	82.3	124	68.5	103	61.1	91.8	52.2	78.5	40.9	61.5
	14	71.0	107	59.1	88.8	52.7	79.1	45.0	67.6	35.3	53.0
	15	61.9	93.0	51.5	77.4	45.9	68.9	39.2	58.9	30.7	46.2
	16	54.4	81.7	45.2	68.0	40.3	60.6	34.5	51.8	27.0	40.6
	17	48.2	72.4	40.1	60.2	35.7	53.7	30.5	45.9	23.9	36.0
	18	43.0	64.6	35.8	53.7	31.9	47.9	27.2	40.9	21.3	32.1
	19	38.6	57.9	32.1	48.2	28.6	43.0	24.4	36.7	19.2	28.8
	20			29.0	43.5	25.8	38.8	22.1	33.1	17.3	26.0
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		292	438	227	341	193	289	157	236	119	179
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		226	340	176	264	149	224	122	183	92.5	139
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		127	191	99.7	150	85.0	128	69.5	104	53.0	79.7
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		26.7	40.2	24.5	36.7	22.3	33.5	19.3	28.9	15.5	23.3
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		61.4	92.3	49.2	73.9	42.2	63.4	34.9	52.5	26.9	40.5
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		26.9	40.5	22.0	33.0	18.7	28.1	13.5	20.4	9.02	13.6
Properties											
Area, in.^{2}		9.74		7.58		6.43		5.24		3.98	
r_y, in.		1.17		1.21		1.24		1.27		1.29	
r_x/r_y		2.46		2.45		2.42		2.39		2.38	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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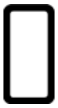

HSS8

Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS8x6x									
		⅝		½		⅜		5⁄16		¼[a]	
t_{des}, in.		0.581		0.465		0.349		0.291		0.233	
lb/ft		50.8		42.1		32.6		27.6		22.4	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	419	630	347	522	269	404	227	342	185	278
	1	418	629	347	521	268	403	227	341	184	277
	2	416	625	345	518	267	401	226	339	183	276
	3	412	619	341	513	264	397	224	336	182	273
	4	406	610	337	506	261	392	221	332	180	270
	5	398	599	331	497	256	385	217	326	177	266
	6	389	585	324	487	251	378	213	320	173	260
	7	379	570	316	474	245	369	208	312	169	254
	8	368	553	306	461	238	358	202	304	165	248
	9	355	534	296	446	231	347	196	295	160	240
	10	342	514	286	429	223	335	189	284	155	232
	11	327	492	274	412	214	322	182	274	149	224
	12	312	469	262	394	205	309	175	263	143	215
	13	297	446	250	375	196	295	167	251	137	205
	14	281	422	237	356	187	280	159	239	130	196
	15	265	398	224	336	177	266	151	226	124	186
	16	248	373	210	316	167	251	142	214	117	176
	17	232	349	197	297	157	236	134	201	110	166
	18	216	325	184	277	147	221	126	189	104	156
	19	200	301	171	258	137	206	117	177	97.0	146
	20	185	278	159	239	128	192	109	164	90.5	136
	22	156	234	135	202	109	164	93.8	141	77.9	117
	24	131	196	113	170	92.1	138	79.2	119	66.0	99.2
	26	111	167	96.4	145	78.5	118	67.5	101	56.3	84.6
	28	96.0	144	83.1	125	67.6	102	58.2	87.5	48.5	72.9
	30	83.7	126	72.4	109	58.9	88.6	50.7	76.2	42.3	63.5
	32	73.5	111	63.6	95.7	51.8	77.8	44.6	67.0	37.1	55.8
	34	65.1	97.9	56.4	84.7	45.9	69.0	39.5	59.3	32.9	49.4
	36	58.1	87.3	50.3	75.6	40.9	61.5	35.2	52.9	29.3	44.1
	38			45.1	67.8	36.7	55.2	31.6	47.5	26.3	39.6
	40							28.5	42.9	23.8	35.7
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		419	630	347	522	269	404	227	342	185	278
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		326	488	270	405	209	313	176	265	143	215
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		131	196	110	166	87.1	131	74.5	112	61.1	91.8
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		88.9	134	77.0	116	62.1	93.3	53.6	80.6	44.4	66.7
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		90.1	135	76.1	114	60.1	90.4	51.4	77.3	42.2	63.4
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		73.6	111	62.1	93.4	49.4	74.3	42.2	63.4	31.8	47.8
Properties											
Area, in.^{2}		14.0		11.6		8.97		7.59		6.17	
r_y, in.		2.27		2.32		2.38		2.40		2.43	
r_x/r_y		1.26		1.25		1.25		1.25		1.25	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS8

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS8x6x		HSS8x4x							
		3⁄16[a, b, c]		⅝		½		⅜		5⁄16	
t_{des}, in.		0.174		0.581		0.465		0.349		0.291	
lb/ft		17.1		42.3		35.2		27.5		23.3	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	128	192	350	526	292	438	227	341	193	289
	1	128	192	349	524	290	436	226	340	192	288
	2	127	191	344	517	287	431	223	336	189	285
	3	126	190	336	505	280	422	219	329	186	279
	4	125	188	325	489	272	409	213	320	181	272
	5	124	186	312	469	262	393	205	308	174	262
	6	122	183	297	446	250	375	196	295	167	251
	7	119	180	279	420	236	355	186	280	159	238
	8	117	176	261	392	221	332	175	263	149	225
	9	114	172	241	362	205	309	163	245	140	210
	10	111	167	221	332	189	284	151	227	130	195
	11	108	162	200	301	173	260	139	209	119	179
	12	104	157	180	271	156	235	126	190	109	164
	13	101	151	161	241	140	211	114	172	98.5	148
	14	96.9	146	142	213	125	188	102	154	88.5	133
	15	93.0	140	124	186	110	165	91.0	137	78.9	119
	16	88.9	134	109	163	96.6	145	80.1	120	69.7	105
	17	84.6	127	96.4	145	85.6	129	71.0	107	61.7	92.7
	18	79.6	120	85.9	129	76.4	115	63.3	95.1	55.0	82.7
	19	74.6	112	77.1	116	68.5	103	56.8	85.4	49.4	74.2
	20	69.7	105	69.6	105	61.9	93.0	51.3	77.1	44.6	67.0
	22	60.2	90.5	57.5	86.5	51.1	76.8	42.4	63.7	36.8	55.4
	24	51.2	77.0	48.3	72.7	43.0	64.6	35.6	53.5	31.0	46.5
	26	43.6	65.6			36.6	55.0	30.3	45.6	26.4	39.6
	28	37.6	56.6								
	30	32.8	49.3								
	32	28.8	43.3								
	34	25.5	38.4								
	36	22.8	34.2								
	38	20.4	30.7								
	40	18.4	27.7								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		140	210	350	527	292	438	227	341	193	289
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		109	163	272	408	226	340	176	264	149	224
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		46.8	70.3	131	196	110	166	87.1	131	74.5	112
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		34.3	51.5	47.2	70.9	43.4	65.3	37.0	55.6	32.7	49.2
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		28.9	43.5	68.4	103	58.6	88.1	46.9	70.5	40.2	60.4
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		21.1	31.7	41.4	62.3	35.7	53.6	28.7	43.1	24.7	37.2
Properties											
Area, in.^{2}		4.67		11.7		9.74		7.58		6.43	
r_y, in.		2.46		1.51		1.56		1.61		1.63	
r_x/r_y		1.24		1.75		1.74		1.73		1.73	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.
[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS8

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS8x4x						HSS8x3x			
		¼[a]		3⁄16[a, c]		⅛[a, b, c]		½		⅜	
t_{des}, in.		0.233		0.174		0.116		0.465		0.349	
lb/ft		19.0		14.5		9.86		31.8		24.9	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	157	236	107	161	60.0	90.1	264	396	206	310
	1	156	235	107	161	59.8	89.9	262	393	204	307
	2	155	232	106	160	59.3	89.1	256	384	200	301
	3	152	228	105	157	58.5	87.9	246	369	193	290
	4	148	222	103	154	57.3	86.2	232	349	183	275
	5	143	214	99.9	150	55.9	84.0	216	325	172	258
	6	137	205	96.8	146	54.2	81.5	198	298	158	238
	7	130	196	93.3	140	52.3	78.6	179	268	144	216
	8	123	185	89.3	134	50.2	75.4	158	238	129	194
	9	115	173	85.0	128	47.9	71.9	138	208	114	171
	10	107	161	80.4	121	45.4	68.2	119	179	99.2	149
	11	98.8	149	75.7	114	42.8	64.4	101	151	85.0	128
	12	90.5	136	70.1	105	40.2	60.4	84.5	127	71.8	108
	13	82.3	124	63.9	96.1	37.5	56.3	72.0	108	61.2	92.0
	14	74.2	112	57.9	87.0	34.8	52.3	62.0	93.3	52.8	79.3
	15	66.4	99.8	52.0	78.1	32.1	48.2	54.1	81.2	46.0	69.1
	16	58.9	88.5	46.3	69.7	29.4	44.2	47.5	71.4	40.4	60.7
	17	52.2	78.4	41.1	61.7	26.8	40.2	42.1	63.2	35.8	53.8
	18	46.5	69.9	36.6	55.0	24.5	36.8	37.5	56.4	31.9	48.0
	19	41.8	62.8	32.9	49.4	22.5	33.8	33.7	50.6	28.6	43.1
	20	37.7	56.6	29.7	44.6	20.6	31.0			25.9	38.9
	22	31.1	46.8	24.5	36.8	17.0	25.6				
	24	26.2	39.3	20.6	31.0	14.3	21.5				
	26	22.3	33.5	17.6	26.4	12.2	18.3				
	28			15.1	22.7	10.5	15.8				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		157	236	119	179	80.8	122	264	396	206	310
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		122	183	92.5	139	62.8	94.2	205	307	160	240
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		61.1	91.8	46.8	70.3	28.7	43.1	110	166	87.1	131
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		27.6	41.5	21.8	32.7	15.2	22.9	26.7	40.2	24.5	36.7
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		33.2	49.9	25.4	38.3	15.4	23.1	49.9	75.0	40.2	60.4
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		18.9	28.4	12.5	18.8	6.94	10.4	24.1	36.2	19.7	29.6
Properties											
Area, in.^{2}		5.24		3.98		2.70		8.81		6.88	
r_y, in.		1.66		1.69		1.71		1.15		1.20	
r_x/r_y		1.72		1.70		1.71		2.24		2.21	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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HSS8

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS8x3x								HSS8x2x	
		5⁄16		1⁄4[a]		3⁄16[a, c]		1⁄8[a, b, c]		3⁄8	
t_{des}, in.		0.291		0.233		0.174		0.116		0.349	
lb/ft		21.2		17.3		13.3		9.01		22.4	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	175	263	143	215	96.9	146	52.8	79.3	185	278
	1	174	261	142	213	96.4	145	52.5	79.0	182	273
	2	170	256	139	209	95.0	143	51.8	77.9	173	259
	3	165	247	134	202	92.8	139	50.6	76.1	158	238
	4	157	236	128	193	89.7	135	49.1	73.7	140	210
	5	147	221	121	181	85.8	129	47.1	70.8	120	180
	6	136	205	112	168	81.3	122	44.8	67.3	98.8	148
	7	125	187	103	154	76.3	115	42.2	63.4	78.7	118
	8	112	169	92.8	139	70.8	106	39.4	59.2	60.9	91.5
	9	99.7	150	82.7	124	64.6	97.1	36.5	54.8	48.1	72.3
	10	87.3	131	72.8	109	57.2	85.9	33.4	50.3	38.9	58.5
	11	75.5	113	63.2	95.0	49.9	75.1	30.4	45.7	32.2	48.4
	12	64.2	96.4	54.0	81.2	43.1	64.7	27.3	41.1	27.0	40.6
	13	54.7	82.2	46.0	69.2	36.7	55.2	24.3	36.5		
	14	47.1	70.8	39.7	59.7	31.7	47.6	21.7	32.6		
	15	41.1	61.7	34.6	52.0	27.6	41.5	19.5	29.3		
	16	36.1	54.2	30.4	45.7	24.2	36.4	17.2	25.9		
	17	32.0	48.0	26.9	40.5	21.5	32.3	15.2	22.9		
	18	28.5	42.9	24.0	36.1	19.2	28.8	13.6	20.4		
	19	25.6	38.5	21.6	32.4	17.2	25.8	12.2	18.3		
	20	23.1	34.7	19.4	29.2	15.5	23.3	11.0	16.6		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		175	263	143	215	109	163	73.7	111	185	278
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		136	204	111	166	84.4	127	57.2	85.8	144	216
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		74.5	112	61.1	91.8	46.8	70.3	28.7	43.1	87.1	131
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		22.3	33.5	19.3	28.9	15.5	23.3	11.0	16.6	11.9	18.0
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		34.7	52.1	28.7	43.1	22.1	33.3	14.9	22.4	33.4	50.3
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		17.1	25.7	13.1	19.7	8.70	13.1	4.80	7.21	11.5	17.3
Properties											
Area, in.^{2}		5.85		4.77		3.63		2.46		6.18	
r_y, in.		1.23		1.25		1.28		1.31		0.777	
r_x/r_y		2.19		2.18		2.16		2.14		3.20	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS8–HSS7

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS8x2x								HSS7x5x	
		5/16		1/4[a]		3/16[a, c]		1/8[a, b, c]		1/2	
t_{des}, in.		0.291		0.233		0.174		0.116		0.465	
lb/ft		19.1		15.6		12.0		8.16		35.2	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	157	237	129	193	86.4	130	45.9	69.0	292	438
	1	155	233	127	191	85.5	129	45.4	68.3	291	437
	2	148	222	121	182	82.9	125	44.2	66.4	288	433
	3	136	204	112	168	78.6	118	42.1	63.2	284	427
	4	121	182	101	151	73.0	110	39.3	59.1	278	419
	5	105	157	87.6	132	66.3	99.6	36.1	54.2	271	408
	6	87.4	131	74.0	111	58.3	87.7	32.4	48.7	263	395
	7	70.6	106	60.6	91.0	48.3	72.6	28.6	43.0	253	380
	8	55.2	82.9	48.0	72.1	38.9	58.5	24.7	37.1	242	364
	9	43.6	65.5	37.9	57.0	30.8	46.2	20.9	31.4	231	347
	10	35.3	53.1	30.7	46.1	24.9	37.4	17.9	26.8	219	328
	11	29.2	43.9	25.4	38.1	20.6	30.9	14.9	22.3	206	309
	12	24.5	36.9	21.3	32.0	17.3	26.0	12.5	18.8	192	289
	13	20.9	31.4	18.2	27.3	14.7	22.2	10.6	16.0	179	269
	14					12.7	19.1	9.18	13.8	166	249
	15									152	229
	16									139	209
	17									127	190
	18									114	172
	19									103	154
	20									92.7	139
	22									76.6	115
	24									64.4	96.8
	26									54.9	82.5
	28									47.3	71.1
	30									41.2	61.9
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		157	237	129	194	98.2	148	66.8	100	292	438
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		122	183	100	150	76.3	114	51.8	77.8	226	340
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		74.5	112	61.1	91.8	46.8	70.3	28.7	43.1	93.6	141
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		11.8	17.8	10.9	16.4	9.25	13.9	6.88	10.3	60.1	90.4
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		28.9	43.5	24.2	36.3	18.7	28.2	12.6	19.0	54.6	82.1
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		10.1	15.2	7.78	11.7	5.21	7.83	2.86	4.30	43.2	64.9
Properties											
Area, in.^{2}		5.26		4.30		3.28		2.23		9.74	
r_y, in.		0.802		0.827		0.853		0.879		1.91	
r_x/r_y		3.15		3.11		3.06		3.01		1.31	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.
[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS7

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS7x5x									
		⅜		5/16		¼[a]		3/16[a, c]		⅛[a, b, c]	
t_{des}, in.		0.349		0.291		0.233		0.174		0.116	
lb/ft		27.5		23.3		19.0		14.5		9.86	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	227	341	193	289	157	236	115	173	62.6	94.1
	1	226	340	192	289	156	235	115	173	62.5	94.0
	2	224	337	190	286	155	233	114	171	62.2	93.5
	3	221	333	188	283	153	230	113	170	61.7	92.8
	4	217	327	184	277	151	226	111	167	61.1	91.8
	5	212	319	180	271	147	221	109	164	60.2	90.5
	6	206	309	175	263	143	215	107	161	59.2	88.9
	7	199	299	169	254	138	208	104	156	58.0	87.1
	8	191	287	162	244	133	200	101	152	56.6	85.1
	9	182	274	155	233	127	191	97.3	146	55.1	82.8
	10	173	260	148	222	121	182	92.8	139	53.4	80.3
	11	163	246	140	210	115	173	88.0	132	51.6	77.6
	12	154	231	131	197	108	163	83.1	125	49.7	74.7
	13	143	216	123	185	101	152	78.0	117	47.3	71.1
	14	133	200	114	172	94.6	142	72.9	110	44.8	67.4
	15	123	185	106	159	87.8	132	67.8	102	42.3	63.6
	16	113	170	97.5	146	81.0	122	62.7	94.3	39.8	59.8
	17	104	156	89.3	134	74.4	112	57.8	86.8	37.3	56.0
	18	94.2	142	81.3	122	68.0	102	52.9	79.5	34.7	52.2
	19	85.1	128	73.6	111	61.8	92.9	48.2	72.5	32.3	48.5
	20	76.8	115	66.4	99.9	55.8	83.9	43.6	65.6	29.8	44.8
	22	63.4	95.4	54.9	82.5	46.1	69.3	36.1	54.2	25.0	37.5
	24	53.3	80.1	46.1	69.4	38.7	58.2	30.3	45.6	21.0	31.5
	26	45.4	68.3	39.3	59.1	33.0	49.6	25.8	38.8	17.9	26.8
	28	39.2	58.9	33.9	51.0	28.5	42.8	22.3	33.5	15.4	23.2
	30	34.1	51.3	29.5	44.4	24.8	37.3	19.4	29.2	13.4	20.2
	32	30.0	45.1	26.0	39.0	21.8	32.8	17.0	25.6	11.8	17.7
	34							15.1	22.7	10.4	15.7
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		227	341	193	289	157	236	119	179	80.8	122
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		176	264	149	224	122	183	92.5	139	62.8	94.2
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		74.6	112	64.1	96.3	52.7	79.3	40.5	60.9	27.7	41.7
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		49.5	74.4	43.2	64.9	36.0	54.1	28.0	42.1	19.4	29.1
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		43.7	65.6	37.4	56.3	30.9	46.5	23.8	35.7	13.0	19.5
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		34.4	51.8	29.7	44.6	24.5	36.9	15.9	23.8	9.05	13.6
Properties											
Area, in.^{2}		7.58		6.43		5.24		3.98		2.70	
r_y, in.		1.97		1.99		2.02		2.05		2.07	
r_x/r_y		1.30		1.30		1.30		1.29		1.29	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS7

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS7x4x									
		½		⅜		5⁄16		¼[a]		3⁄16[a, c]	
t_{des}, in.		0.465		0.349		0.291		0.233		0.174	
lb/ft		31.8		24.9		21.2		17.3		13.3	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	264	396	206	310	175	263	143	215	105	157
	1	263	395	205	308	174	262	142	214	104	157
	2	259	389	203	304	172	259	141	211	103	155
	3	253	381	198	298	169	254	138	207	102	153
	4	245	369	193	289	164	247	134	202	99.6	150
	5	236	354	185	279	158	238	129	195	96.9	146
	6	224	337	177	266	151	227	124	186	93.6	141
	7	212	318	168	252	144	216	118	177	90.0	135
	8	198	297	157	236	135	203	111	167	85.1	128
	9	183	275	146	220	126	189	104	156	79.8	120
	10	168	253	135	203	117	175	96.6	145	74.2	111
	11	153	230	124	186	107	161	88.9	134	68.4	103
	12	138	207	112	169	97.6	147	81.3	122	62.7	94.2
	13	123	185	101	152	88.2	133	73.7	111	57.0	85.6
	14	109	164	90.1	135	79.0	119	66.3	99.7	51.4	77.2
	15	95.7	144	79.7	120	70.2	106	59.2	89.0	46.0	69.1
	16	84.1	126	70.0	105	61.8	92.9	52.3	78.6	40.8	61.3
	17	74.5	112	62.0	93.2	54.8	82.3	46.3	69.6	36.1	54.3
	18	66.4	99.9	55.3	83.2	48.9	73.4	41.3	62.1	32.2	48.4
	19	59.6	89.6	49.7	74.6	43.8	65.9	37.1	55.8	28.9	43.5
	20	53.8	80.9	44.8	67.4	39.6	59.5	33.5	50.3	26.1	39.2
	22	44.5	66.8	37.0	55.7	32.7	49.2	27.7	41.6	21.6	32.4
	24	37.4	56.2	31.1	46.8	27.5	41.3	23.2	34.9	18.1	27.2
	26			26.5	39.9	23.4	35.2	19.8	29.8	15.4	23.2
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		264	396	206	310	175	263	143	215	109	163
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		205	307	160	240	136	204	111	166	84.4	127
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		93.6	141	74.6	112	64.1	96.3	52.7	79.3	40.5	60.9
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		43.4	65.3	37.0	55.6	32.7	49.2	27.6	41.5	21.8	32.7
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		46.9	70.5	37.7	56.6	32.7	49.1	26.9	40.5	20.8	31.2
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		31.4	47.3	25.4	38.3	22.0	33.1	18.3	27.5	11.9	17.9
Properties											
Area, in.^{2}		8.81		6.88		5.85		4.77		3.63	
r_y, in.		1.53		1.58		1.61		1.64		1.66	
r_x/r_y		1.57		1.56		1.55		1.54		1.54	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.
[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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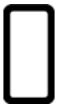

HSS7

Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

	Shape	HSS7x4x		HSS7x3x							
		⅛[a, b, c]		½		⅜		5⁄16		¼[a]	
	t_{des}, in.	0.116		0.465		0.349		0.291		0.233	
	lb/ft	9.01		28.4		22.4		19.1		15.6	
	Design	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
	Available Compressive Strength, kips	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	58.9	88.5	236	355	185	278	157	237	129	193
	1	58.7	88.3	234	352	184	276	156	235	128	192
	2	58.2	87.5	228	343	180	270	153	230	125	188
	3	57.4	86.3	219	330	173	260	148	222	121	182
	4	56.2	84.5	207	311	164	247	140	211	115	173
	5	54.8	82.4	193	290	154	231	132	198	108	163
	6	53.1	79.8	176	265	142	213	122	183	101	151
	7	51.1	76.9	159	238	129	193	111	166	92.0	138
	8	49.0	73.6	140	211	115	173	99.4	149	83.1	125
	9	46.6	70.1	122	184	101	152	88.0	132	73.9	111
	10	44.1	66.3	105	158	88.0	132	76.7	115	64.9	97.6
	11	41.5	62.4	88.3	133	75.3	113	66.0	99.2	56.2	84.5
	12	38.8	58.4	74.2	112	63.4	95.3	55.8	83.9	47.9	72.0
	13	36.1	54.3	63.3	95.1	54.1	81.2	47.6	71.5	40.8	61.4
	14	33.4	50.2	54.5	82.0	46.6	70.0	41.0	61.6	35.2	52.9
	15	30.7	46.1	47.5	71.4	40.6	61.0	35.7	53.7	30.7	46.1
	16	28.0	42.1	41.8	62.8	35.7	53.6	31.4	47.2	27.0	40.5
	17	25.4	38.1	37.0	55.6	31.6	47.5	27.8	41.8	23.9	35.9
	18	22.6	34.0	33.0	49.6	28.2	42.4	24.8	37.3	21.3	32.0
	19	20.3	30.5	29.6	44.5	25.3	38.0	22.3	33.5	19.1	28.7
	20	18.3	27.6					20.1	30.2	17.3	25.9
	22	15.2	22.8								
	24	12.7	19.1								
	26	10.8	16.3								
	28	9.35	14.1								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		73.7	111	236	355	185	278	157	237	129	194
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		57.2	85.8	183	275	144	216	122	183	100	150
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		27.7	41.7	93.6	141	74.6	112	64.1	96.3	52.7	79.3
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		15.2	22.9	26.7	40.2	24.5	36.7	22.3	33.5	19.3	28.9
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		12.6	19.0	39.4	59.3	31.9	48.0	27.7	41.6	23.0	34.6
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		6.73	10.1	21.1	31.7	17.3	26.1	15.1	22.7	12.6	19.0
Properties											
Area, in.^{2}		2.46		7.88		6.18		5.26		4.30	
r_y, in.		1.69		1.14		1.19		1.21		1.24	
r_x/r_y		1.53		1.99		1.97		1.97		1.95	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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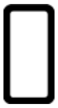

HSS7

Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS7x3x				HSS7x2x					
		3⁄16[a, c]		1⁄8[a, c]		1⁄4[a]		3⁄16[a, c]		1⁄8[a, c]	
t_{des}, in.		0.174		0.116		0.233		0.174		0.116	
lb/ft		12.0		8.16		13.9		10.7		7.31	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	94.1	141	52.0	78.2	115	173	83.6	126	45.1	67.8
	1	93.6	141	51.8	77.8	113	170	82.7	124	44.7	67.2
	2	92.2	139	51.0	76.7	108	162	80.0	120	43.4	65.2
	3	89.8	135	49.8	74.9	99.8	150	75.6	114	41.3	62.0
	4	86.6	130	48.2	72.4	89.4	134	69.3	104	38.5	57.8
	5	82.7	124	46.2	69.4	77.7	117	60.7	91.2	35.1	52.8
	6	77.3	116	43.8	65.8	65.3	98.2	51.6	77.5	31.5	47.3
	7	71.0	107	41.1	61.8	53.3	80.1	42.6	64.0	27.6	41.4
	8	64.2	96.5	38.3	57.6	42.0	63.1	34.1	51.3	23.6	35.5
	9	57.4	86.3	35.3	53.0	33.2	49.9	27.0	40.5	19.6	29.4
	10	50.6	76.0	32.2	48.4	26.9	40.4	21.8	32.8	15.8	23.8
	11	44.0	66.2	29.1	43.7	22.2	33.4	18.0	27.1	13.1	19.7
	12	37.7	56.7	26.0	39.1	18.7	28.1	15.2	22.8	11.0	16.5
	13	32.2	48.3	22.9	34.4	15.9	23.9	12.9	19.4	9.37	14.1
	14	27.7	41.7	19.8	29.7			11.1	16.7	8.08	12.1
	15	24.2	36.3	17.2	25.9						
	16	21.2	31.9	15.1	22.7						
	17	18.8	28.3	13.4	20.1						
	18	16.8	25.2	12.0	18.0						
	19	15.1	22.6	10.7	16.1						
	20	13.6	20.4	9.68	14.6						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		98.2	148	66.8	100	115	173	87.7	132	59.9	90.0
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		76.3	114	51.8	77.8	89.3	134	68.1	102	46.5	69.8
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		40.5	60.9	27.7	41.7	52.7	79.3	40.5	60.9	27.7	41.7
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		15.5	23.3	11.0	16.6	10.9	16.4	9.25	13.9	6.88	10.3
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		17.8	26.8	12.3	18.5	19.1	28.7	14.8	22.3	10.3	15.5
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		8.30	12.5	4.64	6.98	7.53	11.3	4.97	7.47	2.79	4.19
Properties											
Area, in.^{2}		3.28		2.23		3.84		2.93		2.00	
r_y, in.		1.26		1.29		0.819		0.845		0.871	
r_x/r_y		1.94		1.93		2.77		2.73		2.70	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS6

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS6x5x									
		½		⅜		5⁄16		¼		3⁄16[a]	
t_{des}, in.		0.465		0.349		0.291		0.233		0.174	
lb/ft		31.8		24.9		21.2		17.3		13.3	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	264	396	206	310	175	263	143	215	109	163
	1	263	395	205	309	175	263	142	214	108	163
	2	261	392	204	306	173	260	141	212	108	162
	3	257	386	201	302	171	257	139	210	106	160
	4	251	378	197	296	168	252	137	206	104	157
	5	245	368	192	288	163	246	134	201	102	153
	6	237	356	186	279	159	238	130	195	98.9	149
	7	228	342	179	269	153	230	125	188	95.7	144
	8	218	327	172	258	147	220	120	181	92.0	138
	9	207	311	163	246	140	210	115	173	88.0	132
	10	195	293	155	233	133	200	109	164	83.7	126
	11	183	275	146	219	125	188	103	155	79.3	119
	12	171	257	137	205	118	177	97.0	146	74.7	112
	13	159	238	127	191	110	165	90.7	136	70.0	105
	14	146	220	118	177	102	153	84.4	127	65.2	98.0
	15	134	201	108	163	93.9	141	78.0	117	60.5	90.9
	16	122	183	99.2	149	86.2	130	71.8	108	55.8	83.8
	17	110	166	90.2	136	78.7	118	65.7	98.8	51.2	76.9
	18	99.3	149	81.6	123	71.4	107	59.8	89.9	46.7	70.2
	19	89.1	134	73.3	110	64.3	96.7	54.1	81.3	42.4	63.8
	20	80.4	121	66.2	99.5	58.0	87.2	48.8	73.3	38.3	57.5
	22	66.4	99.9	54.7	82.2	48.0	72.1	40.3	60.6	31.6	47.5
	24	55.8	83.9	46.0	69.1	40.3	60.6	33.9	50.9	26.6	39.9
	26	47.6	71.5	39.2	58.9	34.3	51.6	28.9	43.4	22.6	34.0
	28	41.0	61.6	33.8	50.8	29.6	44.5	24.9	37.4	19.5	29.3
	30	35.7	53.7	29.4	44.2	25.8	38.8	21.7	32.6	17.0	25.6
	32			25.9	38.9	22.7	34.1	19.1	28.7	14.9	22.5
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		264	396	206	310	175	263	143	215	109	163
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		205	307	160	240	136	204	111	166	84.4	127
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		77.0	116	62.1	93.3	53.6	80.6	44.4	66.7	34.3	51.5
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		60.1	90.4	49.5	74.4	43.2	64.9	36.0	54.1	28.0	42.1
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		42.9	64.5	34.4	51.8	29.7	44.6	24.6	37.0	19.0	28.6
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		37.9	57.0	30.4	45.8	26.2	39.4	21.8	32.7	15.3	23.0
Properties											
Area, in.^{2}		8.81		6.88		5.85		4.77		3.63	
r_y, in.		1.87		1.92		1.95		1.98		2.01	
r_x/r_y		1.16		1.16		1.15		1.15		1.15	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS6

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS6x5x		HSS6x4x							
		⅛[a, b, c]		½		⅜		5⁄16		¼	
t_{des}, in.		0.116		0.465		0.349		0.291		0.233	
lb/ft		9.01		28.4		22.4		19.1		15.6	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	61.3	92.1	236	355	185	278	157	237	129	193
	1	61.2	92.0	235	353	184	277	157	236	128	193
	2	60.9	91.5	232	348	182	273	155	233	127	190
	3	60.4	90.7	226	340	178	267	152	228	124	187
	4	59.7	89.7	219	329	173	259	147	221	121	181
	5	58.8	88.4	210	315	166	249	142	213	116	175
	6	57.7	86.8	199	300	158	238	135	203	111	167
	7	56.5	84.9	188	282	149	224	128	193	106	159
	8	55.1	82.8	175	263	140	210	120	181	99.3	149
	9	53.5	80.4	161	243	130	195	112	168	92.6	139
	10	51.8	77.9	148	222	119	179	103	155	85.8	129
	11	50.0	75.1	134	201	109	164	94.5	142	78.8	118
	12	47.9	71.9	120	181	98.4	148	85.8	129	71.7	108
	13	45.4	68.3	107	161	88.2	133	77.2	116	64.8	97.4
	14	42.9	64.5	94.3	142	78.4	118	68.9	104	58.1	87.3
	15	40.3	60.6	82.3	124	68.9	104	60.9	91.6	51.6	77.6
	16	37.8	56.8	72.3	109	60.5	91.0	53.5	80.5	45.4	68.3
	17	35.2	52.9	64.0	96.2	53.6	80.6	47.4	71.3	40.3	60.5
	18	32.2	48.4	57.1	85.9	47.8	71.9	42.3	63.6	35.9	54.0
	19	29.3	44.0	51.3	77.1	42.9	64.5	38.0	57.1	32.2	48.4
	20	26.5	39.8	46.3	69.5	38.7	58.2	34.3	51.5	29.1	43.7
	22	21.9	32.9	38.2	57.5	32.0	48.1	28.3	42.6	24.0	36.1
	24	18.4	27.6	32.1	48.3	26.9	40.4	23.8	35.8	20.2	30.4
	26	15.7	23.5					20.3	30.5	17.2	25.9
	28	13.5	20.3								
	30	11.8	17.7								
	32	10.3	15.5								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		73.7	111	236	355	185	278	157	237	129	194
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		57.2	85.8	183	275	144	216	122	183	100	150
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		23.5	35.4	77.0	116	62.1	93.3	53.6	80.6	44.4	66.7
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		19.4	29.1	43.4	65.3	37.0	55.6	32.7	49.2	27.6	41.5
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		10.5	15.7	36.4	54.8	29.7	44.6	25.7	38.6	21.3	32.0
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		8.68	13.0	27.4	41.3	22.3	33.5	19.3	29.1	16.1	24.2
Properties											
Area, in.^{2}		2.46		7.88		6.18		5.26		4.30	
r_y, in.		2.03		1.50		1.55		1.58		1.61	
r_x/r_y		1.15		1.39		1.38		1.37		1.37	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

HSS6

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS6x4x				HSS6x3x					
		3/16[a]		1/8[a, b, c]		1/2		3/8		5/16	
t_{des}, in.		0.174		0.116		0.465		0.349		0.291	
lb/ft		12.0		8.16		25.0		19.8		17.0	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	98.2	148	57.9	87.0	208	313	164	247	140	211
	1	97.8	147	57.7	86.7	206	310	163	245	139	209
	2	96.7	145	57.2	85.9	201	302	159	239	136	204
	3	94.8	142	56.3	84.6	193	290	153	230	131	197
	4	92.2	139	55.1	82.8	182	273	145	218	124	187
	5	88.9	134	53.6	80.6	169	254	135	203	116	175
	6	85.1	128	51.9	77.9	154	231	124	187	107	161
	7	80.9	122	49.8	74.9	138	207	113	169	97.3	146
	8	76.2	115	47.6	71.5	122	183	100	151	87.1	131
	9	71.2	107	45.2	67.9	105	158	88.0	132	76.7	115
	10	66.1	99.3	42.6	64.1	89.9	135	76.0	114	66.6	100
	11	60.8	91.4	39.9	60.0	75.2	113	64.7	97.2	57.0	85.7
	12	55.5	83.4	37.2	55.9	63.2	95.0	54.4	81.7	48.0	72.2
	13	50.3	75.5	34.4	51.8	53.8	80.9	46.3	69.6	40.9	61.5
	14	45.2	67.9	31.6	47.5	46.4	69.8	39.9	60.0	35.3	53.0
	15	40.3	60.5	28.3	42.5	40.4	60.8	34.8	52.3	30.7	46.2
	16	35.5	53.4	25.1	37.7	35.5	53.4	30.6	46.0	27.0	40.6
	17	31.5	47.3	22.2	33.4	31.5	47.3	27.1	40.7	23.9	36.0
	18	28.1	42.2	19.8	29.8	28.1	42.2	24.2	36.3	21.4	32.1
	19	25.2	37.9	17.8	26.7			21.7	32.6	19.2	28.8
	20	22.7	34.2	16.0	24.1						
	22	18.8	28.2	13.3	19.9						
	24	15.8	23.7	11.1	16.7						
	26	13.5	20.2	9.49	14.3						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		98.2	148	66.8	100	208	313	164	247	140	211
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		76.3	114	51.8	77.8	162	242	127	191	109	163
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		34.3	51.5	23.5	35.4	77.0	116	62.1	93.3	53.6	80.6
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		21.8	32.7	15.2	22.9	26.7	40.2	24.5	36.7	22.3	33.5
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		16.5	24.8	10.1	15.2	30.2	45.4	24.7	37.1	21.5	32.3
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		11.4	17.2	6.46	9.71	18.2	27.3	15.0	22.6	13.1	19.8
Properties											
Area, in.^{2}		3.28		2.23		6.95		5.48		4.68	
r_y, in.		1.63		1.66		1.12		1.17		1.19	
r_x/r_y		1.37		1.36		1.76		1.74		1.74	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS6

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS6x3x						HSS6x2x			
		¼		3⁄16[a]		⅛[a, c]		⅜		5⁄16	
t_{des}, in.		0.233		0.174		0.116		0.349		0.291	
lb/ft		13.9		10.7		7.31		17.3		14.8	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	115	173	87.7	132	51.0	76.7	143	215	123	184
	1	114	172	87.1	131	50.7	76.3	141	211	121	181
	2	112	168	85.4	128	50.0	75.1	133	200	115	172
	3	108	162	82.6	124	48.7	73.2	121	183	105	158
	4	103	154	78.8	118	47.0	70.7	107	161	93.4	140
	5	96.3	145	74.1	111	44.9	67.6	90.7	136	80.1	120
	6	89.1	134	68.8	103	42.5	63.9	74.2	112	66.4	99.7
	7	81.3	122	63.1	94.8	39.8	59.8	58.6	88.0	53.1	79.9
	8	73.1	110	57.0	85.7	36.9	55.4	45.0	67.7	41.2	61.9
	9	64.8	97.4	50.8	76.4	33.8	50.8	35.6	53.5	32.6	48.9
	10	56.7	85.2	44.7	67.2	30.6	46.1	28.8	43.3	26.4	39.6
	11	48.8	73.4	38.8	58.3	27.2	40.9	23.8	35.8	21.8	32.8
	12	41.4	62.3	33.2	49.9	23.4	35.2	20.0	30.1	18.3	27.5
	13	35.3	53.1	28.3	42.5	19.9	29.9			15.6	23.5
	14	30.4	45.7	24.4	36.6	17.2	25.8				
	15	26.5	39.9	21.2	31.9	15.0	22.5				
	16	23.3	35.0	18.7	28.1	13.2	19.8				
	17	20.6	31.0	16.5	24.9	11.7	17.5				
	18	18.4	27.7	14.7	22.2	10.4	15.6				
	19	16.5	24.8	13.2	19.9	9.33	14.0				
	20	14.9	22.4	11.9	18.0	8.42	12.7				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		115	173	87.7	132	59.9	90.0	143	215	123	185
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		89.3	134	68.1	102	46.5	69.8	111	167	95.3	143
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		44.4	66.7	34.3	51.5	23.5	35.4	62.1	93.3	53.6	80.6
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		19.3	28.9	15.5	23.3	11.0	16.6	11.9	18.0	11.8	17.8
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		17.9	27.0	13.9	21.0	9.66	14.5	19.8	29.7	17.3	26.1
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		11.0	16.5	7.91	11.9	4.46	6.71	8.63	13.0	7.66	11.5
Properties											
Area, in.^{2}		3.84		2.93		2.00		4.78		4.10	
r_y, in.		1.22		1.25		1.27		0.760		0.785	
r_x/r_y		1.72		1.71		1.71		2.49		2.46	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS6–HSS5

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape	HSS6x2x						HSS5x4x			
	¼		³⁄₁₆[a]		⅛[a, c]		½		⅜	
t_{des}, in.	0.233		0.174		0.116		0.465		0.349	
lb/ft	12.2		9.42		6.46		25.0		19.8	
Design	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y										
0	101	152	77.2	116	44.1	66.3	208	313	164	247
1	99.3	149	76.1	114	43.7	65.6	207	311	163	245
2	94.6	142	72.7	109	42.3	63.6	204	307	161	242
3	87.3	131	67.5	101	40.1	60.3	199	299	157	237
4	78.0	117	60.7	91.2	37.3	56.0	192	289	153	229
5	67.6	102	53.0	79.7	33.9	50.9	184	276	146	220
6	56.6	85.1	44.9	67.5	30.1	45.3	174	262	139	209
7	46.0	69.1	36.9	55.5	26.2	39.4	163	246	131	197
8	36.1	54.2	29.4	44.2	21.4	32.1	152	228	123	184
9	28.5	42.8	23.2	34.9	16.9	25.4	139	210	113	170
10	23.1	34.7	18.8	28.3	13.7	20.6	127	191	104	156
11	19.1	28.7	15.6	23.4	11.3	17.0	114	172	94.5	142
12	16.0	24.1	13.1	19.6	9.51	14.3	102	154	85.1	128
13	13.7	20.5	11.1	16.7	8.10	12.2	90.3	136	76.0	114
14					6.99	10.5	78.9	119	67.2	101
15							68.7	103	58.7	88.3
16							60.4	90.8	51.6	77.6
17							53.5	80.4	45.7	68.7
18							47.7	71.7	40.8	61.3
19							42.8	64.4	36.6	55.0
20							38.7	58.1	33.0	49.7
22							31.9	48.0	27.3	41.0
24							26.8	40.4	22.9	34.5
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	101	152	77.2	116	53.0	79.7	208	313	164	247
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	78.4	118	60.0	90.0	41.2	61.7	162	242	127	191
Available Strength in Shear about X-X Axis, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	44.4	66.7	34.3	51.5	23.5	35.4	60.1	90.4	49.5	74.4
Available Strength in Shear about Y-Y Axis, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	10.9	16.4	9.25	13.9	6.88	10.3	43.4	65.3	37.0	55.6
Available Strength in Flexure about X-X Axis, kip-ft	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	14.6	21.9	11.4	17.2	7.96	12.0	27.2	40.9	22.4	33.6
Available Strength in Flexure about Y-Y Axis, kip-ft	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	6.51	9.79	4.71	7.08	2.67	4.02	23.3	35.1	19.1	28.8
Properties										
Area, in.^{2}	3.37		2.58		1.77		6.95		5.48	
r_y, in.	0.810		0.836		0.861		1.46		1.52	
r_x/r_y	2.43		2.40		2.38		1.20		1.19	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS5

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS5x4x								HSS5x3x	
		5⁄16		¼		3⁄16		⅛[a, b, c]		½	
t_{des}, in.		0.291		0.233		0.174		0.116		0.465	
lb/ft		17.0		13.9		10.7		7.31		21.6	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	140	211	115	173	87.7	132	56.4	84.8	180	271
	1	139	210	114	172	87.4	131	56.2	84.5	179	269
	2	138	207	113	170	86.3	130	55.7	83.6	174	261
	3	135	202	111	166	84.5	127	54.7	82.3	166	250
	4	131	196	107	161	82.1	123	53.5	80.4	156	235
	5	125	188	103	155	79.2	119	51.9	78.0	144	217
	6	119	179	98.6	148	75.7	114	50.1	75.2	131	197
	7	113	169	93.3	140	71.7	108	48.0	72.1	117	175
	8	105	159	87.5	131	67.4	101	45.6	68.6	102	154
	9	97.8	147	81.3	122	62.9	94.5	43.1	64.8	87.9	132
	10	89.9	135	75.0	113	58.1	87.4	40.1	60.3	74.3	112
	11	81.9	123	68.6	103	53.3	80.2	36.9	55.4	61.7	92.7
	12	73.9	111	62.2	93.4	48.5	72.9	33.6	50.5	51.8	77.9
	13	66.2	99.5	55.9	84.0	43.8	65.8	30.4	45.7	44.2	66.4
	14	58.7	88.2	49.8	74.8	39.2	58.9	27.3	41.0	38.1	57.2
	15	51.5	77.4	43.9	66.0	34.8	52.3	24.3	36.5	33.2	49.9
	16	45.3	68.0	38.6	58.0	30.6	46.0	21.4	32.2	29.2	43.8
	17	40.1	60.3	34.2	51.4	27.1	40.7	19.0	28.5	25.8	38.8
	18	35.8	53.7	30.5	45.8	24.2	36.3	16.9	25.4	23.0	34.6
	19	32.1	48.2	27.4	41.1	21.7	32.6	15.2	22.8		
	20	29.0	43.5	24.7	37.1	19.6	29.4	13.7	20.6		
	22	23.9	36.0	20.4	30.7	16.2	24.3	11.3	17.0		
	24	20.1	30.2	17.2	25.8	13.6	20.4	9.51	14.3		
	26			14.6	22.0	11.6	17.4	8.10	12.2		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		140	211	115	173	87.7	132	59.9	90.0	180	271
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		109	163	89.3	134	68.1	102	46.5	69.8	140	210
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		43.2	64.9	36.0	54.1	28.0	42.1	19.4	29.1	60.1	90.4
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		32.7	49.2	27.6	41.5	21.8	32.7	15.2	22.9	26.7	40.2
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		19.4	29.2	16.2	24.3	12.6	18.9	7.85	11.8	22.0	33.1
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		16.6	25.0	13.9	20.9	10.8	16.3	6.11	9.18	15.2	22.9
Properties											
Area, in.^{2}		4.68		3.84		2.93		2.00		6.02	
r_y, in.		1.54		1.57		1.60		1.62		1.09	
r_x/r_y		1.19		1.19		1.19		1.19		1.51	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.
[b] Shape exceeds the compact limit for flexure about the Y-Y axis for F_y = 50 ksi.
[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
Notes: Heavy line indicates L_c/r equal to or greater than 200.
Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS5

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS5x3x									
		⅜		5⁄16		¼		3⁄16		⅛[a, c]	
t_{des}, in.		0.349		0.291		0.233		0.174		0.116	
lb/ft		17.3		14.8		12.2		9.42		6.46	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	143	215	123	184	101	152	77.2	116	49.5	74.4
	1	142	213	122	183	100	151	76.7	115	49.2	74.0
	2	139	208	119	179	97.9	147	75.1	113	48.5	72.8
	3	133	200	115	172	94.4	142	72.5	109	47.2	70.9
	4	126	189	109	163	89.6	135	69.0	104	45.4	68.3
	5	117	176	101	152	83.8	126	64.7	97.3	43.3	65.0
	6	107	161	93.1	140	77.2	116	59.9	90.0	40.8	61.3
	7	96.2	145	84.2	127	70.1	105	54.6	82.1	38.0	57.1
	8	85.2	128	75.0	113	62.7	94.2	49.1	73.8	34.4	51.7
	9	74.2	112	65.8	99.0	55.2	83.0	43.6	65.5	30.7	46.1
	10	63.7	95.7	56.9	85.5	48.0	72.1	38.1	57.2	27.0	40.6
	11	53.6	80.5	48.4	72.7	41.0	61.7	32.8	49.3	23.4	35.2
	12	45.0	67.7	40.7	61.1	34.6	52.0	27.8	41.8	20.0	30.1
	13	38.4	57.7	34.7	52.1	29.5	44.3	23.7	35.6	17.1	25.7
	14	33.1	49.7	29.9	44.9	25.4	38.2	20.5	30.7	14.7	22.1
	15	28.8	43.3	26.0	39.1	22.1	33.3	17.8	26.8	12.8	19.3
	16	25.3	38.1	22.9	34.4	19.5	29.2	15.7	23.5	11.3	16.9
	17	22.4	33.7	20.3	30.5	17.2	25.9	13.9	20.8	9.99	15.0
	18	20.0	30.1	18.1	27.2	15.4	23.1	12.4	18.6	8.91	13.4
	19	18.0	27.0	16.2	24.4	13.8	20.7	11.1	16.7	8.00	12.0
	20							10.0	15.1	7.22	10.8
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		143	215	123	185	101	152	77.2	116	53.0	79.7
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		111	167	95.3	143	78.4	118	60.0	90.0	41.2	61.7
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		49.5	74.4	43.2	64.9	36.0	54.1	28.0	42.1	19.4	29.1
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		24.5	36.7	22.3	33.5	19.3	28.9	15.5	23.3	11.0	16.6
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		18.3	27.5	16.0	24.1	13.4	20.2	10.5	15.8	7.31	11.0
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		12.7	19.1	11.2	16.8	9.41	14.1	7.39	11.1	4.22	6.35
Properties											
Area, in.^{2}		4.78		4.10		3.37		2.58		1.77	
r_y, in.		1.14		1.17		1.19		1.22		1.25	
r_x/r_y		1.51		1.50		1.50		1.49		1.48	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS5

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS5x2½x						HSS5x2x			
		¼		3⁄16		⅛[a, c]		⅜		5⁄16	
t_{des}, in.		0.233		0.174		0.116		0.349		0.291	
lb/ft		11.4		8.78		6.03		14.7		12.7	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	94.0	141	72.2	108	45.9	69.0	122	184	105	158
	1	93.0	140	71.4	107	45.6	68.5	120	181	104	156
	2	90.1	135	69.3	104	44.6	67.0	114	171	98.2	148
	3	85.5	129	65.9	99.0	42.9	64.5	103	155	89.9	135
	4	79.4	119	61.4	92.2	40.7	61.2	90.6	136	79.4	119
	5	72.2	109	56.0	84.2	38.1	57.2	76.5	115	67.8	102
	6	64.3	96.6	50.1	75.3	35.0	52.6	62.2	93.5	55.8	83.9
	7	56.1	84.3	43.9	66.0	30.9	46.5	48.7	73.2	44.3	66.7
	8	47.9	71.9	37.8	56.7	26.8	40.3	37.3	56.1	34.2	51.4
	9	40.0	60.1	31.8	47.8	22.8	34.3	29.5	44.3	27.0	40.6
	10	32.7	49.2	26.2	39.3	19.0	28.5	23.9	35.9	21.9	32.9
	11	27.0	40.6	21.6	32.5	15.7	23.6	19.7	29.7	18.1	27.2
	12	22.7	34.1	18.2	27.3	13.2	19.8	16.6	24.9	15.2	22.9
	13	19.4	29.1	15.5	23.3	11.2	16.9				
	14	16.7	25.1	13.4	20.1	9.69	14.6				
	15	14.5	21.9	11.6	17.5	8.44	12.7				
	16	12.8	19.2	10.2	15.4	7.42	11.1				
	17			9.06	13.6	6.57	9.88				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		94.0	141	72.2	108	49.4	74.3	122	184	105	158
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		73.0	110	56.0	84.0	38.4	57.5	95.1	143	81.8	123
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		36.0	54.1	28.0	42.1	19.4	29.1	49.5	74.4	43.2	64.9
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		15.1	22.6	12.4	18.6	8.96	13.5	11.9	18.0	11.8	17.8
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		12.1	18.1	9.46	14.2	6.61	9.94	14.2	21.4	12.6	18.9
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		7.36	11.1	5.81	8.74	3.34	5.02	7.19	10.8	6.41	9.64
Properties											
Area, in.^{2}		3.14		2.41		1.65		4.09		3.52	
r_y, in.		0.999		1.02		1.05		0.748		0.772	
r_x/r_y		1.73		1.74		1.71		2.13		2.11	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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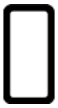

HSS5–HSS4

Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS5x2x						HSS4x3x			
		¼		3⁄16		⅛[a, c]		⅜		5⁄16	
t_{des}, in.		0.233		0.174		0.116		0.349		0.291	
lb/ft		10.5		8.15		5.61		14.7		12.7	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	87.1	131	67.1	101	42.6	64.1	122	184	105	158
	1	85.7	129	66.0	99.2	42.1	63.3	121	182	105	157
	2	81.5	123	63.0	94.7	40.7	61.2	118	178	102	153
	3	75.1	113	58.3	87.6	38.5	57.9	113	170	97.9	147
	4	66.8	100	52.3	78.6	35.6	53.5	107	161	92.4	139
	5	57.6	86.5	45.5	68.3	32.0	48.1	98.9	149	85.8	129
	6	48.0	72.1	38.3	57.6	27.2	40.9	90.0	135	78.3	118
	7	38.7	58.1	31.3	47.1	22.5	33.8	80.6	121	70.4	106
	8	30.1	45.3	24.7	37.2	18.1	27.2	70.9	107	62.2	93.4
	9	23.8	35.8	19.6	29.4	14.3	21.4	61.3	92.1	54.0	81.2
	10	19.3	29.0	15.8	23.8	11.6	17.4	52.1	78.3	46.2	69.4
	11	15.9	24.0	13.1	19.7	9.55	14.4	43.5	65.3	38.8	58.3
	12	13.4	20.1	11.0	16.5	8.03	12.1	36.5	54.9	32.6	49.0
	13	11.4	17.2	9.37	14.1	6.84	10.3	31.1	46.8	27.8	41.7
	14					5.90	8.86	26.8	40.3	23.9	36.0
	15							23.4	35.1	20.9	31.3
	16							20.5	30.9	18.3	27.5
	17							18.2	27.4	16.2	24.4
	18							16.2	24.4	14.5	21.8
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		87.1	131	67.1	101	46.1	69.3	122	184	105	158
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		67.7	101	52.1	78.1	35.8	53.7	95.1	143	81.8	123
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		36.0	54.1	28.0	42.1	19.4	29.1	37.0	55.6	32.7	49.2
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		10.9	16.4	9.25	13.9	6.88	10.3	24.5	36.7	22.3	33.5
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		10.7	16.0	8.41	12.6	5.91	8.89	12.8	19.2	11.3	16.9
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		5.49	8.25	4.37	6.56	2.51	3.78	10.4	15.7	9.21	13.8
Properties											
Area, in.^{2}		2.91		2.24		1.54		4.09		3.52	
r_y, in.		0.797		0.823		0.848		1.11		1.13	
r_x/r_y		2.10		2.07		2.05		1.25		1.26	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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HSS4

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS4x3x						HSS4x2½x			
		¼		3⁄16		⅛[a]		⅜		5⁄16	
t_{des}, in.		0.233		0.174		0.116		0.349		0.291	
lb/ft		10.5		8.15		5.61		13.4		11.6	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	87.1	131	67.1	101	46.1	69.3	112	168	96.7	145
	1	86.4	130	66.6	100	45.8	68.8	111	166	95.6	144
	2	84.4	127	65.1	97.8	44.8	67.3	107	160	92.3	139
	3	81.2	122	62.7	94.3	43.2	65.0	100	151	87.0	131
	4	76.9	116	59.5	89.5	41.1	61.8	91.8	138	80.1	120
	5	71.6	108	55.7	83.7	38.5	57.9	82.2	123	72.1	108
	6	65.7	98.8	51.3	77.1	35.6	53.5	71.7	108	63.4	95.2
	7	59.4	89.2	46.6	70.0	32.4	48.7	61.0	91.7	54.4	81.8
	8	52.8	79.4	41.7	62.6	29.1	43.7	50.7	76.2	45.6	68.6
	9	46.2	69.5	36.7	55.2	25.8	38.7	41.0	61.6	37.3	56.1
	10	39.8	59.9	31.9	47.9	22.5	33.8	33.2	49.9	30.2	45.4
	11	33.8	50.8	27.3	41.0	19.3	29.0	27.4	41.2	25.0	37.6
	12	28.4	42.7	23.0	34.6	16.3	24.6	23.0	34.6	21.0	31.6
	13	24.2	36.3	19.6	29.4	13.9	20.9	19.6	29.5	17.9	26.9
	14	20.9	31.3	16.9	25.4	12.0	18.0	16.9	25.4	15.4	23.2
	15	18.2	27.3	14.7	22.1	10.5	15.7	14.7	22.2	13.4	20.2
	16	16.0	24.0	12.9	19.4	9.19	13.8				
	17	14.1	21.3	11.5	17.2	8.14	12.2				
	18	12.6	19.0	10.2	15.4	7.26	10.9				
	19	11.3	17.0	9.17	13.8	6.52	9.80				
	20					5.88	8.84				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		87.1	131	67.1	101	46.1	69.3	112	168	96.7	145
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		67.7	101	52.1	78.1	35.8	53.7	87.0	130	75.1	113
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		27.6	41.5	21.8	32.7	15.2	22.9	37.0	55.6	32.7	49.2
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		19.3	28.9	15.5	23.3	11.0	16.6	18.2	27.3	17.0	25.6
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		9.51	14.3	7.49	11.3	5.26	7.91	11.2	16.8	9.91	14.9
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		7.78	11.7	6.14	9.23	3.95	5.94	7.98	12.0	7.11	10.7
Properties											
Area, in.^{2}		2.91		2.24		1.54		3.74		3.23	
r_y, in.		1.16		1.19		1.21		0.922		0.947	
r_x/r_y		1.25		1.25		1.26		1.46		1.46	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS4

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS4x2½x						HSS4x2x			
		¼		3/16		1/8[a]		3/8		5/16	
t_{des}, in.		0.233		0.174		0.116		0.349		0.291	
lb/ft		9.66		7.51		5.18		12.2		10.6	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	79.9	120	61.7	92.7	42.5	63.9	101	153	88.0	132
	1	79.1	119	61.0	91.7	42.1	63.3	99.5	150	86.4	130
	2	76.5	115	59.1	88.9	40.9	61.4	93.8	141	81.7	123
	3	72.3	109	56.1	84.3	38.9	58.4	84.9	128	74.5	112
	4	66.9	101	52.1	78.3	36.3	54.5	73.9	111	65.5	98.4
	5	60.5	91.0	47.4	71.2	33.2	49.9	61.9	93.0	55.4	83.3
	6	53.6	80.5	42.2	63.4	29.7	44.7	49.7	74.8	45.2	67.9
	7	46.4	69.7	36.8	55.3	26.1	39.3	38.4	57.7	35.5	53.4
	8	39.2	59.0	31.4	47.2	22.5	33.9	29.4	44.2	27.3	41.0
	9	32.5	48.8	26.2	39.4	19.0	28.6	23.2	34.9	21.5	32.4
	10	26.4	39.7	21.5	32.3	15.7	23.6	18.8	28.3	17.4	26.2
	11	21.8	32.8	17.7	26.7	13.0	19.5	15.5	23.4	14.4	21.7
	12	18.3	27.5	14.9	22.4	10.9	16.4	13.1	19.6	12.1	18.2
	13	15.6	23.5	12.7	19.1	9.30	14.0				
	14	13.5	20.2	10.9	16.5	8.02	12.1				
	15	11.7	17.6	9.54	14.3	6.99	10.5				
	16	10.3	15.5	8.38	12.6	6.14	9.23				
	17					5.44	8.18				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		79.9	120	61.7	92.7	42.5	63.9	101	153	88.0	132
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		62.1	93.1	47.9	71.8	33.0	49.5	78.8	118	68.4	103
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		27.6	41.5	21.8	32.7	15.2	22.9	37.0	55.6	32.7	49.2
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		15.1	22.6	12.4	18.6	8.96	13.5	11.9	18.0	11.8	17.8
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		8.43	12.7	6.66	10.0	4.69	7.05	9.58	14.4	8.56	12.9
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		6.06	9.11	4.82	7.24	3.11	4.67	5.76	8.66	5.19	7.80
Properties											
Area, in.^{2}		2.67		2.06		1.42		3.39		2.94	
r_y, in.		0.973		0.999		1.03		0.729		0.754	
r_x/r_y		1.45		1.44		1.43		1.77		1.75	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS4–HSS3½

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS4x2x						HSS3½x2½x			
		¼		3⁄16		⅛[a]		⅜		5⁄16	
t_{des}, in.		0.233		0.174		0.116		0.349		0.291	
lb/ft		8.81		6.87		4.75		12.2		10.6	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	73.1	110	56.6	85.0	38.9	58.5	101	153	88.0	132
	1	71.8	108	55.7	83.7	38.3	57.6	100	151	87.0	131
	2	68.2	102	53.0	79.7	36.6	55.0	96.4	145	83.8	126
	3	62.5	93.9	48.9	73.5	33.9	51.0	90.4	136	78.9	119
	4	55.3	83.2	43.6	65.5	30.5	45.8	82.6	124	72.4	109
	5	47.3	71.2	37.7	56.6	26.6	39.9	73.5	111	64.9	97.6
	6	39.1	58.8	31.5	47.3	22.5	33.7	63.8	95.9	56.8	85.3
	7	31.2	46.9	25.5	38.3	18.4	27.7	54.0	81.1	48.5	72.8
	8	24.1	36.3	19.9	29.9	14.6	22.0	44.5	66.8	40.4	60.7
	9	19.1	28.7	15.7	23.7	11.5	17.3	35.7	53.6	32.7	49.2
	10	15.5	23.2	12.8	19.2	9.35	14.1	28.9	43.4	26.5	39.9
	11	12.8	19.2	10.5	15.8	7.73	11.6	23.9	35.9	21.9	32.9
	12	10.7	16.1	8.86	13.3	6.49	9.76	20.1	30.2	18.4	27.7
	13	9.15	13.7	7.55	11.3	5.53	8.31	17.1	25.7	15.7	23.6
	14							14.7	22.2	13.5	20.3
	15							12.8	19.3	11.8	17.7
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		73.1	110	56.6	85.1	38.9	58.5	101	153	88.0	132
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		56.7	85.1	43.9	65.9	30.2	45.3	78.8	118	68.4	103
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		27.6	41.5	21.8	32.7	15.2	22.9	30.7	46.2	27.5	41.3
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		10.9	16.4	9.25	13.9	6.88	10.3	18.2	27.3	17.0	25.6
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		7.34	11.0	5.84	8.78	4.14	6.23	8.96	13.5	7.99	12.0
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		4.47	6.71	3.57	5.36	2.34	3.52	7.04	10.6	6.30	9.47
Properties											
Area, in.^{2}		2.44		1.89		1.30		3.39		2.94	
r_y, in.		0.779		0.804		0.830		0.904		0.930	
r_x/r_y		1.75		1.73		1.72		1.31		1.31	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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HSS3½

Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS3½x2½x						HSS3½x2x			
		¼		3⁄16		⅛[a]		¼		3⁄16	
t_{des}, in.		0.233		0.174		0.116		0.233		0.174	
lb/ft		8.81		6.87		4.75		7.96		6.23	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	73.1	110	56.6	85.0	38.9	58.5	66.2	99.4	51.2	76.9
	1	72.2	109	56.0	84.1	38.5	57.9	65.0	97.7	50.3	75.7
	2	69.8	105	54.2	81.4	37.3	56.1	61.6	92.6	47.9	72.0
	3	65.9	99.0	51.3	77.1	35.5	53.3	56.3	84.6	44.0	66.2
	4	60.8	91.3	47.5	71.4	33.0	49.6	49.7	74.6	39.1	58.8
	5	54.8	82.3	43.1	64.8	30.1	45.2	42.2	63.5	33.7	50.6
	6	48.3	72.5	38.2	57.4	26.8	40.3	34.7	52.1	28.0	42.1
	7	41.5	62.4	33.2	49.9	23.5	35.3	27.5	41.3	22.5	33.8
	8	35.0	52.5	28.2	42.3	20.1	30.2	21.1	31.8	17.5	26.3
	9	28.7	43.2	23.4	35.2	16.9	25.3	16.7	25.1	13.8	20.8
	10	23.3	35.0	19.1	28.6	13.8	20.8	13.5	20.3	11.2	16.8
	11	19.2	28.9	15.8	23.7	11.4	17.2	11.2	16.8	9.25	13.9
	12	16.2	24.3	13.2	19.9	9.60	14.4	9.40	14.1	7.78	11.7
	13	13.8	20.7	11.3	17.0	8.18	12.3			6.62	9.96
	14	11.9	17.9	9.72	14.6	7.05	10.6				
	15	10.3	15.5	8.47	12.7	6.14	9.24				
	16			7.44	11.2	5.40	8.12				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		73.1	110	56.6	85.1	38.9	58.5	66.2	99.5	51.2	77.0
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		56.7	85.1	43.9	65.9	30.2	45.3	51.4	77.1	39.8	59.6
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		23.4	35.2	18.6	28.0	13.1	19.7	23.4	35.2	18.6	28.0
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		15.1	22.6	12.4	18.6	8.96	13.5	10.9	16.4	9.25	13.9
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		6.83	10.3	5.43	8.16	3.84	5.78	5.89	8.85	4.72	7.09
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		5.39	8.11	4.30	6.47	3.04	4.57	3.94	5.93	3.17	4.76
Properties											
Area, in.^{2}		2.44		1.89		1.30		2.21		1.71	
r_y, in.		0.956		0.983		1.01		0.766		0.792	
r_x/r_y		1.30		1.30		1.30		1.57		1.55	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS3½–HSS3

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS3½x2x		HSS3½x1½x						HSS3x2½x	
		⅛[a]		¼		3⁄16		⅛[a]		5⁄16	
t_{des}, in.		0.116		0.233		0.174		0.116		0.291	
lb/ft		4.33		7.11		5.59		3.90		9.51	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	35.6	53.5	59.0	88.6	46.1	69.3	32.0	48.1	79.0	119
	1	35.1	52.7	57.1	85.8	44.8	67.3	31.2	46.8	78.0	117
	2	33.5	50.3	51.8	77.8	40.9	61.5	28.7	43.1	75.1	113
	3	30.9	46.5	44.0	66.2	35.2	53.0	25.0	37.6	70.5	106
	4	27.7	41.6	35.1	52.7	28.6	43.0	20.6	31.0	64.4	96.8
	5	24.0	36.1	26.2	39.3	21.9	32.9	16.1	24.2	57.4	86.3
	6	20.2	30.4	18.5	27.8	15.8	23.7	11.9	17.9	49.9	75.0
	7	16.5	24.8	13.6	20.4	11.6	17.4	8.73	13.1	42.3	63.5
	8	13.0	19.5	10.4	15.6	8.86	13.3	6.69	10.0	34.9	52.5
	9	10.3	15.4	8.22	12.4	7.00	10.5	5.28	7.94	28.0	42.2
	10	8.31	12.5					4.28	6.43	22.7	34.1
	11	6.87	10.3							18.8	28.2
	12	5.77	8.67							15.8	23.7
	13	4.92	7.39							13.4	20.2
	14									11.6	17.4
	15									10.1	15.2
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		35.6	53.6	59.0	88.7	46.1	69.3	32.0	48.2	79.0	119
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		27.7	41.5	45.8	68.7	35.8	53.7	24.9	37.3	61.4	92.1
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		13.1	19.7	23.4	35.2	18.6	28.0	13.1	19.7	22.3	33.5
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		6.88	10.3	6.71	10.1	6.11	9.19	4.79	7.20	17.0	25.6
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		3.34	5.03	4.94	7.43	3.99	6.00	2.87	4.31	6.26	9.41
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		2.27	3.41	2.64	3.98	2.16	3.25	1.57	2.35	5.49	8.25
Properties											
Area, in.^{2}		1.19		1.97		1.54		1.07		2.64	
r_y, in.		0.818		0.569		0.594		0.619		0.908	
r_x/r_y		1.55		2.00		1.97		1.95		1.16	

[a] Shape exceeds the compact limit for flexure about the X-X axis for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

HSS3

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS3x2½x						HSS3x2x			
		¼		3⁄16		⅛		5⁄16		¼	
t_{des}, in.		0.233		0.174		0.116		0.291		0.233	
lb/ft		7.96		6.23		4.33		8.45		7.11	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	66.2	99.4	51.2	76.9	35.6	53.5	70.4	106	59.0	88.6
	1	65.4	98.3	50.6	76.1	35.2	53.0	69.0	104	57.9	87.0
	2	63.1	94.8	48.9	73.5	34.1	51.3	64.9	97.6	54.7	82.3
	3	59.4	89.2	46.2	69.5	32.3	48.6	58.8	88.3	49.9	74.9
	4	54.6	82.0	42.7	64.2	30.0	45.1	51.1	76.8	43.8	65.8
	5	49.0	73.6	38.5	57.9	27.2	40.9	42.6	64.1	37.0	55.6
	6	42.9	64.5	34.0	51.1	24.2	36.4	34.2	51.4	30.1	45.3
	7	36.7	55.1	29.4	44.1	21.0	31.6	26.3	39.5	23.6	35.5
	8	30.6	46.0	24.8	37.2	17.9	26.9	20.1	30.3	18.1	27.2
	9	24.9	37.4	20.4	30.7	14.9	22.4	15.9	23.9	14.3	21.5
	10	20.2	30.3	16.6	24.9	12.2	18.3	12.9	19.4	11.6	17.4
	11	16.7	25.0	13.7	20.6	10.1	15.1	10.7	16.0	9.58	14.4
	12	14.0	21.0	11.5	17.3	8.45	12.7	8.95	13.5	8.05	12.1
	13	11.9	17.9	9.79	14.7	7.20	10.8				
	14	10.3	15.5	8.45	12.7	6.21	9.34				
	15	8.96	13.5	7.36	11.1	5.41	8.13				
	16			6.47	9.72	4.76	7.15				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		66.2	99.5	51.2	77.0	35.6	53.6	70.4	106	59.0	88.7
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		51.4	77.1	39.8	59.6	27.7	41.5	54.6	82.0	45.8	68.7
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		19.3	28.9	15.5	23.3	11.0	16.6	22.3	33.5	19.3	28.9
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		15.1	22.6	12.4	18.6	8.96	13.5	11.8	17.8	10.9	16.4
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		5.39	8.10	4.32	6.49	3.07	4.61	5.26	7.91	4.57	6.86
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		4.74	7.13	3.79	5.70	2.72	4.09	3.94	5.93	3.44	5.18
Properties											
Area, in.^{2}		2.21		1.71		1.19		2.35		1.97	
r_y, in.		0.935		0.963		0.990		0.725		0.751	
r_x/r_y		1.16		1.15		1.15		1.39		1.38	

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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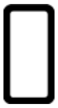

HSS3

Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

A500 Gr. C

F_y = 50 ksi

F_u = 62 ksi

Shape		HSS3x2x				HSS3x1½x					
		3⁄16		⅛		¼		3⁄16		⅛	
t_{des}, in.		0.174		0.116		0.233		0.174		0.116	
lb/ft		5.59		3.90		6.26		4.96		3.48	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	46.1	69.3	32.0	48.1	52.1	78.3	41.0	61.6	28.6	43.0
	1	45.3	68.1	31.5	47.4	50.4	75.7	39.8	59.8	27.8	41.8
	2	43.0	64.6	30.0	45.1	45.5	68.4	36.3	54.5	25.6	38.4
	3	39.4	59.3	27.7	41.6	38.5	57.8	31.1	46.7	22.2	33.3
	4	34.9	52.5	24.7	37.1	30.4	45.7	25.0	37.6	18.2	27.4
	5	29.8	44.9	21.3	32.0	22.4	33.7	19.0	28.5	14.1	21.2
	6	24.6	37.0	17.8	26.8	15.8	23.7	13.5	20.4	10.3	15.5
	7	19.7	29.6	14.4	21.7	11.6	17.4	9.95	15.0	7.58	11.4
	8	15.2	22.8	11.3	17.0	8.87	13.3	7.62	11.5	5.80	8.72
	9	12.0	18.1	8.91	13.4	7.01	10.5	6.02	9.05	4.58	6.89
	10	9.73	14.6	7.22	10.9					3.71	5.58
	11	8.04	12.1	5.97	8.97						
	12	6.76	10.2	5.01	7.54						
	13			4.27	6.42						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		46.1	69.3	32.0	48.2	52.1	78.3	41.0	61.7	28.6	43.0
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		35.8	53.7	24.9	37.3	40.5	60.7	31.9	47.8	22.2	33.3
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		15.5	23.3	11.0	16.6	19.3	28.9	15.5	23.3	11.0	16.6
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		9.25	13.9	6.88	10.3	6.71	10.1	6.11	9.19	4.79	7.20
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		3.69	5.55	2.64	3.98	3.77	5.66	3.09	4.65	2.23	3.36
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		2.79	4.20	2.00	3.01	2.27	3.42	1.88	2.82	1.37	2.06
Properties											
Area, in.^{2}		1.54		1.07		1.74		1.37		0.956	
r_y, in.		0.778		0.804		0.559		0.584		0.610	
r_x/r_y		1.38		1.37		1.76		1.75		1.72	

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

HSS3–HSS2½

Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS3x1x				HSS2½x2x					
		3⁄16		⅛		¼		3⁄16		⅛	
t_{des}, in.		0.174		0.116		0.233		0.174		0.116	
lb/ft		4.32		3.05		6.26		4.96		3.48	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	35.6	53.5	25.1	37.8	52.1	78.3	41.0	61.6	28.6	43.0
	1	33.1	49.8	23.6	35.4	51.1	76.8	40.3	60.5	28.1	42.3
	2	26.6	40.0	19.5	29.2	48.1	72.4	38.1	57.3	26.7	40.2
	3	18.5	27.8	14.1	21.2	43.6	65.6	34.8	52.3	24.5	36.9
	4	11.2	16.8	8.99	13.5	38.0	57.1	30.6	46.0	21.8	32.7
	5	7.17	10.8	5.75	8.65	31.8	47.8	25.9	39.0	18.7	28.1
	6	4.98	7.49	3.99	6.00	25.6	38.5	21.2	31.9	15.5	23.3
	7					19.8	29.8	16.7	25.1	12.4	18.6
	8					15.2	22.8	12.8	19.3	9.61	14.4
	9					12.0	18.0	10.1	15.2	7.59	11.4
	10					9.71	14.6	8.22	12.3	6.15	9.24
	11					8.02	12.1	6.79	10.2	5.08	7.64
	12					6.74	10.1	5.71	8.58	4.27	6.42
	13									3.64	5.47
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		35.6	53.6	25.1	37.8	52.1	78.3	41.0	61.7	28.6	43.0
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		27.7	41.5	19.5	29.3	40.5	60.7	31.9	47.8	22.2	33.3
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		15.5	23.3	11.0	16.6	15.1	22.6	12.4	18.6	8.96	13.5
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		2.99	4.49	2.72	4.08	10.9	16.4	9.25	13.9	6.88	10.3
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		2.47	3.71	1.82	2.73	3.42	5.14	2.79	4.20	2.02	3.03
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		1.08	1.62	0.811	1.22	2.92	4.39	2.39	3.59	1.73	2.60
Properties											
Area, in.^{2}		1.19		0.840		1.74		1.37		0.956	
r_y, in.		0.380		0.405		0.731		0.758		0.785	
r_x/r_y		2.49		2.44		1.20		1.19		1.19	

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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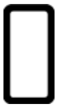

HSS2½

Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

A500 Gr. C

F_y = 50 ksi

F_u = 62 ksi

Shape		HSS2½x1½x						HSS2½x1x			
		¼		3⁄16		⅛		3⁄16		⅛	
t_{des}, in.		0.233		0.174		0.116		0.174		0.116	
lb/ft		5.41		4.32		3.05		3.68		2.63	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	45.2	67.9	35.6	53.5	25.1	37.8	30.5	45.9	21.7	32.6
	1	43.6	65.6	34.5	51.9	24.4	36.7	28.3	42.6	20.3	30.5
	2	39.3	59.0	31.3	47.1	22.3	33.6	22.6	34.0	16.6	25.0
	3	32.9	49.4	26.7	40.1	19.3	29.0	15.5	23.3	12.0	18.0
	4	25.7	38.6	21.3	32.0	15.7	23.6	9.31	14.0	7.52	11.3
	5	18.7	28.1	15.9	24.0	12.0	18.1	5.96	8.95	4.81	7.23
	6	13.1	19.6	11.3	17.0	8.68	13.0	4.14	6.22	3.34	5.02
	7	9.59	14.4	8.29	12.5	6.38	9.59				
	8	7.34	11.0	6.35	9.54	4.88	7.34				
	9	5.80	8.72	5.02	7.54	3.86	5.80				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		45.2	68.0	35.6	53.6	25.1	37.8	30.5	45.9	21.7	32.6
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		35.1	52.7	27.7	41.5	19.5	29.3	23.7	35.6	16.8	25.2
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		15.1	22.6	12.4	18.6	8.96	13.5	12.4	18.6	8.96	13.5
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		6.71	10.1	6.11	9.19	4.79	7.20	2.99	4.49	2.72	4.08
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		2.77	4.16	2.28	3.43	1.67	2.52	1.78	2.67	1.33	2.00
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		1.91	2.87	1.59	2.39	1.17	1.76	0.898	1.35	0.684	1.03
Properties											
Area, in.^{2}		1.51		1.19		0.840		1.02		0.724	
r_y, in.		0.546		0.572		0.597		0.374		0.399	
r_x/r_y		1.51		1.50		1.49		2.13		2.09	

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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Table IV-7B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Rectangular HSS

HSS2¼–HSS2

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS2¼x2x				HSS2x1½x				HSS2x1x	
		3⁄16		⅛		3⁄16		⅛		3⁄16	
t_{des}, in.		0.174		0.116		0.174		0.116		0.174	
lb/ft		4.64		3.27		3.68		2.63		3.04	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	38.3	57.6	26.9	40.4	30.5	45.9	21.7	32.6	25.3	38.0
	1	37.6	56.5	26.4	39.7	29.5	44.4	21.0	31.6	23.4	35.1
	2	35.5	53.4	25.1	37.7	26.6	40.0	19.1	28.8	18.4	27.7
	3	32.3	48.6	23.0	34.5	22.4	33.7	16.4	24.6	12.4	18.7
	4	28.3	42.6	20.3	30.5	17.6	26.5	13.2	19.8	7.34	11.0
	5	23.9	35.9	17.3	26.0	13.0	19.5	9.94	14.9	4.70	7.06
	6	19.4	29.2	14.3	21.5	9.08	13.6	7.09	10.7	3.26	4.91
	7	15.2	22.9	11.4	17.1	6.67	10.0	5.21	7.82		
	8	11.6	17.5	8.77	13.2	5.11	7.67	3.99	5.99		
	9	9.20	13.8	6.93	10.4	4.03	6.06	3.15	4.73		
	10	7.46	11.2	5.62	8.44						
	11	6.16	9.26	4.64	6.98						
	12	5.18	7.78	3.90	5.86						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		38.3	57.6	26.9	40.4	30.5	45.9	21.7	32.6	25.3	38.0
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		29.8	44.6	20.9	31.3	23.7	35.6	16.8	25.2	19.6	29.5
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		10.8	16.3	7.92	11.9	9.25	13.9	6.88	10.3	9.25	13.9
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		9.25	13.9	6.88	10.3	6.11	9.19	4.79	7.20	2.99	4.49
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		2.38	3.57	1.73	2.60	1.59	2.40	1.19	1.78	1.20	1.80
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
		2.19	3.29	1.59	2.40	1.30	1.95	0.971	1.46	0.719	1.08
Properties											
Area, in.^{2}		1.28		0.898		1.02		0.724		0.845	
r_y, in.		0.747		0.774		0.554		0.581		0.365	
r_x/r_y		1.10		1.10		1.26		1.25		1.76	

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

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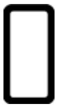

HSS2

Table IV-7B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Rectangular HSS

A500 Gr. C

F_y = 50 ksi

F_u = 62 ksi

Shape		HSS2x1x ⅛	
t_{des}, in.		0.116	
lb/ft		2.20	
Design		ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	18.2	27.4
	1	17.0	25.5
	2	13.8	20.7
	3	9.76	14.7
	4	6.03	9.07
	5	3.86	5.80
	6	2.68	4.03
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$
		18.2	27.4
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$
		14.1	21.2
Available Strength in Shear about X-X Axis, kips		V_n/Ω_v	$\phi_v V_n$
		6.88	10.3
Available Strength in Shear about Y-Y Axis, kips		V_n/Ω_v	$\phi_v V_n$
		2.72	4.08
Available Strength in Flexure about X-X Axis, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$
		0.913	1.37
Available Strength in Flexure about Y-Y Axis, kip-ft		M_{ny}/Ω_b	$\phi_b M_{ny}$
		0.556	0.836

Properties

Area, in.^{2}	0.608
r_y, in.	0.390
r_x/r_y	1.74

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Per AISC *Specification* Section F7.4, lateral-torsional buckling should be considered for bending about the X-X axis.

Table IV-8A

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Square HSS

HSS22–HSS20

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape		HSS22x22x				HSS20x20x					
		⅞		¾		⅞		¾		⅝^f	
t_{des}, in.		0.875		0.750		0.875		0.750		0.625	
lb/ft		245		212		221		192		161	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	2160	3240	1870	2800	1950	2920	1690	2530	1420	2130
	1	2160	3240	1870	2800	1950	2920	1690	2530	1420	2130
	2	2150	3240	1860	2800	1940	2920	1680	2530	1420	2130
	3	2150	3240	1860	2800	1940	2920	1680	2530	1420	2130
	4	2150	3230	1860	2800	1940	2920	1680	2530	1420	2130
	5	2150	3230	1860	2790	1940	2910	1680	2520	1410	2120
	6	2140	3220	1860	2790	1930	2910	1680	2520	1410	2120
	7	2140	3220	1850	2780	1930	2900	1670	2510	1410	2120
	8	2140	3210	1850	2780	1920	2890	1670	2510	1400	2110
	9	2130	3200	1840	2770	1920	2880	1660	2500	1400	2100
	10	2120	3190	1840	2760	1910	2870	1660	2490	1400	2100
	11	2120	3180	1830	2760	1910	2860	1650	2480	1390	2090
	12	2110	3170	1830	2750	1900	2850	1640	2470	1380	2080
	13	2100	3160	1820	2740	1890	2840	1640	2460	1380	2070
	14	2100	3150	1810	2730	1880	2830	1630	2450	1370	2060
	15	2090	3140	1810	2720	1870	2810	1620	2440	1370	2050
	16	2080	3120	1800	2700	1860	2800	1610	2420	1360	2040
	17	2070	3110	1790	2690	1850	2780	1600	2410	1350	2030
	18	2060	3090	1780	2680	1840	2760	1590	2400	1340	2020
	19	2050	3080	1770	2660	1830	2750	1580	2380	1330	2010
	20	2040	3060	1760	2650	1810	2730	1570	2360	1330	1990
	22	2010	3020	1740	2620	1790	2690	1550	2330	1310	1960
	24	1980	2980	1720	2580	1760	2640	1530	2290	1290	1930
	26	1960	2940	1690	2550	1730	2600	1500	2250	1270	1900
	28	1930	2890	1670	2510	1700	2550	1470	2210	1240	1870
	30	1890	2850	1640	2470	1660	2500	1440	2170	1220	1830
	32	1860	2800	1610	2420	1630	2440	1410	2120	1190	1790
	34	1830	2740	1580	2380	1590	2390	1380	2080	1170	1750
	36	1790	2690	1550	2330	1550	2330	1350	2030	1140	1710
	38	1750	2630	1520	2280	1510	2270	1310	1970	1110	1670
	40	1710	2570	1490	2230	1470	2210	1280	1920	1080	1630
	42	1670	2510	1450	2180	1430	2150	1240	1870	1050	1580
	44	1630	2450	1420	2130	1390	2080	1210	1810	1020	1540
	46	1590	2390	1380	2080	1340	2020	1170	1760	991	1490
	48	1550	2330	1350	2020	1300	1950	1130	1700	960	1440
	50	1510	2260	1310	1970	1260	1890	1090	1650	929	1400
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		2160	3240	1870	2800	1950	2930	1690	2530	1420	2130
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		1760	2630	1520	2280	1590	2380	1370	2060	1160	1740
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		610	917	534	802	547	822	480	721	406	611
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		1410	2120	1230	1850	1150	1730	1010	1510	828	1240
Properties											
Area, in.^{2}		72.0		62.3		65.0		56.3		47.4	
r_x = r_y, in.		8.56		8.62		7.75		7.81		7.88	
I_x = I_y, in.^{4}		5280		4630		3900		3430		2940	

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Note: Confirm ASTM A1085 material availability before specifying.

Table IV-8A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

HSS20–HSS18

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS20x20x		HSS18x18x							
		½$^{c, f}$		⅞		¾		⅝		½^{f}	
t_{des}, in.		0.500		0.875		0.750		0.625		0.500	
lb/ft		131		197		171		144		117	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	1100	1650	1740	2610	1510	2260	1270	1910	1030	1550
	1	1100	1650	1740	2610	1510	2260	1270	1910	1030	1550
	2	1100	1650	1740	2610	1500	2260	1270	1910	1030	1550
	3	1100	1650	1730	2600	1500	2260	1270	1900	1030	1550
	4	1090	1650	1730	2600	1500	2260	1270	1900	1030	1540
	5	1090	1640	1730	2600	1500	2250	1260	1900	1020	1540
	6	1090	1640	1720	2590	1490	2250	1260	1890	1020	1540
	7	1090	1640	1720	2580	1490	2240	1260	1890	1020	1530
	8	1090	1640	1710	2570	1490	2230	1250	1880	1020	1530
	9	1090	1630	1710	2560	1480	2220	1250	1880	1010	1520
	10	1080	1630	1700	2550	1470	2220	1240	1870	1010	1520
	11	1080	1630	1690	2540	1470	2210	1240	1860	1000	1510
	12	1080	1620	1680	2530	1460	2190	1230	1850	1000	1500
	13	1080	1620	1670	2510	1450	2180	1220	1840	994	1490
	14	1070	1610	1660	2500	1440	2170	1220	1830	989	1490
	15	1070	1610	1650	2480	1430	2160	1210	1820	983	1480
	16	1070	1600	1640	2470	1430	2140	1200	1810	976	1470
	17	1060	1590	1630	2450	1420	2130	1190	1790	970	1460
	18	1060	1590	1620	2430	1400	2110	1190	1780	963	1450
	19	1050	1580	1600	2410	1390	2090	1180	1770	955	1440
	20	1050	1570	1590	2390	1380	2080	1170	1750	948	1420
	22	1040	1560	1560	2350	1360	2040	1150	1720	931	1400
	24	1030	1540	1530	2300	1330	2000	1120	1690	914	1370
	26	1010	1520	1500	2250	1300	1960	1100	1650	895	1340
	28	1000	1510	1460	2200	1270	1910	1080	1620	875	1310
	30	988	1480	1420	2140	1240	1860	1050	1580	854	1280
	32	968	1460	1390	2080	1210	1820	1020	1540	832	1250
	34	947	1420	1350	2020	1170	1760	994	1490	810	1220
	36	925	1390	1310	1960	1140	1710	965	1450	786	1180
	38	902	1360	1260	1900	1100	1660	935	1410	762	1150
	40	879	1320	1220	1840	1070	1600	905	1360	738	1110
	42	855	1290	1180	1770	1030	1550	874	1310	713	1070
	44	831	1250	1130	1710	992	1490	842	1270	688	1030
	46	806	1210	1090	1640	955	1430	811	1220	663	996
	48	781	1170	1050	1570	917	1380	779	1170	637	958
	50	756	1140	1000	1510	879	1320	748	1120	612	920
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		1150	1730	1740	2610	1510	2260	1270	1910	1030	1550
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		936	1400	1410	2120	1230	1840	1030	1550	839	1260
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		332	500	484	728	426	640	362	543	296	446
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		578	869	918	1380	803	1210	684	1030	497	747
Properties											
Area, in.^{2}		38.4		58.0		50.3		42.4		34.4	
$r_x = r_y$, in.		7.92		6.92		6.99		7.05		7.11	
$I_x = I_y$, in.^{4}		2410		2780		2460		2110		1740	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Note: Confirm ASTM A1085 material availability before specifying.

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Table IV-8A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

HSS16

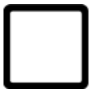
A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS16x16x									
		⅞		¾		⅝		½[f]		⅜[c, f]	
t_{des}, in.		0.875		0.750		0.625		0.500		0.375	
lb/ft		173		151		127		103		78.5	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	1530	2290	1330	1990	1120	1680	910	1370	626	940
	1	1530	2290	1330	1990	1120	1680	910	1370	625	940
	2	1530	2290	1320	1990	1120	1680	909	1370	625	940
	3	1520	2290	1320	1990	1120	1680	908	1360	625	939
	4	1520	2280	1320	1980	1110	1680	906	1360	624	938
	5	1520	2280	1320	1980	1110	1670	904	1360	623	936
	6	1510	2270	1310	1970	1110	1670	901	1350	622	934
	7	1510	2260	1310	1970	1100	1660	898	1350	620	932
	8	1500	2250	1300	1960	1100	1650	895	1340	619	930
	9	1490	2240	1300	1950	1100	1650	891	1340	617	927
	10	1480	2230	1290	1940	1090	1640	886	1330	615	924
	11	1480	2220	1280	1930	1080	1630	881	1320	613	921
	12	1470	2200	1270	1920	1080	1620	876	1320	610	917
	13	1460	2190	1270	1900	1070	1610	870	1310	607	913
	14	1440	2170	1260	1890	1060	1600	864	1300	605	909
	15	1430	2150	1250	1870	1050	1580	857	1290	601	904
	16	1420	2130	1240	1860	1040	1570	850	1280	598	899
	17	1410	2110	1220	1840	1040	1560	843	1270	595	894
	18	1390	2090	1210	1820	1030	1540	835	1250	591	888
	19	1380	2070	1200	1800	1020	1530	827	1240	587	883
	20	1360	2050	1190	1790	1000	1510	818	1230	583	877
	22	1330	2000	1160	1740	982	1480	800	1200	575	864
	24	1300	1950	1130	1700	958	1440	780	1170	565	850
	26	1260	1900	1100	1650	932	1400	760	1140	555	835
	28	1220	1840	1070	1610	905	1360	738	1110	545	819
	30	1180	1780	1030	1560	877	1320	716	1080	533	802
	32	1140	1720	1000	1500	848	1270	692	1040	522	784
	34	1100	1650	964	1450	818	1230	668	1000	509	765
	36	1060	1590	928	1390	788	1180	644	968	493	741
	38	1010	1530	891	1340	757	1140	619	930	474	713
	40	971	1460	853	1280	725	1090	594	892	455	685
	42	927	1390	816	1230	694	1040	568	854	436	656
	44	883	1330	778	1170	662	995	543	816	417	627
	46	839	1260	740	1110	631	948	517	778	398	598
	48	796	1200	703	1060	599	901	492	740	379	570
	50	753	1130	666	1000	568	854	467	702	360	541
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		1530	2300	1330	1990	1120	1680	910	1370	692	1040
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		1240	1870	1080	1620	913	1370	741	1110	562	843
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		421	633	372	559	317	476	260	392	201	302
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		711	1070	624	938	534	803	424	637	272	409
Properties											
Area, in.^{2}		51.0		44.3		37.4		30.4		23.1	
$r_x = r_y$, in.		6.10		6.18		6.23		6.28		6.35	
$I_x = I_y$, in.^{4}		1900		1690		1450		1200		931	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Note: Confirm ASTM A1085 material availability before specifying.



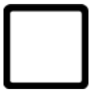
HSS16–HSS14

Table IV-8A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS16x16x		HSS14x14x							
		5⁄16[c, f]		7⁄8		3⁄4		5⁄8		1⁄2	
t_{des}, in.		0.313		0.875		0.750		0.625		0.500	
lb/ft		65.9		150		130		110		89.7	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	458	689	1320	1980	1150	1720	970	1460	790	1190
	1	458	689	1320	1980	1150	1720	970	1460	790	1190
	2	458	688	1320	1980	1150	1720	969	1460	789	1190
	3	458	688	1310	1970	1140	1720	967	1450	788	1180
	4	457	687	1310	1970	1140	1710	965	1450	786	1180
	5	456	686	1310	1960	1140	1710	961	1440	783	1180
	6	456	685	1300	1950	1130	1700	958	1440	780	1170
	7	455	683	1290	1940	1130	1690	953	1430	777	1170
	8	454	682	1290	1930	1120	1680	948	1420	773	1160
	9	452	680	1280	1920	1110	1670	942	1420	768	1150
	10	451	678	1270	1910	1110	1660	936	1410	763	1150
	11	449	675	1260	1890	1100	1650	929	1400	757	1140
	12	448	673	1250	1880	1090	1630	921	1380	751	1130
	13	446	670	1240	1860	1080	1620	913	1370	745	1120
	14	444	667	1220	1840	1070	1600	904	1360	738	1110
	15	442	664	1210	1820	1060	1590	895	1350	730	1100
	16	439	660	1200	1800	1040	1570	885	1330	722	1090
	17	437	657	1180	1780	1030	1550	875	1310	714	1070
	18	435	653	1170	1750	1020	1530	864	1300	705	1060
	19	432	649	1150	1730	1000	1510	852	1280	696	1050
	20	429	645	1130	1700	990	1490	840	1260	687	1030
	22	423	636	1100	1650	960	1440	816	1230	667	1000
	24	417	626	1060	1590	928	1400	789	1190	645	970
	26	410	616	1020	1540	895	1350	761	1140	623	936
	28	402	605	981	1470	860	1290	732	1100	600	902
	30	395	593	939	1410	825	1240	703	1060	576	866
	32	387	581	896	1350	788	1180	672	1010	551	829
	34	378	568	853	1280	751	1130	641	963	526	791
	36	369	555	809	1220	713	1070	610	916	501	753
	38	360	541	765	1150	675	1020	578	869	476	715
	40	350	527	722	1080	638	959	547	822	450	677
	42	341	512	678	1020	601	903	515	775	425	639
	44	331	497	636	956	564	848	485	728	400	601
	46	320	481	594	893	528	794	454	683	375	564
	48	310	466	554	832	493	741	425	638	351	528
	50	299	450	514	773	459	689	396	595	328	493
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		581	873	1320	1980	1150	1720	970	1460	790	1190
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		475	712	1070	1610	933	1400	790	1180	644	965
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		170	255	358	539	318	478	272	408	225	338
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		213	320	531	799	469	705	402	604	329	495
Properties											
Area, in.^{2}		19.4		44.0		38.3		32.4		26.4	
$r_x = r_y$, in.		6.38		5.29		5.36		5.42		5.47	
$I_x = I_y$, in.^{4}		790		1230		1100		952		791	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.
Note: Confirm ASTM A1085 material availability before specifying.



HSS14–HSS12

Table IV-8A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS14x14x				HSS12x12x					
		3⁄8[c, f]		5⁄16[c, f]		3⁄4		5⁄8		1⁄2	
t_{des}, in.		0.375		0.313		0.750		0.625		0.500	
lb/ft		68.3		57.4		110		93.3		76.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	598	898	442	665	967	1450	820	1230	671	1010
	1	598	898	442	665	967	1450	820	1230	670	1010
	2	597	898	442	664	965	1450	819	1230	669	1010
	3	596	896	442	664	963	1450	817	1230	668	1000
	4	595	895	441	663	959	1440	814	1220	665	1000
	5	594	893	440	661	955	1440	810	1220	663	996
	6	593	891	439	660	949	1430	806	1210	659	991
	7	591	888	438	658	943	1420	801	1200	655	984
	8	589	885	436	656	936	1410	795	1190	650	977
	9	585	880	435	653	928	1390	788	1180	645	969
	10	581	874	433	650	919	1380	781	1170	639	960
	11	577	868	431	647	909	1370	772	1160	632	951
	12	573	861	428	644	898	1350	764	1150	625	940
	13	568	853	426	640	887	1330	754	1130	618	929
	14	563	845	423	636	875	1320	744	1120	610	917
	15	557	837	421	632	862	1300	733	1100	601	904
	16	551	828	418	628	849	1280	722	1090	592	890
	17	545	819	415	623	834	1250	710	1070	583	876
	18	538	809	411	618	820	1230	698	1050	573	861
	19	531	799	408	613	804	1210	685	1030	563	846
	20	524	788	404	608	788	1180	672	1010	552	830
	22	509	766	397	596	755	1140	645	969	530	797
	24	494	742	388	584	721	1080	616	926	507	762
	26	477	717	379	570	685	1030	586	881	483	726
	28	459	691	370	556	648	974	555	835	459	689
	30	441	663	360	541	611	918	524	788	434	652
	32	423	636	350	526	573	861	493	741	408	614
	34	404	608	339	510	536	805	462	694	383	575
	36	385	579	325	489	499	750	430	647	358	538
	38	366	550	309	465	462	695	400	601	333	500
	40	347	521	293	441	427	642	370	556	309	464
	42	328	493	277	417	393	590	341	513	285	429
	44	309	464	262	393	359	539	313	470	262	394
	46	290	437	246	370	328	494	286	430	240	361
	48	272	409	231	347	302	453	263	395	220	331
	50	254	382	216	325	278	418	242	364	203	305
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		602	905	506	761	967	1450	820	1230	671	1010
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		491	736	413	619	787	1180	670	1000	546	819
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		174	261	147	221	263	395	227	341	189	284
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		217	326	171	257	334	503	289	435	238	358
Properties											
Area, in.^{2}		20.1		16.9		32.3		27.4		22.4	
$r_x = r_y$, in.		5.53		5.56		4.54		4.60		4.66	
$I_x = I_y$, in.^{4}		615		523		666		580		486	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Note: Confirm ASTM A1085 material availability before specifying.

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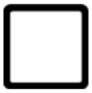

HSS12–HSS10

Table IV-8A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Square HSS

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape		HSS12x12x								HSS10x10x	
		3/8^{f}		5/16$^{c, f}$		1/4$^{c, f}$		3/16$^{c, f}$		3/4	
t_{des}, in.		0.375		0.313		0.250		0.188		0.750	
lb/ft		58.1		48.9		39.4		29.8		89.5	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	512	769	421	632	288	432	174	261	787	1180
	1	512	769	421	632	288	432	173	261	787	1180
	2	511	768	420	632	287	432	173	260	785	1180
	3	510	766	420	631	287	431	173	260	782	1180
	4	508	764	419	629	286	430	173	260	778	1170
	5	506	760	417	627	286	429	172	259	773	1160
	6	503	756	416	625	285	428	172	258	766	1150
	7	500	752	414	622	284	426	171	257	759	1140
	8	497	746	412	619	282	424	170	256	750	1130
	9	493	740	410	616	281	422	170	255	740	1110
	10	488	734	407	612	279	420	169	254	730	1100
	11	483	727	404	608	278	417	168	252	718	1080
	12	478	719	401	603	276	414	167	251	706	1060
	13	473	710	398	598	274	411	166	249	692	1040
	14	466	701	393	591	271	408	164	247	678	1020
	15	460	692	388	583	269	404	163	245	664	997
	16	453	681	382	575	267	401	162	243	648	974
	17	446	671	377	566	264	397	160	241	632	950
	18	439	660	370	557	261	392	159	238	615	925
	19	431	648	364	547	258	388	157	236	598	899
	20	423	636	357	537	255	384	155	233	581	873
	22	407	612	344	517	249	374	152	228	545	819
	24	390	585	329	495	242	363	148	222	508	764
	26	371	558	314	472	235	352	144	216	471	708
	28	353	530	299	449	227	341	139	209	434	652
	30	334	502	283	425	219	329	135	203	397	597
	32	315	473	267	401	210	316	130	195	361	543
	34	296	445	251	377	202	303	125	188	327	491
	36	277	416	235	353	191	287	120	181	293	441
	38	258	388	219	329	179	268	115	173	263	395
	40	240	360	204	306	166	250	110	165	237	357
	42	222	333	189	284	154	232	105	157	215	324
	44	204	307	174	262	142	214	99.4	149	196	295
	46	187	281	160	240	131	197	94.0	141	180	270
	48	172	258	147	220	120	180	88.7	133	165	248
	50	158	238	135	203	111	166	83.8	126	152	228
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		512	770	431	648	347	522	263	396	787	1180
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		416	624	351	527	283	424	214	321	640	960
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		147	221	125	188	101	153	75.3	113	209	314
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		177	266	132	198	97.8	147	66.3	99.6	223	335
Properties											
Area, in.^{2}		17.1		14.4		11.6		8.79		26.3	
$r_x = r_y$, in.		4.71		4.74		4.78		4.81		3.72	
$I_x = I_y$, in.^{4}		380		324		265		203		364	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Note: Confirm ASTM A1085 material availability before specifying.

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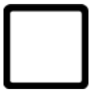
Table IV-8A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

HSS10

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS10x10x									
		⅝		½		⅜		5⁄16[f]		¼[c, f]	
t_{des}, in.		0.625		0.500		0.375		0.313		0.250	
lb/ft		76.3		62.5		47.9		40.4		32.6	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	671	1010	551	828	422	634	356	535	272	409
	1	670	1010	551	827	422	634	356	535	272	409
	2	669	1010	549	826	421	633	355	534	272	408
	3	666	1000	547	823	420	631	354	532	271	407
	4	663	996	545	819	418	628	352	530	270	406
	5	658	990	541	813	415	624	350	526	269	404
	6	653	982	537	807	412	619	348	523	268	402
	7	647	972	532	800	408	613	345	518	266	400
	8	640	962	526	791	404	607	341	513	264	397
	9	632	950	520	781	399	600	337	507	262	394
	10	623	937	513	771	394	592	333	500	260	391
	11	614	922	505	759	388	584	328	493	257	387
	12	603	907	497	747	382	574	323	485	255	383
	13	593	891	488	734	376	564	318	477	252	378
	14	581	873	479	720	369	554	312	469	249	374
	15	569	855	469	705	361	543	306	459	245	369
	16	556	836	459	690	354	531	299	450	242	363
	17	543	816	448	674	346	519	293	440	237	356
	18	529	795	437	657	337	507	286	429	231	348
	19	515	774	426	640	329	494	279	419	226	339
	20	500	752	414	622	320	481	271	408	220	330
	22	470	707	390	586	302	454	256	385	208	312
	24	440	661	365	549	283	426	241	362	195	294
	26	409	614	340	511	264	397	225	338	183	275
	28	378	567	315	473	245	369	209	314	170	256
	30	347	521	290	435	226	340	193	290	157	237
	32	317	476	265	399	208	312	177	266	145	218
	34	287	432	241	363	190	285	162	244	133	199
	36	259	389	218	328	172	259	147	221	121	182
	38	233	350	196	295	155	233	133	200	109	164
	40	210	315	177	266	140	210	120	180	98.6	148
	42	190	286	161	241	127	191	109	163	89.4	134
	44	173	261	146	220	116	174	99.1	149	81.5	122
	46	159	239	134	201	106	159	90.7	136	74.6	112
	48	146	219	123	185	97.2	146	83.3	125	68.5	103
	50	134	202	113	170	89.5	135	76.7	115	63.1	94.8
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		671	1010	551	828	422	635	356	536	287	432
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		546	819	449	673	345	517	290	435	234	351
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		183	274	153	230	120	180	102	153	83.1	125
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		194	291	161	242	126	189	103	155	72.6	109
Properties											
Area, in.^{2}		22.4		18.4		14.1		11.9		9.59	
$r_x = r_y$, in.		3.79		3.84		3.90		3.93		3.97	
$I_x = I_y$, in.^{4}		321		271		214		184		151	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.
Note: Confirm ASTM A1085 material availability before specifying.



HSS10–HSS9

Table IV-8A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS10x10x		HSS9x9x							
		3⁄16[c, f]		5⁄8		1⁄2		3⁄8		5⁄16	
t_{des}, in.		0.188		0.625		0.500		0.375		0.313	
lb/ft		24.7		67.8		55.7		42.8		36.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	167	251	596	895	491	738	377	567	317	477
	1	167	251	595	895	491	737	377	567	317	477
	2	167	250	594	892	489	735	376	565	316	475
	3	166	250	591	888	487	732	374	563	315	473
	4	166	249	587	882	484	728	372	559	313	471
	5	165	248	582	875	480	722	369	555	311	467
	6	164	247	576	866	475	715	366	550	308	463
	7	164	246	570	856	470	706	362	544	304	458
	8	162	244	562	844	464	697	357	537	301	452
	9	161	242	553	831	457	686	352	529	296	445
	10	160	241	543	817	449	675	346	520	292	438
	11	159	238	533	801	441	662	340	511	287	431
	12	157	236	522	784	432	649	333	501	281	422
	13	156	234	510	766	422	634	326	490	275	414
	14	154	231	497	748	412	619	319	479	269	404
	15	152	228	484	728	401	603	311	467	262	394
	16	150	225	471	707	390	587	303	455	256	384
	17	148	222	456	686	379	570	294	442	249	374
	18	146	219	442	664	367	552	286	429	241	363
	19	143	215	427	642	355	534	277	416	234	352
	20	141	212	412	619	343	516	267	402	226	340
	22	136	204	381	573	318	479	249	374	211	317
	24	131	196	350	527	293	441	230	346	195	293
	26	125	188	320	480	268	403	211	317	179	269
	28	119	179	289	435	243	366	192	289	164	246
	30	113	170	260	391	219	330	174	262	148	223
	32	107	161	232	348	196	295	156	235	134	201
	34	101	152	205	309	174	262	139	209	119	179
	36	92.6	139	183	275	155	234	124	187	106	160
	38	83.9	126	164	247	139	210	112	168	95.5	144
	40	75.7	114	148	223	126	189	101	151	86.2	130
	42	68.7	103	135	202	114	172	91.3	137	78.2	117
	44	62.6	94.0	123	184	104	156	83.2	125	71.2	107
	46	57.3	86.0	112	169	95.2	143	76.1	114	65.2	97.9
	48	52.6	79.0	103	155	87.4	131	69.9	105	59.8	89.9
	50	48.5	72.8	94.9	143	80.6	121	64.4	96.9	55.1	82.9
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		218	328	596	896	491	738	377	567	317	477
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		178	267	484	726	400	600	307	461	258	388
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		63.8	95.8	160	241	135	203	106	160	90.6	136
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		49.2	73.9	153	231	128	193	101	151	85.6	129
Properties											
Area, in.^{2}		7.29		19.9		16.4		12.6		10.6	
$r_x = r_y$, in.		3.99		3.38		3.43		3.50		3.53	
$I_x = I_y$, in.^{4}		116		227		193		154		132	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.
Note: Confirm ASTM A1085 material availability before specifying.

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Table IV-8A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

HSS9–HSS8

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

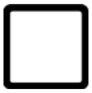
Shape		HSS9x9x				HSS8x8x					
		¼[f]		3⁄16[c, f]		⅝		½		⅜	
t_{des}, in.		0.250		0.188		0.625		0.500		0.375	
lb/ft		29.2		22.2		59.3		48.9		37.7	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	257	387	163	244	521	783	431	648	332	499
	1	257	386	162	244	520	782	431	647	332	499
	2	256	385	162	244	518	779	429	645	331	497
	3	255	384	162	243	515	775	427	641	329	495
	4	254	381	161	242	511	768	423	636	327	491
	5	252	379	160	241	506	760	419	630	323	486
	6	250	375	159	240	499	750	414	622	319	480
	7	247	371	158	238	491	739	407	612	315	473
	8	244	367	157	236	483	725	400	602	310	465
	9	240	361	156	234	473	711	393	590	304	457
	10	237	356	154	232	462	695	384	577	298	447
	11	233	350	152	229	451	678	375	564	291	437
	12	228	343	151	226	439	659	365	549	284	426
	13	223	336	149	223	426	640	355	533	276	415
	14	219	328	146	220	412	620	344	517	268	402
	15	213	321	144	217	398	599	333	500	259	390
	16	208	312	142	213	384	577	321	482	251	377
	17	202	304	139	209	369	555	309	464	242	363
	18	196	295	137	205	354	532	297	446	232	349
	19	191	286	134	201	339	509	284	427	223	335
	20	184	277	131	197	323	486	272	408	214	321
	22	172	259	125	188	292	439	247	371	195	293
	24	159	240	119	178	262	394	222	333	176	265
	26	147	220	112	168	232	349	198	297	158	237
	28	134	202	103	155	204	307	174	262	140	210
	30	122	183	93.5	141	178	268	152	229	123	185
	32	110	165	84.4	127	156	235	134	201	108	162
	34	98.3	148	75.7	114	139	208	119	178	95.7	144
	36	87.7	132	67.5	101	124	186	106	159	85.4	128
	38	78.7	118	60.6	91.1	111	167	94.9	143	76.6	115
	40	71.0	107	54.7	82.2	100	150	85.7	129	69.1	104
	42	64.4	96.8	49.6	74.5	90.8	137	77.7	117	62.7	94.3
	44	58.7	88.2	45.2	67.9	82.8	124	70.8	106	57.1	85.9
	46	53.7	80.7	41.3	62.1	75.7	114	64.8	97.4	52.3	78.6
	48	49.3	74.1	38.0	57.1	69.5	105	59.5	89.4	48.0	72.2
	50	45.5	68.3	35.0	52.6			54.8	82.4	44.3	66.5
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		257	387	196	294	521	783	431	648	332	500
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		209	314	160	239	426	639	351	527	271	406
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		74.1	111	57.0	85.7	138	207	117	176	92.7	139
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		61.7	92.7	41.5	62.3	118	177	99.1	149	78.1	117
Properties											
Area, in.^{2}		8.59		6.54		17.4		14.4		11.1	
$r_x = r_y$, in.		3.56		3.58		2.97		3.02		3.09	
$I_x = I_y$, in.^{4}		109		84.0		153		131		106	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A1085 material availability before specifying.



HSS8–HSS7

Table IV-8A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS8x8x						HSS7x7x			
		5⁄16		¼^f		3⁄16^{c,f}		⅝		½	
t_{des}, in.		0.313		0.250		0.188		0.625		0.500	
lb/ft		31.8		25.8		19.6		50.8		42.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	281	422	227	342	157	236	446	670	371	558
	1	280	421	227	341	157	235	445	669	371	557
	2	279	420	226	340	156	235	443	666	369	555
	3	278	418	225	338	156	234	440	661	366	550
	4	276	414	223	336	155	233	435	653	362	544
	5	273	410	221	333	154	232	429	644	357	537
	6	270	406	219	329	153	230	421	633	351	528
	7	266	400	216	324	152	228	412	620	344	517
	8	262	393	212	319	150	225	403	605	336	505
	9	257	386	209	313	148	223	392	589	328	492
	10	252	378	204	307	146	220	380	571	318	478
	11	246	370	200	300	144	216	367	552	308	463
	12	240	361	195	293	142	213	354	532	297	447
	13	234	351	190	285	139	209	340	511	286	430
	14	227	341	185	277	137	205	326	489	274	412
	15	220	331	179	269	134	201	311	467	262	394
	16	213	320	173	260	131	197	296	444	250	376
	17	205	308	167	251	128	192	280	421	238	357
	18	198	297	161	242	124	186	265	398	225	338
	19	190	285	155	233	119	179	250	375	212	319
	20	182	274	149	223	114	172	235	353	200	301
	22	166	250	136	204	105	157	205	308	176	264
	24	150	226	123	185	95.0	143	177	266	152	229
	26	135	203	111	167	85.6	129	151	227	130	196
	28	120	181	98.9	149	76.5	115	130	195	112	169
	30	106	159	87.3	131	67.8	102	113	170	98.0	147
	32	93.0	140	76.8	115	59.6	89.5	99.5	150	86.1	129
	34	82.4	124	68.0	102	52.8	79.3	88.2	133	76.3	115
	36	73.5	110	60.7	91.2	47.1	70.8	78.6	118	68.0	102
	38	65.9	99.1	54.4	81.8	42.3	63.5	70.6	106	61.1	91.8
	40	59.5	89.4	49.1	73.8	38.1	57.3	63.7	95.7	55.1	82.8
	42	54.0	81.1	44.6	67.0	34.6	52.0	57.8	86.8	50.0	75.1
	44	49.2	73.9	40.6	61.0	31.5	47.4				
	46	45.0	67.6	37.2	55.8	28.8	43.3				
	48	41.3	62.1	34.1	51.3	26.5	39.8				
	50	38.1	57.2	31.4	47.3	24.4	36.7				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		281	422	227	342	173	260	446	671	371	558
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		228	343	185	277	141	212	364	546	302	453
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		79.4	119	65.1	97.9	50.3	75.5	115	173	98.8	149
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		66.9	101	52.6	79.0	34.2	51.4	86.8	131	73.9	111
Properties											
Area, in.^{2}		9.37		7.59		5.78		14.9		12.4	
$r_x = r_y$, in.		3.12		3.15		3.18		2.56		2.61	
$I_x = I_y$, in.^{4}		91.0		75.2		58.4		97.6		84.7	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A1085 material availability before specifying.

Table IV-8A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Square HSS

HSS7–HSS6

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape		HSS7x7x								HSS6x6x	
		3⁄8		5⁄16		1⁄4		3⁄16[c, f]		5⁄8	
t_{des}, in.		0.375		0.313		0.250		0.188		0.625	
lb/ft		32.6		27.6		22.4		17.1		42.3	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	287	431	243	365	197	297	150	225	371	558
	1	286	430	243	365	197	296	150	225	370	557
	2	285	429	242	363	196	295	149	224	368	553
	3	283	425	240	361	195	293	149	223	364	547
	4	280	421	238	357	193	290	147	221	358	538
	5	277	416	235	353	191	286	146	219	351	527
	6	272	409	231	347	188	282	143	215	342	514
	7	267	401	227	341	184	277	141	212	332	499
	8	261	392	222	333	180	271	138	207	321	482
	9	255	383	216	325	176	265	135	203	309	464
	10	248	372	211	317	171	258	131	197	296	444
	11	240	361	204	307	167	250	128	192	282	424
	12	232	349	198	297	161	242	124	186	267	402
	13	224	336	191	287	156	234	119	180	253	380
	14	215	323	184	276	150	225	115	173	238	357
	15	206	310	176	265	144	216	111	166	222	334
	16	197	296	168	253	138	207	106	159	207	311
	17	188	282	161	241	132	198	101	152	192	289
	18	178	268	153	230	125	188	96.5	145	177	267
	19	169	254	145	218	119	179	91.8	138	163	245
	20	160	240	137	206	113	169	87.0	131	149	224
	22	141	212	121	183	100	150	77.5	117	124	186
	24	123	185	106	160	88.0	132	68.3	103	104	156
	26	106	160	92.1	138	76.4	115	59.6	89.5	88.5	133
	28	91.6	138	79.4	119	65.9	99.0	51.4	77.2	76.3	115
	30	79.8	120	69.2	104	57.4	86.2	44.8	67.3	66.5	99.9
	32	70.1	105	60.8	91.4	50.4	75.8	39.3	59.1	58.4	87.8
	34	62.1	93.4	53.8	80.9	44.7	67.1	34.8	52.4	51.8	77.8
	36	55.4	83.3	48.0	72.2	39.8	59.9	31.1	46.7		
	38	49.7	74.8	43.1	64.8	35.8	53.8	27.9	41.9		
	40	44.9	67.5	38.9	58.5	32.3	48.5	25.2	37.8		
	42	40.7	61.2	35.3	53.0	29.3	44.0	22.8	34.3		
	44	37.1	55.8	32.2	48.3	26.7	40.1	20.8	31.3		
	46							19.0	28.6		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		287	431	243	365	197	297	151	226	371	558
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		234	351	198	297	161	241	123	184	302	453
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		79.2	119	68.1	102	56.1	84.4	43.5	65.4	92.7	139
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		58.6	88.1	50.1	75.4	41.2	61.9	27.3	41.0	60.6	91.1
Properties											
Area, in.^{2}		9.58		8.12		6.59		5.03		12.4	
$r_x = r_y$, in.		2.68		2.71		2.74		2.77		2.15	
$I_x = I_y$, in.^{4}		68.7		59.6		49.4		38.6		57.4	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A1085 material availability before specifying.

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Table IV-8A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Square HSS

HSS6

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS6x6x									
		½		⅜		5⁄16		¼		3⁄16^f	
t_{des}, in.		0.500		0.375		0.313		0.250		0.188	
lb/ft		35.2		27.5		23.3		19.0		14.5	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	311	468	242	364	206	309	167	252	128	193
	1	311	467	241	363	205	309	167	251	128	192
	2	309	464	240	361	204	307	166	250	127	191
	3	305	459	238	357	202	304	164	247	126	189
	4	301	452	234	352	199	299	162	244	124	187
	5	295	443	230	345	196	294	159	240	122	184
	6	288	433	225	338	191	288	156	235	120	180
	7	280	421	219	329	187	280	152	229	117	176
	8	271	407	212	319	181	272	148	222	114	171
	9	261	392	205	308	175	263	143	215	110	165
	10	251	377	197	296	169	253	138	207	106	159
	11	239	360	189	284	162	243	132	199	102	153
	12	228	342	180	271	154	232	127	190	97.6	147
	13	216	324	171	257	147	221	121	181	93.1	140
	14	203	306	162	244	139	209	114	172	88.5	133
	15	191	287	153	230	131	198	108	163	83.7	126
	16	178	268	143	216	124	186	102	153	79.0	119
	17	166	250	134	201	116	174	95.6	144	74.2	112
	18	154	231	125	188	108	162	89.3	134	69.5	104
	19	142	213	116	174	100	151	83.1	125	64.8	97.3
	20	130	196	107	161	92.8	139	77.0	116	60.2	90.4
	22	109	163	89.8	135	78.4	118	65.5	98.4	51.3	77.1
	24	91.2	137	75.4	113	65.9	99.0	55.0	82.7	43.2	64.9
	26	77.7	117	64.3	96.6	56.1	84.3	46.9	70.4	36.8	55.3
	28	67.0	101	55.4	83.3	48.4	72.7	40.4	60.7	31.7	47.7
	30	58.4	87.7	48.3	72.6	42.1	63.4	35.2	52.9	27.6	41.6
	32	51.3	77.1	42.4	63.8	37.0	55.7	30.9	46.5	24.3	36.5
	34	45.5	68.3	37.6	56.5	32.8	49.3	27.4	41.2	21.5	32.4
	36	40.5	60.9	33.5	50.4	29.3	44.0	24.4	36.7	19.2	28.9
	38					26.3	39.5	21.9	33.0	17.2	25.9
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		311	468	242	364	206	309	167	252	128	193
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		254	380	197	295	167	251	136	204	104	156
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		80.8	122	65.7	98.8	56.9	85.5	47.2	70.9	36.7	55.2
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		52.1	78.4	41.9	63.0	36.2	54.4	29.7	44.6	22.3	33.5
Properties											
Area, in.^{2}		10.4		8.08		6.87		5.59		4.28	
$r_x = r_y$, in.		2.20		2.27		2.30		2.33		2.36	
$I_x = I_y$, in.^{4}		50.5		41.6		36.3		30.3		23.8	

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A1085 material availability before specifying.

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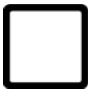


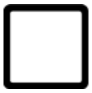
HSS5½–HSS5

Table IV-8A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS5½x5½x								HSS5x5x	
		⅜		5⁄16		¼		3⁄16		½	
t_{des}, in.		0.375		0.313		0.250		0.188		0.500	
lb/ft		24.9		21.2		17.3		13.3		28.4	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	219	330	187	281	152	229	117	175	250	376
	1	219	329	186	280	152	229	117	175	249	375
	2	217	327	185	278	151	227	116	174	247	371
	3	215	323	183	275	149	224	114	172	243	365
	4	211	317	180	270	147	221	113	169	238	357
	5	206	310	176	265	144	216	110	166	231	347
	6	201	302	171	258	140	211	108	162	223	335
	7	195	292	166	250	136	204	104	157	213	321
	8	188	282	160	241	131	197	101	152	203	306
	9	180	270	154	231	126	190	97.1	146	192	289
	10	172	258	147	221	121	182	93.0	140	181	272
	11	163	245	140	210	115	173	88.6	133	169	254
	12	154	232	132	199	109	164	84.1	126	157	236
	13	145	218	125	188	103	155	79.5	119	145	217
	14	136	204	117	176	96.7	145	74.7	112	132	199
	15	126	190	109	164	90.4	136	69.9	105	120	181
	16	117	176	101	152	84.1	126	65.2	98.0	109	164
	17	108	162	93.7	141	77.9	117	60.5	90.9	97.9	147
	18	99.0	149	86.2	130	71.8	108	55.8	83.9	87.3	131
	19	90.4	136	78.9	119	65.9	99.1	51.3	77.1	78.3	118
	20	82.0	123	71.8	108	60.2	90.5	46.9	70.6	70.7	106
	22	67.7	102	59.3	89.2	49.8	74.9	38.9	58.4	58.4	87.8
	24	56.9	85.5	49.9	75.0	41.8	62.9	32.7	49.1	49.1	73.8
	26	48.5	72.9	42.5	63.9	35.7	53.6	27.8	41.8	41.8	62.9
	28	41.8	62.9	36.6	55.1	30.7	46.2	24.0	36.1	36.1	54.2
	30	36.4	54.7	31.9	48.0	26.8	40.3	20.9	31.4	31.4	47.2
	32	32.0	48.1	28.1	42.2	23.5	35.4	18.4	27.6		
	34	28.4	42.6	24.8	37.3	20.9	31.3	16.3	24.5		
	36							14.6	22.0		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		219	330	187	281	152	229	117	176	250	376
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		179	268	152	228	124	186	95.2	143	204	306
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		59.0	88.7	51.3	77.1	42.7	64.1	33.4	50.2	62.9	94.5
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		34.4	51.8	29.9	45.0	24.7	37.2	19.2	28.9	34.2	51.4
Properties											
Area, in.^{2}		7.33		6.24		5.09		3.90		8.36	
$r_x = r_y$, in.		2.07		2.10		2.13		2.15		1.80	
$I_x = I_y$, in.^{4}		31.3		27.4		23.0		18.1		27.1	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A1085 material availability before specifying.



HSS5–HSS4½

Table IV-8A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Square HSS

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape		HSS5x5x								HSS4½x4½x	
		⅜		5⁄16		¼		3⁄16		½	
t_{des}, in.		0.375		0.313		0.250		0.188		0.500	
lb/ft		22.4		19.1		15.6		12.0		25.0	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	197	296	168	253	137	207	106	159	220	331
	1	196	295	168	252	137	206	105	158	219	330
	2	195	293	166	250	136	204	105	157	217	326
	3	192	288	164	246	134	201	103	155	212	319
	4	188	282	161	241	131	197	101	152	206	310
	5	183	274	156	235	128	192	98.6	148	199	298
	6	177	265	151	227	124	186	95.7	144	190	285
	7	170	255	146	219	119	180	92.3	139	180	270
	8	162	244	139	209	114	172	88.5	133	169	254
	9	154	231	133	199	109	164	84.5	127	157	236
	10	145	218	125	188	103	155	80.1	120	145	218
	11	136	205	118	177	97.3	146	75.6	114	133	200
	12	127	191	110	165	91.1	137	70.9	107	121	182
	13	118	177	102	154	84.8	127	66.2	99.5	109	164
	14	108	163	94.4	142	78.5	118	61.4	92.3	97.4	146
	15	99.3	149	86.7	130	72.3	109	56.7	85.2	86.3	130
	16	90.4	136	79.1	119	66.1	99.4	52.0	78.2	75.9	114
	17	81.8	123	71.8	108	60.2	90.5	47.5	71.4	67.2	101
	18	73.3	110	64.7	97.2	54.5	81.9	43.1	64.8	59.9	90.1
	19	65.8	98.9	58.0	87.2	48.9	73.5	38.8	58.3	53.8	80.9
	20	59.4	89.3	52.4	78.7	44.2	66.4	35.0	52.6	48.6	73.0
	22	49.1	73.8	43.3	65.1	36.5	54.8	28.9	43.5	40.1	60.3
	24	41.3	62.0	36.4	54.7	30.7	46.1	24.3	36.6	33.7	50.7
	26	35.1	52.8	31.0	46.6	26.1	39.3	20.7	31.2	28.7	43.2
	28	30.3	45.6	26.7	40.2	22.5	33.9	17.9	26.9		
	30	26.4	39.7	23.3	35.0	19.6	29.5	15.6	23.4		
	32					17.2	25.9	13.7	20.6		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		197	296	168	253	137	207	106	159	220	331
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		161	241	137	206	112	168	86.1	129	179	269
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		52.3	78.6	45.7	68.6	38.2	57.4	30.0	45.1	53.9	81.0
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		27.9	42.0	24.2	36.4	20.1	30.3	15.7	23.7	26.7	40.1
Properties											
Area, in.^{2}		6.58		5.62		4.59		3.53		7.36	
$r_x = r_y$, in.		1.86		1.89		1.92		1.95		1.59	
$I_x = I_y$, in.^{4}		22.8		20.1		16.9		13.4		18.7	

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A1085 material availability before specifying.

Table IV-8A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Square HSS

HSS4½–HSS4

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape		HSS4½x4½x								HSS4x4x	
		⅜		5⁄16		¼		3⁄16		½	
t_{des}, in.		0.375		0.313		0.250		0.188		0.500	
lb/ft		19.8		17.0		13.9		10.7		21.6	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	175	262	149	225	122	184	94.3	142	190	286
	1	174	261	149	224	122	183	94.0	141	189	285
	2	172	258	147	221	121	181	93.0	140	186	280
	3	169	253	145	217	119	178	91.4	137	181	273
	4	164	247	141	212	116	174	89.3	134	175	262
	5	159	238	136	205	112	168	86.5	130	166	250
	6	152	229	131	197	108	162	83.3	125	156	235
	7	145	218	125	187	103	155	79.7	120	146	219
	8	137	205	118	177	97.5	147	75.7	114	134	202
	9	128	193	111	167	91.8	138	71.4	107	122	184
	10	119	179	103	155	85.8	129	66.9	101	110	166
	11	110	165	95.6	144	79.6	120	62.2	93.5	98.5	148
	12	101	151	87.9	132	73.4	110	57.5	86.4	86.9	131
	13	91.5	138	80.1	120	67.1	101	52.8	79.3	75.8	114
	14	82.5	124	72.5	109	61.0	91.6	48.1	72.3	65.4	98.4
	15	73.9	111	65.2	98.0	55.0	82.6	43.5	65.4	57.0	85.7
	16	65.5	98.5	58.1	87.3	49.2	74.0	39.1	58.8	50.1	75.3
	17	58.0	87.2	51.5	77.4	43.7	65.7	34.8	52.4	44.4	66.7
	18	51.8	77.8	45.9	69.0	39.0	58.6	31.1	46.7	39.6	59.5
	19	46.5	69.8	41.2	61.9	35.0	52.6	27.9	41.9	35.5	53.4
	20	41.9	63.0	37.2	55.9	31.6	47.5	25.2	37.8	32.1	48.2
	22	34.6	52.1	30.7	46.2	26.1	39.2	20.8	31.3	26.5	39.8
	24	29.1	43.8	25.8	38.8	21.9	33.0	17.5	26.3		
	26	24.8	37.3	22.0	33.1	18.7	28.1	14.9	22.4		
	28			19.0	28.5	16.1	24.2	12.8	19.3		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		175	262	149	225	122	184	94.3	142	190	286
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		142	213	122	182	99.8	150	76.7	115	155	233
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		45.5	68.4	40.0	60.2	33.7	50.6	26.6	40.0	44.9	67.5
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		22.0	33.0	19.2	28.8	16.0	24.1	12.6	18.9	20.0	30.1
Properties											
Area, in.^{2}		5.83		4.99		4.09		3.15		6.36	
r_x = r_y, in.		1.66		1.69		1.72		1.75		1.39	
I_x = I_y, in.^{4}		16.0		14.2		12.1		9.62		12.3	

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A1085 material availability before specifying.

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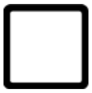


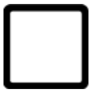
HSS4–HSS3½

Table IV-8A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS4x4x								HSS3½x3½x	
		⅜		5⁄16		¼		3⁄16		⅜	
t_{des}, in.		0.375		0.313		0.250		0.188		0.375	
lb/ft		17.3		14.8		12.2		9.42		14.7	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	152	229	131	196	107	162	83.2	125	130	195
	1	151	227	130	195	107	161	82.9	125	129	194
	2	149	224	128	192	106	159	81.8	123	126	190
	3	145	219	125	188	103	155	80.0	120	122	183
	4	140	211	121	182	99.8	150	77.5	117	116	175
	5	134	202	116	174	95.8	144	74.5	112	110	165
	6	127	191	110	165	91.0	137	70.9	107	102	153
	7	119	179	103	155	85.7	129	67.0	101	93.2	140
	8	110	166	96.0	144	80.0	120	62.6	94.2	84.2	127
	9	101	152	88.4	133	73.9	111	58.1	87.3	75.1	113
	10	92.2	139	80.7	121	67.7	102	53.4	80.3	66.1	99.3
	11	83.0	125	73.0	110	61.5	92.4	48.6	73.1	57.4	86.2
	12	73.9	111	65.3	98.2	55.3	83.1	43.9	66.0	49.0	73.7
	13	65.2	98.1	57.9	87.1	49.3	74.0	39.3	59.1	41.8	62.8
	14	56.9	85.5	50.9	76.4	43.5	65.3	34.9	52.4	36.0	54.2
	15	49.5	74.5	44.3	66.6	38.0	57.1	30.6	46.0	31.4	47.2
	16	43.5	65.5	38.9	58.5	33.4	50.2	26.9	40.4	27.6	41.5
	17	38.6	58.0	34.5	51.8	29.6	44.4	23.8	35.8	24.4	36.7
	18	34.4	51.7	30.8	46.2	26.4	39.6	21.2	31.9	21.8	32.8
	19	30.9	46.4	27.6	41.5	23.7	35.6	19.1	28.7	19.6	29.4
	20	27.9	41.9	24.9	37.5	21.4	32.1	17.2	25.9	17.7	26.5
	22	23.0	34.6	20.6	31.0	17.7	26.5	14.2	21.4		
	24	19.4	29.1	17.3	26.0	14.8	22.3	11.9	18.0		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		152	229	131	196	107	162	83.2	125	130	195
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		124	186	106	159	87.4	131	67.9	102	106	158
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		38.8	58.3	34.4	51.7	29.2	43.9	23.2	34.9	32.1	48.2
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		16.8	25.2	14.7	22.2	12.4	18.6	9.78	14.7	12.3	18.4
Properties											
Area, in.^{2}		5.08		4.36		3.59		2.78		4.33	
$r_x = r_y$, in.		1.45		1.48		1.51		1.54		1.25	
$I_x = I_y$, in.^{4}		10.7		9.59		8.22		6.61		6.74	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A1085 material availability before specifying.



HSS3½–HSS3

Table IV-8A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS3½x3½x						HSS3x3x			
		5⁄16		¼		3⁄16		⅜		5⁄16	
t_{des}, in.		0.313		0.250		0.188		0.375		0.313	
lb/ft		12.7		10.5		8.15		12.2		10.6	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	112	168	92.5	139	71.9	108	107	161	93.1	140
	1	111	167	91.9	138	71.4	107	106	160	92.3	139
	2	109	164	90.3	136	70.2	105	103	155	89.8	135
	3	106	159	87.5	132	68.2	102	98.2	148	85.7	129
	4	101	152	83.9	126	65.4	98.3	91.7	138	80.4	121
	5	95.4	143	79.4	119	62.1	93.3	84.0	126	74.0	111
	6	88.8	134	74.2	111	58.2	87.4	75.5	113	66.9	101
	7	81.7	123	68.5	103	53.9	81.0	66.5	100	59.3	89.2
	8	74.2	112	62.5	93.9	49.4	74.2	57.5	86.4	51.7	77.7
	9	66.5	100	56.3	84.6	44.7	67.2	48.7	73.2	44.2	66.4
	10	58.9	88.5	50.1	75.3	40.0	60.1	40.4	60.7	37.1	55.8
	11	51.5	77.3	44.0	66.2	35.3	53.1	33.4	50.2	30.7	46.2
	12	44.4	66.7	38.2	57.5	30.9	46.4	28.1	42.2	25.8	38.8
	13	37.8	56.9	32.8	49.2	26.6	40.0	23.9	35.9	22.0	33.1
	14	32.6	49.0	28.2	42.4	23.0	34.5	20.6	31.0	19.0	28.5
	15	28.4	42.7	24.6	37.0	20.0	30.0	18.0	27.0	16.5	24.8
	16	25.0	37.6	21.6	32.5	17.6	26.4	15.8	23.7	14.5	21.8
	17	22.1	33.3	19.2	28.8	15.6	23.4	14.0	21.0	12.9	19.3
	18	19.7	29.7	17.1	25.7	13.9	20.9				
	19	17.7	26.6	15.3	23.0	12.5	18.7				
	20	16.0	24.0	13.8	20.8	11.2	16.9				
	22					9.29	14.0				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		112	168	92.5	139	71.9	108	107	161	93.1	140
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		91.3	137	75.4	113	58.5	87.8	87.4	131	75.7	114
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		28.8	43.3	24.7	37.1	19.9	29.8	25.3	38.1	23.2	34.8
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		10.9	16.4	9.21	13.8	7.34	11.0	8.43	12.7	7.58	11.4
Properties											
Area, in.^{2}		3.74		3.09		2.40		3.58		3.11	
$r_x = r_y$, in.		1.28		1.31		1.34		1.04		1.07	
$I_x = I_y$, in.^{4}		6.11		5.29		4.30		3.89		3.59	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A1085 material availability before specifying.

Table IV-8A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

HSS3–HSS2½

A1085 Gr. A
F_y = 50 ksi
F_u = 65 ksi

Shape		HSS3x3x				HSS2½x2½x					
		¼		3⁄16		5⁄16		¼		3⁄16	
t_{des}, in.		0.250		0.188		0.313		0.250		0.188	
lb/ft		8.81		6.87		8.45		7.11		5.59	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	77.5	117	60.5	90.9	74.6	112	62.6	94.0	49.4	74.2
	1	76.9	116	60.0	90.2	73.5	110	61.8	92.8	48.8	73.4
	2	74.9	113	58.6	88.0	70.5	106	59.4	89.3	47.1	70.7
	3	71.7	108	56.2	84.5	65.8	98.8	55.7	83.6	44.3	66.6
	4	67.5	101	53.1	79.8	59.6	89.6	50.8	76.4	40.7	61.1
	5	62.4	93.8	49.4	74.2	52.6	79.1	45.2	67.9	36.5	54.8
	6	56.7	85.2	45.2	67.9	45.1	67.8	39.1	58.8	31.9	47.9
	7	50.6	76.1	40.7	61.1	37.6	56.6	33.0	49.7	27.2	40.9
	8	44.4	66.8	36.0	54.1	30.5	45.9	27.2	40.9	22.7	34.1
	9	38.3	57.6	31.4	47.2	24.2	36.4	21.8	32.7	18.4	27.7
	10	32.5	48.8	26.9	40.4	19.6	29.5	17.6	26.5	14.9	22.4
	11	27.0	40.6	22.6	34.0	16.2	24.4	14.6	21.9	12.3	18.5
	12	22.7	34.1	19.0	28.6	13.6	20.5	12.2	18.4	10.4	15.6
	13	19.4	29.1	16.2	24.4	11.6	17.5	10.4	15.7	8.83	13.3
	14	16.7	25.1	14.0	21.0	10.0	15.1	9.00	13.5	7.62	11.4
	15	14.5	21.9	12.2	18.3			7.84	11.8	6.63	9.97
	16	12.8	19.2	10.7	16.1						
	17	11.3	17.0	9.48	14.3						
	18	10.1	15.2	8.46	12.7						
	19			7.59	11.4						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		77.5	117	60.5	90.9	74.6	112	62.6	94.1	49.4	74.3
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		63.1	94.6	49.4	74.1	60.8	91.2	51.0	76.5	40.3	60.5
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		20.2	30.4	16.5	24.8	17.5	26.4	15.7	23.6	13.1	19.7
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		6.51	9.79	5.24	7.88	4.89	7.35	4.27	6.41	3.49	5.25

Properties

	HSS3x3x¼	HSS3x3x3⁄16	HSS2½x2½x5⁄16	HSS2½x2½x¼	HSS2½x2½x3⁄16
Area, in.^{2}	2.59	2.02	2.49	2.09	1.65
$r_x = r_y$, in.	1.10	1.14	0.869	0.899	0.931
$I_x = I_y$, in.^{4}	3.16	2.61	1.88	1.69	1.43

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A1085 material availability before specifying.

Table IV-8A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Square HSS

HSS2¼–HSS2

A1085 Gr. A

F_y = 50 ksi

F_u = 65 ksi

Shape		HSS2¼x2¼x				HSS2x2x			
		¼		3⁄16		¼		3⁄16	
t_{des}, in.		0.250		0.188		0.250		0.188	
lb/ft		6.26		4.96		5.41		4.32	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	55.1	82.8	43.7	65.7	47.6	71.5	38.0	57.1
	1	54.2	81.4	43.0	64.7	46.6	70.0	37.3	56.0
	2	51.6	77.5	41.1	61.8	43.6	65.6	35.1	52.8
	3	47.5	71.3	38.1	57.2	39.1	58.8	31.8	47.7
	4	42.3	63.5	34.2	51.4	33.6	50.5	27.6	41.5
	5	36.4	54.7	29.8	44.8	27.6	41.5	23.1	34.7
	6	30.3	45.6	25.1	37.8	21.7	32.6	18.5	27.8
	7	24.5	36.8	20.6	31.0	16.4	24.6	14.3	21.4
	8	19.1	28.7	16.3	24.5	12.5	18.8	10.9	16.4
	9	15.1	22.6	12.9	19.4	9.90	14.9	8.63	13.0
	10	12.2	18.3	10.4	15.7	8.02	12.0	6.99	10.5
	11	10.1	15.2	8.63	13.0	6.63	9.96	5.77	8.68
	12	8.47	12.7	7.26	10.9			4.85	7.29
	13	7.22	10.8	6.18	9.29				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		55.1	82.8	43.7	65.7	47.6	71.6	38.0	57.2
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		44.9	67.3	35.8	53.6	38.7	58.0	31.0	46.5
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		13.5	20.3	11.4	17.2	11.2	16.9	9.73	14.6
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		3.32	4.99	2.74	4.13	2.50	3.75	2.10	3.15

Properties

	HSS2¼x2¼x¼	HSS2¼x2¼x3⁄16	HSS2x2x¼	HSS2x2x3⁄16
Area, in.^{2}	1.84	1.46	1.59	1.27
$r_x = r_y$, in.	0.797	0.828	0.695	0.726
$I_x = I_y$, in.^{4}	1.17	1.00	0.769	0.670

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A1085 material availability before specifying.

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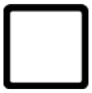
Table IV-8B
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

HSS22–HSS20

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS22x22x				HSS20x20x					
		⅞		$\frac{3}{4}^f$		⅞		¾		$\frac{5}{8}^f$	
t_{des}, in.		0.814		0.698		0.814		0.698		0.581	
lb/ft		245		212		221		192		161	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	2010	3030	1740	2620	1820	2740	1570	2370	1330	1990
	1	2010	3030	1740	2620	1820	2740	1570	2370	1330	1990
	2	2010	3030	1740	2620	1820	2730	1570	2370	1330	1990
	3	2010	3020	1740	2620	1820	2730	1570	2360	1320	1990
	4	2010	3020	1740	2610	1820	2730	1570	2360	1320	1990
	5	2010	3020	1740	2610	1810	2720	1570	2360	1320	1990
	6	2000	3010	1730	2610	1810	2720	1570	2350	1320	1980
	7	2000	3010	1730	2600	1800	2710	1560	2350	1320	1980
	8	2000	3000	1730	2600	1800	2710	1560	2340	1310	1970
	9	1990	2990	1720	2590	1790	2700	1550	2330	1310	1970
	10	1990	2990	1720	2580	1790	2690	1550	2330	1300	1960
	11	1980	2980	1710	2570	1780	2680	1540	2320	1300	1950
	12	1970	2970	1710	2570	1780	2670	1540	2310	1290	1950
	13	1970	2960	1700	2560	1770	2660	1530	2300	1290	1940
	14	1960	2940	1700	2550	1760	2640	1520	2290	1280	1930
	15	1950	2930	1690	2540	1750	2630	1520	2280	1280	1920
	16	1940	2920	1680	2530	1740	2620	1510	2270	1270	1910
	17	1930	2910	1670	2510	1730	2600	1500	2250	1260	1900
	18	1920	2890	1660	2500	1720	2590	1490	2240	1260	1890
	19	1910	2880	1660	2490	1710	2570	1480	2230	1250	1880
	20	1900	2860	1650	2480	1700	2550	1470	2210	1240	1860
	22	1880	2830	1630	2450	1670	2510	1450	2180	1220	1840
	24	1860	2790	1610	2420	1650	2470	1430	2140	1200	1810
	26	1830	2750	1580	2380	1620	2430	1400	2110	1180	1780
	28	1800	2710	1560	2350	1590	2390	1380	2070	1160	1750
	30	1770	2660	1540	2310	1560	2340	1350	2030	1140	1710
	32	1740	2620	1510	2270	1520	2290	1320	1990	1110	1680
	34	1710	2570	1480	2230	1490	2240	1290	1940	1090	1640
	36	1670	2520	1450	2180	1450	2180	1260	1900	1060	1600
	38	1640	2460	1420	2140	1420	2130	1230	1850	1040	1560
	40	1600	2410	1390	2090	1380	2070	1200	1800	1010	1520
	42	1570	2350	1360	2040	1340	2010	1160	1750	983	1480
	44	1530	2300	1330	1990	1300	1950	1130	1700	955	1440
	46	1490	2240	1290	1940	1260	1890	1100	1650	926	1390
	48	1450	2180	1260	1890	1220	1830	1060	1600	897	1350
	50	1410	2120	1230	1840	1180	1770	1030	1540	868	1300
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		2010	3030	1740	2620	1820	2740	1570	2370	1330	1990
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		1570	2350	1350	2030	1410	2120	1220	1840	1030	1540
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		573	862	499	750	515	774	449	675	382	574
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		1320	1990	1120	1680	1080	1620	943	1420	723	1090
Properties											
Area, in.^{2}		67.3		58.2		60.8		52.6		44.3	
$r_x = r_y$, in.		8.59		8.65		7.77		7.84		7.88	
$I_x = I_y$, in.^{4}		4970		4350		3670		3230		2750	

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.



HSS20–HSS18

Table IV-8B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS20x20x		HSS18x18x							
		½$^{c, f}$		⅞		¾		⅝^{f}		½$^{c, f}$	
t_{des}, in.		0.465		0.814		0.698		0.581		0.465	
lb/ft		131		197		171		144		117	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	962	1450	1630	2440	1410	2120	1190	1780	928	1400
	1	961	1450	1630	2440	1410	2120	1190	1780	928	1390
	2	961	1440	1620	2440	1410	2120	1180	1780	928	1390
	3	961	1440	1620	2440	1410	2120	1180	1780	927	1390
	4	960	1440	1620	2440	1410	2110	1180	1780	926	1390
	5	959	1440	1620	2430	1400	2110	1180	1770	925	1390
	6	958	1440	1610	2420	1400	2100	1180	1770	923	1390
	7	956	1440	1610	2420	1400	2100	1170	1760	922	1390
	8	955	1430	1600	2410	1390	2090	1170	1760	920	1380
	9	953	1430	1600	2400	1390	2080	1170	1750	918	1380
	10	951	1430	1590	2390	1380	2070	1160	1740	915	1380
	11	949	1430	1580	2380	1370	2070	1160	1740	912	1370
	12	946	1420	1580	2370	1370	2060	1150	1730	909	1370
	13	944	1420	1570	2360	1360	2040	1140	1720	906	1360
	14	941	1410	1560	2340	1350	2030	1140	1710	903	1360
	15	938	1410	1550	2330	1340	2020	1130	1700	899	1350
	16	935	1400	1540	2310	1340	2010	1120	1690	895	1340
	17	931	1400	1530	2300	1330	1990	1120	1680	891	1340
	18	928	1390	1520	2280	1320	1980	1110	1660	886	1330
	19	924	1390	1500	2260	1310	1960	1100	1650	881	1320
	20	920	1380	1490	2240	1290	1950	1090	1640	876	1320
	22	911	1370	1460	2200	1270	1910	1070	1610	866	1300
	24	902	1360	1430	2160	1250	1870	1050	1580	853	1280
	26	892	1340	1400	2110	1220	1830	1030	1550	836	1260
	28	881	1320	1370	2060	1190	1790	1010	1510	817	1230
	30	869	1310	1340	2010	1160	1750	981	1470	798	1200
	32	857	1290	1300	1960	1130	1700	956	1440	777	1170
	34	844	1270	1270	1900	1100	1660	929	1400	756	1140
	36	831	1250	1230	1850	1070	1610	902	1360	735	1100
	38	817	1230	1190	1790	1040	1560	875	1310	713	1070
	40	803	1210	1150	1730	1000	1510	846	1270	690	1040
	42	787	1180	1110	1670	967	1450	818	1230	667	1000
	44	772	1160	1070	1610	932	1400	789	1190	644	967
	46	753	1130	1030	1540	897	1350	759	1140	620	932
	48	730	1100	987	1480	862	1300	730	1100	596	896
	50	707	1060	946	1420	827	1240	700	1050	573	861
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		1070	1610	1630	2440	1410	2120	1190	1780	961	1440
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		834	1250	1260	1890	1090	1640	921	1380	747	1120
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		311	467	456	686	399	599	340	511	277	417
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		527	792	863	1300	753	1130	626	941	442	664
Properties											
Area, in.^{2}		35.8		54.3		47.1		39.6		32.1	
$r_x = r_y$, in.		7.95		6.97		7.02		7.07		7.13	
$I_x = I_y$, in.^{4}		2260		2630		2320		1980		1630	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Table IV-8B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

HSS16

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS16x16x									
		⅞		¾		⅝		½^f		⅜^{c,f}	
t_{des}, in.		0.814		0.698		0.581		0.465		0.349	
lb/ft		173		151		127		103		78.5	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	1430	2150	1240	1870	1050	1570	847	1270	549	825
	1	1430	2150	1240	1870	1050	1570	847	1270	549	825
	2	1430	2140	1240	1870	1050	1570	846	1270	549	824
	3	1420	2140	1240	1860	1050	1570	845	1270	548	824
	4	1420	2140	1240	1860	1040	1570	844	1270	547	823
	5	1420	2130	1230	1850	1040	1560	842	1270	547	822
	6	1410	2130	1230	1850	1040	1560	839	1260	546	820
	7	1410	2120	1230	1840	1030	1550	836	1260	544	818
	8	1400	2110	1220	1830	1030	1550	833	1250	543	816
	9	1400	2100	1220	1830	1030	1540	829	1250	541	814
	10	1390	2090	1210	1820	1020	1530	825	1240	540	811
	11	1380	2080	1200	1810	1010	1520	821	1230	538	808
	12	1370	2060	1190	1800	1010	1520	816	1230	536	805
	13	1360	2050	1190	1780	1000	1500	810	1220	533	802
	14	1350	2030	1180	1770	994	1490	805	1210	531	798
	15	1340	2020	1170	1760	986	1480	798	1200	528	794
	16	1330	2000	1160	1740	978	1470	792	1190	526	790
	17	1320	1980	1150	1720	969	1460	785	1180	523	786
	18	1300	1960	1140	1710	960	1440	778	1170	520	781
	19	1290	1940	1130	1690	951	1430	770	1160	516	776
	20	1280	1920	1110	1670	941	1410	762	1150	513	771
	22	1250	1880	1090	1630	920	1380	746	1120	506	760
	24	1220	1830	1060	1590	897	1350	728	1090	498	748
	26	1180	1780	1030	1550	873	1310	709	1070	489	735
	28	1150	1720	1000	1510	848	1270	689	1040	480	722
	30	1110	1670	970	1460	822	1240	668	1000	471	707
	32	1070	1610	938	1410	795	1200	646	971	460	692
	34	1030	1550	904	1360	767	1150	624	938	450	676
	36	994	1490	870	1310	739	1110	601	904	439	660
	38	954	1430	836	1260	710	1070	578	869	428	643
	40	913	1370	800	1200	681	1020	555	834	416	625
	42	873	1310	765	1150	651	979	531	799	404	607
	44	832	1250	730	1100	622	935	508	763	390	585
	46	791	1190	695	1040	592	890	484	728	372	559
	48	750	1130	660	992	563	846	461	692	354	532
	50	710	1070	625	940	534	803	437	657	336	506
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		1430	2150	1240	1870	1050	1580	847	1270	644	968
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		1110	1660	964	1450	815	1220	657	986	499	749
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		398	598	349	524	299	449	244	367	188	283
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		669	1010	586	881	499	750	372	559	247	371
Properties											
Area, in.^{2}		47.7		41.5		35.0		28.3		21.5	
$r_x = r_y$, in.		6.14		6.19		6.25		6.31		6.37	
$I_x = I_y$, in.^{4}		1800		1590		1370		1130		873	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

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Table IV-8B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

HSS16–HSS14

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS16x16x		HSS14x14x							
		5⁄16$^{c, f}$		7⁄8		3⁄4		5⁄8		1⁄2^{f}	
t_{des}, in.		0.291		0.814		0.698		0.581		0.465	
lb/ft		65.9		150		130		110		89.7	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	403	606	1230	1850	1070	1620	907	1360	737	1110
	1	403	606	1230	1850	1070	1610	907	1360	736	1110
	2	403	606	1230	1850	1070	1610	906	1360	735	1110
	3	403	605	1230	1850	1070	1610	904	1360	734	1100
	4	402	605	1230	1840	1070	1610	902	1360	732	1100
	5	402	604	1220	1840	1070	1600	899	1350	730	1100
	6	401	603	1220	1830	1060	1590	896	1350	727	1090
	7	400	601	1210	1820	1060	1590	892	1340	724	1090
	8	399	600	1200	1810	1050	1580	887	1330	720	1080
	9	398	598	1200	1800	1040	1570	881	1320	716	1080
	10	397	596	1190	1790	1040	1560	875	1320	711	1070
	11	395	594	1180	1770	1030	1550	869	1310	706	1060
	12	394	592	1170	1760	1020	1530	862	1300	700	1050
	13	392	590	1160	1740	1010	1520	854	1280	694	1040
	14	391	587	1150	1720	1000	1500	846	1270	688	1030
	15	389	585	1130	1710	990	1490	837	1260	681	1020
	16	387	582	1120	1690	979	1470	828	1240	674	1010
	17	385	578	1110	1670	968	1450	819	1230	666	1000
	18	383	575	1090	1640	955	1440	808	1220	658	989
	19	380	572	1080	1620	943	1420	798	1200	649	976
	20	378	568	1060	1600	929	1400	787	1180	640	963
	22	373	560	1030	1550	901	1350	764	1150	622	935
	24	367	552	996	1500	872	1310	739	1110	602	905
	26	361	543	960	1440	841	1260	713	1070	582	874
	28	355	534	922	1390	808	1210	686	1030	560	842
	30	348	523	884	1330	775	1160	659	990	538	808
	32	341	513	844	1270	741	1110	630	947	515	774
	34	334	502	804	1210	706	1060	601	904	492	739
	36	326	490	763	1150	671	1010	572	860	468	704
	38	318	478	722	1090	636	955	543	816	445	668
	40	310	466	682	1020	601	903	513	772	421	633
	42	301	453	642	964	566	850	484	728	398	598
	44	293	440	602	905	531	799	456	685	375	563
	46	284	427	563	846	498	748	427	642	352	529
	48	275	413	525	789	465	699	400	601	329	495
	50	266	399	488	734	433	651	373	560	308	462
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		542	815	1230	1850	1070	1620	907	1360	737	1110
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		422	632	958	1440	834	1250	704	1060	574	860
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		158	237	339	510	298	449	257	386	211	316
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		193	291	501	754	442	664	377	566	309	464
Properties											
Area, in.^{2}		18.1		41.2		35.9		30.3		24.6	
$r_x = r_y$, in.		6.39		5.33		5.38		5.44		5.49	
$I_x = I_y$, in.^{4}		739		1170		1040		897		743	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

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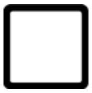


HSS14–HSS12

Table IV-8B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Square HSS

A500 Gr. C

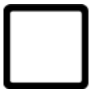
F_y = 50 ksi

F_u = 62 ksi

Shape		HSS14x14x				HSS12x12x					
		3⁄8[c, f]		5⁄16[c, f]		3⁄4		5⁄8		1⁄2	
t_{des}, in.		0.349		0.291		0.698		0.581		0.465	
lb/ft		68.3		57.4		110		93.3		76.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	527	792	388	584	907	1360	769	1160	626	940
	1	526	791	388	584	907	1360	769	1160	625	940
	2	526	791	388	583	905	1360	768	1150	625	939
	3	526	790	388	583	903	1360	766	1150	623	936
	4	525	789	387	582	900	1350	763	1150	621	933
	5	524	787	386	581	896	1350	760	1140	618	929
	6	522	785	385	579	891	1340	756	1140	615	924
	7	521	783	384	577	885	1330	751	1130	611	919
	8	519	780	383	576	878	1320	746	1120	607	912
	9	517	777	382	573	871	1310	739	1110	602	905
	10	515	773	380	571	862	1300	732	1100	596	896
	11	512	770	378	568	853	1280	725	1090	590	887
	12	509	765	376	566	843	1270	717	1080	584	878
	13	506	761	374	563	833	1250	708	1060	577	867
	14	503	756	372	559	821	1230	699	1050	569	856
	15	500	751	370	556	810	1220	689	1040	562	844
	16	496	746	367	552	797	1200	678	1020	553	832
	17	492	740	365	548	784	1180	667	1000	545	819
	18	488	734	362	544	770	1160	656	986	535	805
	19	484	727	359	539	756	1140	644	968	526	791
	20	480	721	356	535	741	1110	632	949	516	776
	22	470	707	349	525	710	1070	606	911	496	745
	24	460	691	342	514	678	1020	579	870	474	713
	26	444	668	335	503	644	968	551	829	452	680
	28	428	644	327	491	610	917	523	786	429	645
	30	412	619	318	478	575	864	494	742	406	610
	32	395	593	309	465	540	812	464	698	382	575
	34	377	567	300	451	505	759	435	654	359	540
	36	360	540	291	437	471	707	406	610	336	504
	38	342	514	281	422	437	656	377	567	313	470
	40	324	487	271	407	404	606	349	525	290	436
	42	306	460	259	389	371	558	322	484	268	403
	44	289	434	244	367	340	511	296	445	247	371
	46	272	408	230	345	311	467	271	407	226	339
	48	255	383	216	324	285	429	249	374	207	312
	50	238	358	202	303	263	395	229	344	191	287
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		560	842	470	707	907	1360	769	1160	626	941
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		434	651	366	549	704	1060	598	897	487	730
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		163	245	137	206	249	374	215	323	177	266
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		198	297	155	234	317	476	272	409	224	336
Properties											
Area, in.^{2}		18.7		15.7		30.3		25.7		20.9	
$r_x = r_y$, in.		5.55		5.58		4.56		4.62		4.68	
$I_x = I_y$, in.^{4}		577		490		631		548		457	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.



HSS12–HSS10

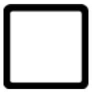
Table IV-8B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS12x12x								HSS10x10x	
		⅜^{f}		5⁄16$^{c, f}$		¼$^{c, f}$		3⁄16$^{c, f}$		¾	
t_{des}, in.		0.349		0.291		0.233		0.174		0.698	
lb/ft		58.1		48.9		39.4		29.8		89.5	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	479	720	372	559	253	380	149	225	740	1110
	1	479	720	372	559	253	380	149	225	739	1110
	2	478	719	371	558	252	379	149	224	737	1110
	3	477	717	371	557	252	379	149	224	735	1100
	4	475	715	370	556	252	378	149	224	731	1100
	5	473	712	369	554	251	377	148	223	726	1090
	6	471	708	368	552	250	376	148	223	720	1080
	7	468	704	366	550	249	375	148	222	713	1070
	8	465	699	364	548	248	373	147	221	705	1060
	9	461	693	362	545	247	371	146	220	696	1050
	10	457	687	360	541	246	369	146	219	686	1030
	11	453	680	358	538	244	367	145	217	675	1020
	12	448	673	355	534	242	364	144	216	664	998
	13	442	665	352	530	241	362	143	215	652	979
	14	437	657	349	525	239	359	142	213	639	960
	15	431	648	346	520	237	356	141	211	625	939
	16	425	638	343	515	235	353	139	210	611	918
	17	418	628	339	510	232	349	138	208	596	895
	18	411	618	335	504	230	346	137	206	580	872
	19	404	608	331	498	227	342	136	204	564	848
	20	397	596	327	492	225	338	134	202	548	824
	22	381	573	319	479	219	330	131	197	515	774
	24	365	549	307	461	213	321	128	192	480	722
	26	349	524	293	440	207	311	124	187	446	670
	28	331	498	279	419	200	301	121	181	411	618
	30	314	471	264	397	194	291	117	176	377	567
	32	296	445	249	375	186	280	113	170	344	516
	34	278	418	234	352	179	269	109	163	311	468
	36	260	391	220	330	171	257	105	157	280	420
	38	243	365	205	308	163	246	100	151	251	377
	40	226	339	191	287	155	233	95.9	144	227	341
	42	209	314	177	266	144	216	91.4	137	206	309
	44	193	289	163	245	133	200	86.9	131	187	281
	46	177	265	150	225	122	184	82.4	124	171	258
	48	162	244	138	207	112	169	77.9	117	157	237
	50	149	225	127	191	103	156	73.8	111	145	218
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		479	720	401	603	323	486	244	367	740	1110
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		372	558	313	470	251	377	189	284	574	860
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		138	207	116	174	94.6	142	64.4	96.8	198	298
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		156	235	120	181	88.9	134	59.5	89.4	211	318
Properties											
Area, in.^{2}		16.0		13.4		10.8		8.15		24.7	
$r_x = r_y$, in.		4.73		4.76		4.79		4.82		3.75	
$I_x = I_y$, in.^{4}		357		304		248		189		347	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.



HSS10

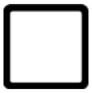
Table IV-8B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS10x10x									
		⅝		½		⅜		5⁄16[f]		¼[c, f]	
t_{des}, in.		0.581		0.465		0.349		0.291		0.233	
lb/ft		76.3		62.5		47.9		40.4		32.6	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	629	945	515	774	395	594	332	499	241	362
	1	628	944	515	773	395	594	332	499	241	362
	2	627	942	514	772	394	592	331	498	241	362
	3	625	939	512	769	393	590	330	496	240	361
	4	621	934	509	765	391	588	329	494	239	360
	5	617	928	506	760	388	584	327	491	238	358
	6	612	921	502	755	386	580	324	487	237	357
	7	607	912	497	748	382	574	321	483	236	354
	8	600	902	492	740	378	569	318	478	234	352
	9	593	891	486	731	374	562	315	473	232	349
	10	585	879	480	721	369	555	311	467	231	346
	11	576	865	473	711	364	547	306	460	228	343
	12	566	851	465	699	358	538	301	453	226	340
	13	556	835	457	687	352	529	296	445	223	336
	14	545	819	448	674	346	519	291	437	221	332
	15	534	802	439	660	339	509	285	429	218	327
	16	522	784	430	646	332	498	279	420	215	323
	17	509	765	420	631	324	487	273	411	212	318
	18	496	746	410	616	317	476	267	401	208	313
	19	483	726	399	600	309	464	260	391	205	308
	20	470	706	388	583	300	452	253	381	201	302
	22	442	664	366	550	284	426	239	360	193	291
	24	413	621	343	515	266	400	225	338	183	274
	26	384	577	319	480	249	374	210	316	171	257
	28	355	534	296	445	231	347	195	293	159	239
	30	326	490	273	410	213	321	180	271	147	221
	32	298	448	250	375	196	294	166	249	135	203
	34	271	407	228	342	179	269	152	228	124	186
	36	244	367	206	310	163	244	138	207	113	170
	38	219	329	185	278	147	220	125	187	102	153
	40	198	297	167	251	132	199	112	169	92.1	138
	42	179	270	152	228	120	180	102	153	83.6	126
	44	163	246	138	208	109	164	92.9	140	76.1	114
	46	150	225	126	190	100	150	85.0	128	69.7	105
	48	137	206	116	175	91.9	138	78.1	117	64.0	96.2
	50	127	190	107	161	84.7	127	71.9	108	59.0	88.6
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		629	945	515	774	395	594	332	500	268	403
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		490	735	400	600	307	460	258	387	208	312
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		172	259	144	216	112	169	95.5	143	77.9	117
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		183	275	151	228	118	177	90.9	137	65.8	98.9
Properties											
Area, in.^{2}		21.0		17.2		13.2		11.1		8.96	
$r_x = r_y$, in.		3.80		3.86		3.92		3.94		3.97	
$I_x = I_y$, in.^{4}		304		256		202		172		141	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.



HSS10–HSS9

Table IV-8B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS10x10x		HSS9x9x							
		3/16$^{c, f}$		5/8		1/2		3/8		5/16^{f}	
t_{des}, in.		0.174		0.581		0.465		0.349		0.291	
lb/ft		24.7		67.8		55.7		42.8		36.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	145	218	560	841	458	688	353	531	297	446
	1	145	218	559	841	458	688	353	531	297	446
	2	145	217	558	838	456	686	352	529	296	445
	3	144	217	555	835	454	683	351	527	295	443
	4	144	216	552	829	452	679	348	524	293	440
	5	143	216	547	823	448	673	346	520	291	437
	6	143	215	542	814	444	667	343	515	288	433
	7	142	213	535	805	439	659	339	509	285	428
	8	141	212	528	794	433	651	334	503	281	423
	9	140	211	520	782	426	641	330	495	277	417
	10	139	209	511	768	419	630	324	488	273	410
	11	138	207	501	754	412	619	319	479	268	403
	12	137	205	491	738	403	606	312	470	263	396
	13	135	203	480	721	394	593	306	460	258	387
	14	134	201	468	704	385	579	299	449	252	379
	15	132	199	456	686	375	564	291	438	246	370
	16	130	196	443	666	365	549	284	427	240	360
	17	129	193	430	647	355	533	276	415	233	350
	18	127	191	417	626	344	517	268	403	226	340
	19	125	188	403	606	333	500	260	390	219	330
	20	123	185	389	585	322	483	251	377	212	319
	22	119	178	360	542	299	449	234	351	198	297
	24	114	172	331	498	275	414	216	325	183	275
	26	109	164	302	455	252	379	198	298	168	253
	28	105	157	274	412	229	344	181	272	154	231
	30	99.4	149	247	371	207	311	164	246	139	210
	32	94.2	142	220	331	185	278	147	221	126	189
	34	88.8	134	195	293	164	247	131	197	112	169
	36	83.4	125	174	262	147	220	117	176	100	150
	38	78.0	117	156	235	132	198	105	158	89.9	135
	40	70.6	106	141	212	119	179	94.8	143	81.1	122
	42	64.0	96.2	128	192	108	162	86.0	129	73.6	111
	44	58.3	87.6	117	175	98.2	148	78.4	118	67.0	101
	46	53.4	80.2	107	160	89.8	135	71.7	108	61.3	92.2
	48	49.0	73.6	97.9	147	82.5	124	65.9	99.0	56.3	84.6
	50	45.2	67.9	90.3	136	76.0	114	60.7	91.2	51.9	78.0
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		202	304	560	842	458	689	353	531	297	446
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		157	236	434	651	357	535	274	412	231	346
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		59.3	89.1	152	228	127	191	99.7	150	85.0	128
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		44.2	66.5	145	218	121	182	94.3	142	78.6	118
Properties											
Area, in.^{2}		6.76		18.7		15.3		11.8		9.92	
$r_x = r_y$, in.		4.00		3.40		3.45		3.51		3.54	
$I_x = I_y$, in.^{4}		108		216		183		145		124	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

HSS9–HSS8

Table IV-8B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS9x9x				HSS8x8x					
		¼$^{c,\,f}$		$\frac{3}{16}^{c,\,f}$		⅝		½		⅜	
t_{des}, in.		0.233		0.174		0.581		0.465		0.349	
lb/ft		29.2		22.2		59.3		48.9		37.7	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	233	350	141	213	491	738	404	607	311	468
	1	232	349	141	212	490	737	404	607	311	467
	2	232	349	141	212	489	735	402	605	310	466
	3	231	348	141	212	486	730	400	601	308	463
	4	230	346	140	211	482	724	397	597	306	460
	5	229	345	140	210	477	717	393	590	303	455
	6	228	342	139	209	471	707	388	583	299	450
	7	226	340	138	207	463	697	382	575	295	444
	8	224	337	137	206	455	684	376	565	290	436
	9	222	334	136	204	446	671	369	554	285	428
	10	220	330	134	202	436	656	361	542	279	419
	11	217	326	133	200	426	640	352	529	273	410
	12	213	321	131	197	414	623	343	516	266	400
	13	209	314	130	195	402	605	333	501	259	389
	14	204	307	128	192	390	586	323	486	251	378
	15	199	300	126	189	377	566	313	470	243	366
	16	194	292	124	186	363	546	302	454	235	354
	17	189	284	122	183	349	525	291	437	227	341
	18	184	276	119	179	335	504	279	420	218	328
	19	178	268	117	176	321	482	268	403	210	315
	20	172	259	115	172	307	461	256	385	201	302
	22	161	242	110	165	278	417	233	350	183	275
	24	149	224	104	157	249	374	210	315	166	249
	26	137	206	98.6	148	221	333	187	281	148	223
	28	125	188	92.9	140	195	293	165	249	132	198
	30	114	171	87.0	131	170	256	145	217	116	174
	32	103	154	78.6	118	149	225	127	191	102	153
	34	91.9	138	70.5	106	132	199	113	169	90.2	136
	36	82.0	123	62.9	94.5	118	177	100	151	80.5	121
	38	73.6	111	56.5	84.9	106	159	90.2	136	72.2	109
	40	66.4	99.8	51.0	76.6	95.6	144	81.4	122	65.2	98.0
	42	60.2	90.5	46.2	69.5	86.8	130	73.8	111	59.1	88.9
	44	54.9	82.5	42.1	63.3	79.0	119	67.3	101	53.9	81.0
	46	50.2	75.5	38.5	57.9	72.3	109	61.5	92.5	49.3	74.1
	48	46.1	69.3	35.4	53.2	66.4	99.8	56.5	85.0	45.3	68.1
	50	42.5	63.9	32.6	49.0			52.1	78.3	41.7	62.7
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		240	361	181	273	491	738	404	608	311	468
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		187	280	141	212	381	572	313	470	242	363
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		69.5	104	53.0	79.7	131	196	110	166	87.1	131
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		55.3	83.2	37.3	56.1	112	168	93.6	141	73.4	110
Properties											
Area, in.^{2}		8.03		6.06		16.4		13.5		10.4	
$r_x = r_y$, in.		3.56		3.59		2.99		3.04		3.10	
$I_x = I_y$, in.^{4}		102		78.2		146		125		100	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Note: Heavy line indicates L_c/r equal to or greater than 200.

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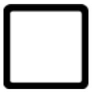

HSS8–HSS7

Table IV-8B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Square HSS

A500 Gr. C

F_y = 50 ksi

F_u = 62 ksi

Shape		HSS8x8x						HSS7x7x			
		5⁄16		1⁄4[f]		3⁄16[c, f]		5⁄8		1⁄2	
t_{des}, in.		0.291		0.233		0.174		0.581		0.465	
lb/ft		31.8		25.8		19.6		50.8		42.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	262	394	213	319	137	206	419	630	347	522
	1	262	394	212	319	137	206	418	629	347	521
	2	261	393	212	318	137	206	417	626	345	519
	3	260	390	211	316	136	205	413	621	343	515
	4	258	387	209	314	136	204	409	614	339	509
	5	255	384	207	311	135	203	403	606	334	503
	6	252	379	205	308	134	201	396	595	329	494
	7	249	374	202	303	133	199	388	583	322	484
	8	245	368	199	299	131	197	379	569	315	474
	9	240	361	195	293	130	195	369	554	307	461
	10	236	354	191	287	128	193	358	538	298	448
	11	230	346	187	281	126	190	346	520	289	434
	12	225	338	182	274	124	187	334	502	279	419
	13	219	329	178	267	122	184	321	482	269	404
	14	212	319	173	260	120	180	307	462	258	387
	15	206	310	167	252	118	177	294	441	247	371
	16	199	299	162	243	115	173	280	420	235	354
	17	192	289	156	235	112	169	265	399	224	336
	18	185	278	151	227	110	165	251	377	212	319
	19	178	267	145	218	107	160	237	356	200	301
	20	171	256	139	209	104	156	223	335	189	284
	22	156	234	127	191	97.1	146	195	293	166	250
	24	141	212	115	173	88.3	133	169	253	145	217
	26	127	191	104	156	79.5	120	144	216	124	186
	28	113	170	92.5	139	71.1	107	124	186	107	161
	30	99.5	150	81.7	123	63.0	94.7	108	162	93.1	140
	32	87.5	131	71.8	108	55.4	83.2	95.0	143	81.8	123
	34	77.5	116	63.6	95.6	49.0	73.7	84.1	126	72.4	109
	36	69.1	104	56.7	85.3	43.7	65.7	75.1	113	64.6	97.1
	38	62.0	93.2	50.9	76.5	39.3	59.0	67.4	101	58.0	87.2
	40	56.0	84.1	46.0	69.1	35.4	53.2	60.8	91.4	52.3	78.7
	42	50.8	76.3	41.7	62.7	32.1	48.3	55.1	82.9	47.5	71.4
	44	46.3	69.5	38.0	57.1	29.3	44.0				
	46	42.3	63.6	34.8	52.2	26.8	40.3				
	48	38.9	58.4	31.9	48.0	24.6	37.0				
	50	35.8	53.9	29.4	44.2	22.7	34.1				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		262	394	213	320	161	242	419	630	347	522
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		204	306	165	248	125	187	326	488	270	405
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		74.5	112	61.1	91.8	46.8	70.3	110	165	93.6	141
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		62.6	94.1	46.7	70.2	30.8	46.3	82.6	124	69.6	105
Properties											
Area, in.^{2}		8.76		7.10		5.37		14.0		11.6	
$r_x = r_y$, in.		3.13		3.15		3.18		2.58		2.63	
$I_x = I_y$, in.^{4}		85.6		70.7		54.4		93.4		80.5	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Note: Heavy line indicates L_c/r equal to or greater than 200.

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Table IV-8B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

HSS7–HSS6

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS7x7x								HSS6x6x	
		⅜		5⁄16		¼^{f}		3⁄16$^{c, f}$		⅝	
t_{des}, in.		0.349		0.291		0.233		0.174		0.581	
lb/ft		32.6		27.6		22.4		17.1		42.3	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	269	404	227	342	185	278	132	198	350	526
	1	268	403	227	341	184	277	131	198	350	525
	2	267	401	226	340	184	276	131	197	347	522
	3	265	398	224	337	182	274	130	196	343	516
	4	262	394	222	334	181	272	130	195	338	508
	5	259	389	219	330	178	268	129	193	331	498
	6	255	383	216	324	176	264	127	191	323	486
	7	250	376	212	319	173	259	126	189	314	472
	8	245	368	207	312	169	254	124	186	304	456
	9	239	359	203	304	165	248	122	183	292	439
	10	232	349	197	296	161	242	120	180	280	421
	11	225	338	191	288	156	235	117	176	267	402
	12	218	327	185	278	151	227	115	172	254	382
	13	210	316	179	269	146	219	111	167	240	361
	14	202	303	172	258	141	211	107	161	226	340
	15	194	291	165	248	135	203	103	154	212	318
	16	185	278	158	237	129	194	98.4	148	198	297
	17	176	265	151	226	124	186	94.0	141	184	276
	18	168	252	143	215	118	177	89.6	135	170	255
	19	159	239	136	204	112	168	85.2	128	156	235
	20	150	226	129	193	106	159	80.8	121	143	215
	22	133	200	114	172	94.2	142	72.0	108	119	179
	24	116	175	100	150	82.8	125	63.4	95.3	99.8	150
	26	100	151	86.7	130	72.1	108	55.3	83.1	85.1	128
	28	86.4	130	74.8	112	62.1	93.4	47.7	71.7	73.4	110
	30	75.3	113	65.1	97.9	54.1	81.3	41.6	62.5	63.9	96.0
	32	66.2	99.4	57.2	86.0	47.6	71.5	36.5	54.9	56.2	84.4
	34	58.6	88.1	50.7	76.2	42.1	63.3	32.4	48.6	49.7	74.8
	36	52.3	78.6	45.2	68.0	37.6	56.5	28.9	43.4	44.4	66.7
	38	46.9	70.5	40.6	61.0	33.7	50.7	25.9	38.9		
	40	42.3	63.6	36.6	55.1	30.4	45.8	23.4	35.1		
	42	38.4	57.7	33.2	49.9	27.6	41.5	21.2	31.9		
	44	35.0	52.6	30.3	45.5	25.2	37.8	19.3	29.0		
	46							17.7	26.6		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		269	404	227	342	185	278	140	210	350	527
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		209	313	176	265	144	215	109	163	272	408
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		74.6	112	64.1	96.3	52.7	79.3	40.5	60.9	88.9	134
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		55.1	82.9	47.2	70.9	38.7	58.1	24.7	37.1	57.9	87.0
Properties											
Area, in.^{2}		8.97		7.59		6.17		4.67		11.7	
$r_x = r_y$, in.		2.69		2.72		2.75		2.77		2.17	
$I_x = I_y$, in.^{4}		65.0		56.1		46.5		36.0		55.2	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.
[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

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HSS6

Table IV-8B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS6x6x									
		½		⅜		5/16		¼		3/16^f	
t_{des}, in.		0.465		0.349		0.291		0.233		0.174	
lb/ft		35.2		27.5		23.3		19.0		14.5	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	292	438	227	341	193	289	157	236	119	179
	1	291	437	226	340	192	289	157	235	119	179
	2	289	435	225	338	191	287	156	234	118	178
	3	286	430	223	335	189	284	154	232	117	176
	4	282	424	220	330	187	280	152	229	116	174
	5	277	416	216	324	183	275	150	225	114	171
	6	270	406	211	317	179	270	146	220	111	167
	7	263	395	206	309	175	263	143	215	109	163
	8	255	383	199	300	170	255	139	208	106	159
	9	246	369	193	289	164	247	134	202	102	154
	10	236	355	185	279	158	238	129	195	98.8	148
	11	226	339	178	267	152	228	124	187	95.0	143
	12	215	323	170	255	145	218	119	179	91.0	137
	13	204	306	161	242	138	207	113	170	86.8	130
	14	193	289	153	229	131	197	108	162	82.5	124
	15	181	272	144	216	123	186	102	153	78.2	117
	16	170	255	135	203	116	175	95.9	144	73.7	111
	17	158	238	126	190	109	164	90.0	135	69.3	104
	18	147	221	118	177	102	153	84.1	126	64.9	97.6
	19	136	204	109	164	94.4	142	78.4	118	60.6	91.0
	20	125	188	101	152	87.4	131	72.7	109	56.3	84.6
	22	104	157	85.0	128	74.0	111	61.9	93.0	48.1	72.3
	24	87.8	132	71.4	107	62.2	93.5	52.0	78.1	40.5	60.9
	26	74.8	112	60.8	91.4	53.0	79.6	44.3	66.6	34.5	51.9
	28	64.5	96.9	52.5	78.8	45.7	68.7	38.2	57.4	29.8	44.7
	30	56.2	84.4	45.7	68.7	39.8	59.8	33.3	50.0	25.9	39.0
	32	49.4	74.2	40.2	60.4	35.0	52.6	29.2	44.0	22.8	34.2
	34	43.7	65.7	35.6	53.5	31.0	46.6	25.9	38.9	20.2	30.3
	36	39.0	58.6	31.7	47.7	27.6	41.5	23.1	34.7	18.0	27.1
	38			28.5	42.8	24.8	37.3	20.7	31.2	16.2	24.3
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		292	438	227	341	193	289	157	236	119	179
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		227	340	176	265	149	224	122	183	92.7	139
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		77.0	116	62.1	93.3	53.6	80.6	44.4	66.7	34.3	51.5
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		49.4	74.3	39.4	59.3	33.9	51.0	27.9	42.0	19.5	29.3
Properties											
Area, in.^{2}		9.74		7.58		6.43		5.24		3.98	
$r_x = r_y$, in.		2.23		2.28		2.31		2.34		2.37	
$I_x = I_y$, in.^{4}		48.3		39.5		34.3		28.6		22.3	

^{f} Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table IV-8B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Square HSS

HSS5½–HSS5

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS5½x5½x								HSS5x5x	
		⅜		5⁄16		¼		3⁄16^f		½	
t_{des}, in.		0.349		0.291		0.233		0.174		0.465	
lb/ft		24.9		21.2		17.3		13.3		28.4	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	206	310	175	263	143	215	109	163	236	355
	1	205	309	175	263	142	214	108	163	235	353
	2	204	307	174	261	141	213	108	162	233	350
	3	202	303	171	258	140	210	106	160	229	345
	4	198	298	169	253	138	207	105	158	224	337
	5	194	291	165	248	135	203	103	154	218	328
	6	189	284	161	242	131	197	100	151	210	316
	7	183	275	156	234	127	192	97.3	146	202	303
	8	176	265	151	226	123	185	94.1	141	192	289
	9	169	254	145	217	118	178	90.5	136	182	274
	10	161	243	138	208	113	170	86.7	130	172	258
	11	153	231	132	198	108	162	82.7	124	161	241
	12	145	218	125	187	102	154	78.5	118	149	224
	13	137	205	117	177	96.5	145	74.2	112	138	207
	14	128	192	110	166	90.6	136	69.8	105	127	190
	15	119	179	103	155	84.7	127	65.4	98.3	115	173
	16	110	166	95.6	144	78.8	118	61.0	91.7	105	157
	17	102	153	88.4	133	73.0	110	56.6	85.1	94.2	142
	18	93.6	141	81.4	122	67.3	101	52.3	78.6	84.1	126
	19	85.6	129	74.6	112	61.8	92.9	48.1	72.3	75.5	113
	20	77.7	117	68.0	102	56.4	84.8	44.1	66.2	68.1	102
	22	64.2	96.5	56.2	84.4	46.7	70.1	36.5	54.9	56.3	84.6
	24	53.9	81.1	47.2	70.9	39.2	58.9	30.7	46.1	47.3	71.1
	26	46.0	69.1	40.2	60.4	33.4	50.2	26.2	39.3	40.3	60.6
	28	39.6	59.6	34.7	52.1	28.8	43.3	22.5	33.9	34.8	52.2
	30	34.5	51.9	30.2	45.4	25.1	37.7	19.6	29.5	30.3	45.5
	32	30.3	45.6	26.5	39.9	22.1	33.2	17.3	25.9		
	34	26.9	40.4	23.5	35.3	19.5	29.4	15.3	23.0		
	36							13.6	20.5		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		206	310	175	263	143	215	109	163	236	355
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		160	240	136	204	111	166	84.3	126	183	275
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		55.8	83.9	48.4	72.8	40.2	60.4	31.1	46.8	60.1	90.4
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		32.7	49.1	28.2	42.4	23.3	35.0	17.3	26.0	32.7	49.1
Properties											
Area, in.^{2}		6.88		5.85		4.77		3.63		7.88	
$r_x = r_y$, in.		2.08		2.11		2.13		2.16		1.82	
$I_x = I_y$, in.^{4}		29.7		25.9		21.7		17.0		26.0	

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table IV-8B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

HSS5–HSS4½

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS5x5x								HSS4½x4½x	
		⅜		5⁄16		¼		3⁄16		½	
t_{des}, in.		0.349		0.291		0.233		0.174		0.465	
lb/ft		22.4		19.1		15.6		12.0		25.0	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	**0**	185	278	157	237	129	193	98.2	148	208	313
	1	184	277	157	236	128	193	97.9	147	207	311
	2	183	275	156	234	127	191	97.1	146	205	308
	3	180	271	153	231	126	189	95.8	144	201	302
	4	176	265	150	226	123	185	94.0	141	195	293
	5	172	258	146	220	120	180	91.7	138	188	283
	6	166	250	142	213	116	175	89.0	134	180	270
	7	160	240	137	205	112	168	85.9	129	171	256
	8	153	229	131	196	107	161	82.4	124	160	241
	9	145	218	124	187	102	154	78.7	118	150	225
	10	137	206	118	177	97.0	146	74.7	112	139	208
	11	129	193	111	166	91.5	137	70.5	106	127	191
	12	120	180	103	156	85.7	129	66.2	99.5	116	174
	13	111	167	96.2	145	79.8	120	61.8	92.9	105	157
	14	103	154	88.9	134	74.0	111	57.4	86.3	93.9	141
	15	94.0	141	81.7	123	68.2	102	53.0	79.7	83.4	125
	16	85.6	129	74.6	112	62.4	93.8	48.7	73.2	73.5	110
	17	77.5	116	67.8	102	56.9	85.5	44.5	66.8	65.1	97.8
	18	69.6	105	61.2	91.9	51.5	77.4	40.4	60.7	58.0	87.2
	19	62.5	93.9	54.9	82.5	46.3	69.6	36.4	54.8	52.1	78.3
	20	56.4	84.8	49.6	74.5	41.8	62.8	32.9	49.4	47.0	70.7
	22	46.6	70.0	41.0	61.5	34.5	51.9	27.2	40.8	38.9	58.4
	24	39.2	58.9	34.4	51.7	29.0	43.6	22.8	34.3	32.6	49.1
	26	33.4	50.2	29.3	44.1	24.7	37.2	19.5	29.2	27.8	41.8
	28	28.8	43.2	25.3	38.0	21.3	32.1	16.8	25.2		
	30	25.1	37.7	22.0	33.1	18.6	27.9	14.6	22.0		
	32					16.3	24.5	12.8	19.3		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		185	278	157	237	129	194	98.2	148	208	313
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		144	216	122	184	100	150	76.3	114	162	242
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		49.5	74.4	43.2	64.9	36.0	54.1	28.0	42.1	51.8	77.8
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		26.4	39.8	22.9	34.4	19.0	28.5	14.7	22.1	25.4	38.3
Properties											
Area, in.^{2}		6.18		5.26		4.30		3.28		6.95	
$r_x = r_y$, in.		1.87		1.90		1.93		1.96		1.61	
$I_x = I_y$, in.^{4}		21.7		19.0		16.0		12.6		18.1	

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table IV-8B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Square HSS

HSS4½–HSS4

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS4½x4½x								HSS4x4x	
		⅜		5⁄16		¼		3⁄16		½	
t_{des}, in.		0.349		0.291		0.233		0.174		0.465	
lb/ft		19.8		17.0		13.9		10.7		21.6	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	164	247	140	211	115	173	87.7	132	180	271
	1	163	246	140	210	115	172	87.4	131	179	269
	2	162	243	138	208	113	170	86.5	130	176	265
	3	159	238	136	204	111	167	85.1	128	172	258
	4	154	232	132	199	109	163	83.0	125	166	249
	5	149	224	128	192	105	158	80.5	121	158	237
	6	143	215	123	185	101	152	77.5	117	149	224
	7	136	205	117	176	96.8	145	74.1	111	139	209
	8	129	194	111	167	91.8	138	70.4	106	128	193
	9	121	182	104	157	86.5	130	66.4	99.8	117	176
	10	112	169	97.3	146	80.9	122	62.2	93.5	106	160
	11	104	156	90.2	136	75.1	113	57.9	87.0	95.0	143
	12	95.3	143	82.9	125	69.3	104	53.5	80.4	84.1	126
	13	86.7	130	75.7	114	63.4	95.4	49.1	73.7	73.6	111
	14	78.3	118	68.6	103	57.7	86.7	44.7	67.2	63.7	95.8
	15	70.2	105	61.7	92.8	52.1	78.3	40.5	60.8	55.5	83.5
	16	62.3	93.7	55.1	82.9	46.7	70.2	36.4	54.7	48.8	73.3
	17	55.2	83.0	48.8	73.4	41.5	62.4	32.4	48.7	43.2	65.0
	18	49.2	74.0	43.6	65.5	37.0	55.6	28.9	43.4	38.6	58.0
	19	44.2	66.4	39.1	58.8	33.2	49.9	25.9	39.0	34.6	52.0
	20	39.9	59.9	35.3	53.0	30.0	45.1	23.4	35.2	31.2	46.9
	22	33.0	49.5	29.2	43.8	24.8	37.3	19.4	29.1	25.8	38.8
	24	27.7	41.6	24.5	36.8	20.8	31.3	16.3	24.4		
	26	23.6	35.5	20.9	31.4	17.7	26.7	13.9	20.8		
	28			18.0	27.1	15.3	23.0	11.9	18.0		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		164	247	140	211	115	173	87.7	132	180	271
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		127	191	109	163	89.3	134	68.2	102	140	210
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		43.3	65.0	38.0	57.0	31.8	47.8	24.9	37.4	43.4	65.3
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		20.9	31.4	18.1	27.3	15.1	22.7	11.8	17.7	19.2	28.9
Properties											
Area, in.^{2}		5.48		4.68		3.84		2.93		6.02	
$r_x = r_y$, in.		1.67		1.70		1.73		1.75		1.41	
$I_x = I_y$, in.^{4}		15.3		13.5		11.4		9.02		11.9	

Note: Heavy line indicates L_c/r equal to or greater than 200.

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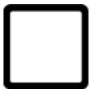


HSS4–HSS3½

Table IV-8B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Square HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS4x4x								HSS3½x3½x	
		⅜		5⁄16		¼		3⁄16		⅜	
t_{des}, in.		0.349		0.291		0.233		0.174		0.349	
lb/ft		17.3		14.8		12.2		9.42		14.7	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	143	215	123	184	101	152	77.2	116	122	184
	1	142	214	122	184	100	151	76.9	116	122	183
	2	140	211	120	181	99.1	149	75.9	114	119	179
	3	137	206	118	177	96.8	146	74.3	112	115	173
	4	132	199	114	171	93.8	141	72.0	108	110	166
	5	127	190	109	164	90.0	135	69.2	104	104	156
	6	120	180	103	156	85.6	129	66.0	99.2	96.4	145
	7	113	169	97.3	146	80.7	121	62.3	93.7	88.5	133
	8	105	157	90.6	136	75.4	113	58.4	87.7	80.1	120
	9	96.4	145	83.6	126	69.8	105	54.2	81.4	71.6	108
	10	87.9	132	76.4	115	64.0	96.1	49.8	74.9	63.1	94.8
	11	79.4	119	69.2	104	58.1	87.4	45.5	68.3	54.9	82.5
	12	71.0	107	62.0	93.2	52.3	78.7	41.1	61.8	47.1	70.7
	13	62.8	94.4	55.1	82.8	46.7	70.2	36.8	55.4	40.1	60.3
	14	55.0	82.7	48.5	72.8	41.3	62.1	32.7	49.2	34.6	52.0
	15	47.9	72.0	42.2	63.5	36.1	54.3	28.8	43.2	30.1	45.3
	16	42.1	63.3	37.1	55.8	31.7	47.7	25.3	38.0	26.5	39.8
	17	37.3	56.1	32.9	49.4	28.1	42.3	22.4	33.6	23.5	35.2
	18	33.3	50.0	29.3	44.1	25.1	37.7	20.0	30.0	20.9	31.4
	19	29.9	44.9	26.3	39.6	22.5	33.8	17.9	26.9	18.8	28.2
	20	27.0	40.5	23.8	35.7	20.3	30.5	16.2	24.3	16.9	25.5
	22	22.3	33.5	19.6	29.5	16.8	25.2	13.4	20.1		
	24	18.7	28.1	16.5	24.8	14.1	21.2	11.2	16.9		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		143	215	123	185	101	152	77.2	116	122	184
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		111	167	95.5	143	78.4	118	60.1	90.2	95.2	143
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		37.0	55.6	32.7	49.2	27.6	41.5	21.8	32.7	30.7	46.2
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		15.9	24.0	13.9	21.0	11.7	17.6	9.16	13.8	11.7	17.6
Properties											
Area, in.^{2}		4.78		4.10		3.37		2.58		4.09	
$r_x = r_y$, in.		1.47		1.49		1.52		1.55		1.26	
$I_x = I_y$, in.^{4}		10.3		9.14		7.80		6.21		6.49	

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table IV-8B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

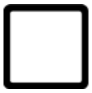
Square HSS

HSS3½–HSS3

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS3½x3½x						HSS3x3x			
		5/16		1/4		3/16		3/8		5/16	
t_{des}, in.		0.291		0.233		0.174		0.349		0.291	
lb/ft		12.7		10.5		8.15		12.2		10.6	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	105	158	87.1	131	67.1	101	101	153	88.0	132
	1	105	157	86.6	130	66.7	100	101	151	87.2	131
	2	103	154	85.0	128	65.5	98.5	97.8	147	84.9	128
	3	99.6	150	82.5	124	63.7	95.7	93.3	140	81.2	122
	4	95.2	143	79.1	119	61.1	91.9	87.4	131	76.2	115
	5	90.0	135	74.9	113	58.0	87.2	80.3	121	70.2	106
	6	83.9	126	70.1	105	54.5	81.9	72.4	109	63.6	95.6
	7	77.3	116	64.8	97.4	50.5	75.9	64.1	96.4	56.6	85.0
	8	70.3	106	59.2	89.0	46.3	69.6	55.7	83.7	49.4	74.2
	9	63.1	94.9	53.4	80.3	42.0	63.1	47.5	71.4	42.4	63.7
	10	56.0	84.1	47.6	71.6	37.6	56.6	39.8	59.8	35.7	53.6
	11	49.0	73.7	41.9	63.0	33.3	50.1	32.9	49.4	29.6	44.5
	12	42.4	63.7	36.5	54.9	29.2	43.9	27.6	41.5	24.9	37.4
	13	36.2	54.4	31.3	47.1	25.2	37.9	23.5	35.4	21.2	31.8
	14	31.2	46.9	27.0	40.6	21.7	32.7	20.3	30.5	18.3	27.4
	15	27.2	40.8	23.5	35.4	18.9	28.5	17.7	26.6	15.9	23.9
	16	23.9	35.9	20.7	31.1	16.6	25.0	15.5	23.3	14.0	21.0
	17	21.2	31.8	18.3	27.5	14.7	22.2	13.8	20.7	12.4	18.6
	18	18.9	28.4	16.3	24.6	13.2	19.8			11.0	16.6
	19	16.9	25.5	14.7	22.0	11.8	17.7				
	20	15.3	23.0	13.2	19.9	10.7	16.0				
	22			10.9	16.4	8.80	13.2				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		105	158	87.1	131	67.1	101	101	153	88.0	132
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		81.8	123	67.6	101	52.1	78.1	78.7	118	68.5	103
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		27.5	41.3	23.4	35.2	18.6	28.0	24.5	36.7	22.3	33.5
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		10.3	15.5	8.73	13.1	6.89	10.4	8.11	12.2	7.24	10.9
Properties											
Area, in.^{2}		3.52		2.91		2.24		3.39		2.94	
$r_x = r_y$, in.		1.29		1.32		1.35		1.06		1.08	
$I_x = I_y$, in.^{4}		5.84		5.04		4.05		3.78		3.45	

Note: Heavy line indicates L_c/r equal to or greater than 200.



HSS3–HSS2½

Table IV-8B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Square HSS

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS3x3x				HSS2½x2½x					
		¼		3/16		5/16		¼		3/16	
t_{des}, in.		0.233		0.174		0.291		0.233		0.174	
lb/ft		8.81		6.87		8.45		7.11		5.59	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	73.1	110	56.6	85.0	70.4	106	59.0	88.6	46.1	69.3
	1	72.4	109	56.1	84.4	69.4	104	58.2	87.5	45.6	68.5
	2	70.6	106	54.8	82.3	66.6	100	56.0	84.2	43.9	66.1
	3	67.6	102	52.6	79.1	62.3	93.6	52.6	79.0	41.4	62.2
	4	63.7	95.8	49.7	74.7	56.6	85.1	48.1	72.3	38.1	57.2
	5	59.0	88.7	46.2	69.5	50.1	75.3	42.9	64.4	34.2	51.3
	6	53.7	80.7	42.3	63.5	43.1	64.8	37.2	56.0	29.9	45.0
	7	48.1	72.2	38.0	57.2	36.1	54.3	31.5	47.4	25.6	38.5
	8	42.3	63.5	33.7	50.6	29.5	44.3	26.0	39.1	21.4	32.2
	9	36.6	55.0	29.4	44.1	23.5	35.2	20.9	31.5	17.4	26.2
	10	31.1	46.7	25.2	37.8	19.0	28.6	17.0	25.5	14.1	21.2
	11	25.9	39.0	21.2	31.8	15.7	23.6	14.0	21.1	11.7	17.5
	12	21.8	32.8	17.8	26.8	13.2	19.8	11.8	17.7	9.80	14.7
	13	18.6	27.9	15.2	22.8	11.2	16.9	10.0	15.1	8.35	12.6
	14	16.0	24.1	13.1	19.7	9.69	14.6	8.65	13.0	7.20	10.8
	15	13.9	21.0	11.4	17.1			7.53	11.3	6.27	9.43
	16	12.3	18.4	10.0	15.1						
	17	10.9	16.3	8.87	13.3						
	18	9.69	14.6	7.91	11.9						
	19			7.10	10.7						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		73.1	110	56.6	85.1	70.4	106	59.0	88.7	46.1	69.3
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		56.7	85.1	44.0	66.0	54.6	81.8	45.9	68.8	36.0	53.9
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		19.3	28.9	15.5	23.3	17.0	25.6	15.1	22.6	12.4	18.6
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		6.19	9.30	4.92	7.39	4.69	7.05	4.07	6.11	3.29	4.95
Properties											
Area, in.^{2}		2.44		1.89		2.35		1.97		1.54	
$r_x = r_y$, in.		1.11		1.14		0.880		0.908		0.937	
$I_x = I_y$, in.^{4}		3.02		2.46		1.82		1.63		1.35	

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table IV-8B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Square HSS

HSS2¼–HSS2

A500 Gr. C
F_y = 50 ksi
F_u = 62 ksi

Shape		HSS2¼x2¼x				HSS2x2x			
		¼		3⁄16		¼		3⁄16	
t_{des}, in.		0.233		0.174		0.233		0.174	
lb/ft		6.26		4.96		5.41		4.32	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the least radius of gyration, r_y	0	52.1	78.3	41.0	61.6	45.2	67.9	35.6	53.5
	1	51.3	77.0	40.4	60.7	44.3	66.5	34.9	52.5
	2	48.8	73.4	38.6	58.0	41.5	62.4	32.9	49.5
	3	45.0	67.7	35.8	53.8	37.3	56.1	29.9	44.9
	4	40.2	60.4	32.2	48.4	32.2	48.4	26.0	39.1
	5	34.7	52.2	28.1	42.3	26.6	40.0	21.8	32.8
	6	29.1	43.7	23.8	35.8	21.0	31.6	17.6	26.4
	7	23.5	35.4	19.6	29.4	15.9	24.0	13.6	20.5
	8	18.4	27.7	15.6	23.4	12.2	18.3	10.4	15.7
	9	14.6	21.9	12.3	18.5	9.64	14.5	8.24	12.4
	10	11.8	17.7	9.97	15.0	7.81	11.7	6.67	10.0
	11	9.75	14.7	8.24	12.4	6.46	9.70	5.52	8.29
	12	8.19	12.3	6.92	10.4			4.63	6.97
	13	6.98	10.5	5.90	8.87				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		52.1	78.3	41.0	61.7	45.2	68.0	35.6	53.6
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		40.6	60.9	31.9	47.9	35.0	52.5	27.7	41.5
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		13.0	19.5	10.8	16.3	10.9	16.4	9.25	13.9
Available Strength in Flexure, kip-ft		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
		3.19	4.80	2.59	3.90	2.41	3.62	1.99	2.99
Properties									
Area, in.^{2}		1.74		1.37		1.51		1.19	
$r_x = r_y$, in.		0.806		0.835		0.704		0.733	
$I_x = I_y$, in.^{4}		1.13		0.953		0.747		0.641	

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table IV-9A
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

HSS20.000–HSS16.000

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS20.000x				HSS18.000x				HSS16.000x	
		0.500		0.375[f]		0.500		0.375[f]		0.625	
t_{des}, in.		0.500		0.375		0.500		0.375		0.625	
lb/ft		104		78.7		93.5		70.7		103	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	916	1380	692	1040	823	1240	623	936	904	1360
	1	916	1380	691	1040	823	1240	623	936	904	1360
	2	915	1380	691	1040	822	1240	622	935	903	1360
	3	914	1370	690	1040	821	1230	621	934	901	1350
	4	913	1370	689	1040	820	1230	620	932	899	1350
	5	911	1370	688	1030	818	1230	619	930	896	1350
	6	909	1370	686	1030	815	1230	617	927	893	1340
	7	906	1360	684	1030	812	1220	615	924	889	1340
	8	903	1360	682	1030	809	1220	612	920	884	1330
	9	900	1350	679	1020	805	1210	609	916	879	1320
	10	896	1350	677	1020	801	1200	606	911	873	1310
	11	892	1340	674	1010	796	1200	603	906	866	1300
	12	887	1330	670	1010	791	1190	599	900	859	1290
	13	883	1330	667	1000	786	1180	595	894	851	1280
	14	877	1320	663	996	780	1170	591	888	843	1270
	15	872	1310	658	990	774	1160	586	881	835	1250
	16	866	1300	654	983	767	1150	581	873	825	1240
	17	859	1290	649	976	760	1140	576	865	816	1230
	18	853	1280	644	968	753	1130	570	857	806	1210
	19	846	1270	639	961	746	1120	565	849	795	1200
	20	839	1260	634	952	738	1110	559	840	784	1180
	22	823	1240	622	935	721	1080	546	821	761	1140
	24	807	1210	610	917	703	1060	533	801	737	1110
	26	789	1190	597	897	684	1030	518	779	711	1070
	28	770	1160	583	876	664	998	503	757	684	1030
	30	751	1130	568	854	643	966	488	733	656	987
	32	731	1100	553	831	621	934	472	709	628	944
	34	709	1070	537	807	599	901	455	684	599	901
	36	688	1030	521	783	577	867	438	659	570	857
	38	666	1000	504	758	554	832	421	633	541	813
	40	643	967	487	733	530	797	403	606	512	769
	42	620	932	470	707	507	762	386	580	483	726
	44	597	897	453	681	484	727	368	554	454	682
	46	574	862	435	655	460	692	351	527	426	640
	48	550	827	418	628	437	657	333	501	398	599
	50	527	792	400	602	414	623	316	475	372	558
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		916	1380	692	1040	823	1240	623	936	904	1360
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		746	1120	563	845	670	1010	507	761	736	1100
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		275	413	207	312	247	371	187	281	271	408
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		474	713	340	511	382	574	280	421	369	555
Properties											
Area, in.^{2}		30.6		23.1		27.5		20.8		30.2	
I, in.^{4}		1460		1110		1050		807		894	
r, in.		6.90		6.94		6.19		6.23		5.44	

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Note: Confirm ASTM A1085 material availability before specifying.

HSS16.000

Table IV-9A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS16.000x									
		0.500		0.438		0.375[f]		0.312[f]		0.250[c, f]	
t_{des}, in.		0.500		0.438		0.375		0.312		0.250	
lb/ft		82.8		72.9		62.6		52.3		42.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	728	1090	641	963	551	828	461	693	375	564
	1	727	1090	640	963	551	828	461	693	375	564
	2	727	1090	640	962	550	827	460	692	375	563
	3	725	1090	639	960	549	825	460	691	374	562
	4	723	1090	637	958	548	823	459	689	373	561
	5	721	1080	635	955	546	821	457	687	372	559
	6	718	1080	633	951	544	818	455	685	371	557
	7	715	1070	630	947	542	814	453	681	369	555
	8	711	1070	627	942	539	810	451	678	367	552
	9	707	1060	623	936	536	805	448	674	365	549
	10	702	1060	619	930	532	800	446	670	363	545
	11	697	1050	614	923	528	794	442	665	360	541
	12	692	1040	609	916	524	788	439	660	357	537
	13	686	1030	604	908	520	781	435	654	354	533
	14	679	1020	598	899	515	774	431	648	351	528
	15	672	1010	592	890	510	766	427	642	348	523
	16	665	1000	586	881	504	758	422	635	344	517
	17	657	988	579	871	499	750	418	628	340	511
	18	649	976	572	860	493	741	413	620	336	505
	19	641	963	565	849	487	731	408	613	332	499
	20	632	950	557	838	480	721	402	604	328	493
	22	614	923	541	814	466	701	391	587	318	479
	24	595	894	524	788	452	679	379	569	309	464
	26	574	863	506	761	437	656	366	550	298	449
	28	553	831	488	733	421	632	353	530	288	432
	30	531	798	468	704	404	607	339	509	277	416
	32	508	764	449	674	387	582	325	488	265	399
	34	485	729	428	644	370	556	311	467	254	381
	36	462	694	408	613	353	530	296	445	242	363
	38	439	659	388	583	335	504	281	423	230	346
	40	415	624	367	552	318	477	267	401	218	328
	42	392	589	347	521	300	451	252	379	206	310
	44	369	555	327	491	283	425	238	358	195	292
	46	346	521	307	461	266	400	224	336	183	275
	48	324	488	287	432	249	375	210	315	172	258
	50	303	455	268	403	233	350	196	295	161	242
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		728	1090	641	963	551	828	461	693	371	558
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		592	888	522	782	449	673	375	563	302	453
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		218	328	192	289	165	248	138	208	111	167
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		299	450	264	398	225	339	183	275	143	214
Properties											
Area, in.^{2}		24.3		21.4		18.4		15.4		12.4	
I, in.^{4}		732		649		562		473		384	
r, in.		5.48		5.50		5.53		5.55		5.57	

[c] Shape is slender with respect to uniform compression for F_y = 50 ksi.

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Note: Confirm ASTM A1085 material availability before specifying.

HSS14.000

Table IV-9A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS14.000x									
		0.625		0.500		0.375		0.312[f]		0.250[f]	
t_{des}, in.		0.625		0.500		0.375		0.312		0.250	
lb/ft		89.4		72.2		54.6		45.7		36.7	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	787	1180	635	954	482	724	401	603	323	486
	1	787	1180	634	954	482	724	401	603	323	486
	2	786	1180	634	952	481	723	400	602	323	485
	3	784	1180	632	950	480	722	400	601	322	484
	4	782	1170	630	947	479	719	398	599	321	483
	5	778	1170	627	943	477	716	397	596	320	481
	6	774	1160	624	938	474	713	395	593	318	478
	7	769	1160	621	933	471	709	392	590	316	475
	8	764	1150	616	926	468	704	390	586	314	472
	9	758	1140	611	919	465	698	387	581	312	469
	10	751	1130	606	911	461	692	384	576	309	465
	11	744	1120	600	902	456	686	380	571	306	460
	12	736	1110	594	893	452	679	376	565	303	456
	13	727	1090	587	883	446	671	372	559	300	451
	14	718	1080	580	872	441	663	367	552	296	445
	15	708	1060	572	860	435	654	363	545	292	440
	16	698	1050	564	848	429	645	358	537	288	434
	17	687	1030	556	835	423	636	352	530	284	427
	18	676	1020	547	822	416	626	347	521	280	421
	19	664	999	537	808	409	615	341	513	275	414
	20	652	980	528	793	402	604	335	504	271	407
	22	627	942	508	763	387	582	323	485	261	392
	24	600	902	487	732	371	558	310	465	250	376
	26	573	861	465	699	355	533	296	445	239	360
	28	544	818	442	665	338	508	282	424	228	343
	30	516	775	419	630	321	482	268	402	216	325
	32	486	731	396	595	303	456	253	381	205	308
	34	457	687	373	560	285	429	239	359	193	290
	36	428	643	349	525	268	403	224	337	181	273
	38	399	600	326	490	251	377	210	315	170	255
	40	371	557	304	456	233	351	195	294	158	238
	42	343	516	282	423	217	326	182	273	147	221
	44	317	476	260	391	200	301	168	253	136	205
	46	290	436	239	359	185	277	155	233	126	189
	48	267	401	219	330	169	255	142	214	116	174
	50	246	369	202	304	156	235	131	197	107	160
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		787	1180	635	954	482	725	401	603	323	486
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		641	962	517	775	392	589	327	490	263	395
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		236	355	190	286	145	217	120	181	97.0	146
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		279	420	228	342	174	261	142	214	111	167
Properties											
Area, in.^{2}		26.3		21.2		16.1		13.4		10.8	
I, in.^{4}		589		484		373		314		255	
r, in.		4.73		4.78		4.82		4.84		4.86	

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Note: Confirm ASTM A1085 material availability before specifying.

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HSS12.750–
HSS10.750

Table IV-9A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS12.750x						HSS10.750x			
		0.500		0.375		0.250[f]		0.500		0.375	
t_{des}, in.		0.500		0.375		0.250		0.500		0.375	
lb/ft		65.5		49.6		33.4		54.8		41.6	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	575	864	437	657	294	442	482	724	365	549
	1	575	864	437	657	294	442	482	724	365	549
	2	574	862	436	656	293	441	480	722	364	547
	3	572	860	435	654	293	440	479	719	363	545
	4	570	856	433	651	291	438	476	715	361	542
	5	567	852	431	648	290	436	473	710	358	538
	6	563	847	429	644	288	433	468	704	355	534
	7	559	841	426	640	286	430	464	697	352	528
	8	555	833	422	634	284	427	458	688	347	522
	9	549	826	418	628	281	423	452	679	343	515
	10	543	817	414	622	279	419	445	669	338	508
	11	537	807	409	615	275	414	438	658	332	499
	12	530	797	404	607	272	409	430	646	326	491
	13	523	786	398	599	268	403	421	633	320	481
	14	515	774	393	590	265	398	412	619	313	471
	15	507	761	386	581	260	391	403	605	306	460
	16	498	748	380	571	256	385	393	590	299	449
	17	489	735	373	561	252	378	383	575	291	438
	18	479	720	366	550	247	371	372	559	284	426
	19	469	705	359	539	242	364	361	543	275	414
	20	459	690	351	528	237	356	350	526	267	402
	22	438	658	335	504	227	340	327	492	250	376
	24	416	625	319	479	216	324	304	457	233	350
	26	393	591	302	453	204	307	281	422	215	324
	28	370	556	284	427	193	290	258	387	198	297
	30	347	521	267	401	181	272	235	353	181	272
	32	323	486	249	375	169	254	213	320	164	247
	34	300	451	232	348	158	237	191	288	148	222
	36	278	417	215	323	146	220	171	257	132	199
	38	255	384	198	297	135	203	153	230	119	179
	40	234	352	182	273	124	187	138	208	107	161
	42	213	320	166	249	114	171	126	189	97.2	146
	44	194	292	151	227	103	155	114	172	88.6	133
	46	178	267	138	208	94.6	142	105	157	81.1	122
	48	163	245	127	191	86.9	131	96.1	144	74.4	112
	50	150	226	117	176	80.1	120	88.6	133	68.6	103
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		575	864	437	657	294	442	482	725	365	549
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		468	702	356	534	239	359	392	589	297	446
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		172	259	131	197	88.2	133	145	217	110	165
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		187	282	143	215	93.0	140	131	197	101	152
Properties											
Area, in.^{2}		19.2		14.6		9.82		16.1		12.2	
I, in.^{4}		362		279		192		212		165	
r, in.		4.33		4.38		4.42		3.63		3.67	

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Note: Confirm ASTM A1085 material availability before specifying.

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HSS10.750–
HSS10.000

Table IV-9A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS10.750x		HSS10.000x							
		0.250[f]		0.625		0.500		0.375		0.312	
t_{des}, in.		0.250		0.625		0.500		0.375		0.312	
lb/ft		28.1		62.6		50.8		38.6		32.3	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	**0**	247	371	551	828	446	670	338	508	284	427
	1	247	371	550	827	446	670	338	508	284	427
	2	246	370	549	825	444	668	337	507	283	426
	3	245	369	546	821	442	665	336	504	282	424
	4	244	367	543	815	440	661	333	501	280	421
	5	242	364	538	808	436	655	331	497	278	418
	6	240	361	532	800	431	648	327	492	275	414
	7	238	358	526	790	426	641	324	486	272	409
	8	235	354	518	779	420	632	319	480	269	404
	9	232	349	510	766	414	622	314	473	265	398
	10	229	344	501	753	406	611	309	464	260	391
	11	225	338	491	738	399	599	303	456	255	384
	12	221	333	480	722	390	586	297	446	250	376
	13	217	326	469	705	381	573	290	436	245	367
	14	213	320	457	687	372	558	283	426	239	359
	15	208	313	444	668	362	544	276	415	233	350
	16	203	305	431	648	351	528	268	403	226	340
	17	198	298	418	628	341	512	260	391	220	330
	18	193	290	404	608	330	496	252	379	213	320
	19	187	282	390	587	319	479	244	367	206	309
	20	182	273	376	565	307	462	236	354	199	299
	22	171	256	347	521	284	427	218	328	184	277
	24	159	239	318	478	261	392	201	302	170	255
	26	147	221	289	434	237	357	183	276	155	233
	28	136	204	261	392	215	323	166	250	141	212
	30	124	186	233	350	193	290	150	225	127	191
	32	113	170	207	311	171	258	134	201	114	171
	34	102	153	183	275	152	228	119	178	101	152
	36	91.5	137	163	246	135	204	106	159	90.0	135
	38	82.1	123	147	220	122	183	95.0	143	80.8	121
	40	74.1	111	132	199	110	165	85.7	129	72.9	110
	42	67.2	101	120	180	99.5	150	77.8	117	66.1	99.4
	44	61.2	92.0	109	164	90.7	136	70.8	106	60.3	90.6
	46	56.0	84.2	100	150	83.0	125	64.8	97.4	55.1	82.9
	48	51.4	77.3	91.9	138	76.2	115	59.5	89.5	50.6	76.1
	50	47.4	71.3	84.7	127	70.2	106	54.9	82.5	46.7	70.1
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		247	371	551	828	446	671	338	509	284	428
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		201	302	449	673	363	545	275	413	232	347
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		74.1	111	165	248	134	201	101	153	85.3	128
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		67.9	102	137	206	113	170	86.8	131	73.1	110

Properties

Area, in.^{2}	8.25	18.4	14.9	11.3	9.50
I, in.^{4}	114	203	169	132	112
r, in.	3.71	3.32	3.36	3.41	3.43

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Note: Confirm ASTM A1085 material availability before specifying.

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HSS10.000–
HSS9.625

Table IV-9A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS10.000x				HSS9.625x					
		0.250		0.188^f		0.500		0.375		0.312	
t_{des}, in.		0.250		0.188		0.500		0.375		0.312	
lb/ft		26.1		19.7		48.8		37.1		31.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	229	345	174	261	428	643	326	490	273	411
	1	229	344	174	261	428	643	326	490	273	410
	2	229	343	173	260	426	641	325	489	272	409
	3	228	342	172	259	424	638	323	486	271	407
	4	226	340	171	257	421	633	321	483	269	405
	5	224	337	170	255	417	627	318	479	267	401
	6	222	334	168	253	413	621	315	473	264	397
	7	220	330	166	250	407	612	311	467	261	392
	8	217	326	164	247	401	603	306	461	257	386
	9	213	321	162	243	395	593	301	453	253	380
	10	210	316	159	239	387	582	296	445	248	373
	11	206	310	156	235	379	570	290	435	243	365
	12	202	303	153	230	370	556	283	426	238	357
	13	197	297	150	225	361	543	276	415	232	349
	14	193	290	146	220	351	528	269	404	226	340
	15	188	282	143	214	341	513	261	393	220	330
	16	183	275	139	209	331	497	254	381	213	320
	17	178	267	135	203	320	481	246	369	206	310
	18	172	259	131	197	309	464	237	357	199	300
	19	167	250	127	190	297	447	229	344	192	289
	20	161	242	122	184	286	430	220	331	185	278
	22	149	225	114	171	263	395	203	305	171	257
	24	138	207	105	158	239	360	185	278	156	235
	26	126	190	96.2	145	216	325	168	252	142	213
	28	115	172	87.5	131	194	292	151	227	128	192
	30	103	155	79.0	119	173	259	135	202	114	171
	32	92.7	139	70.9	107	152	229	119	179	101	151
	34	82.3	124	63.1	94.8	135	202	105	158	89.2	134
	36	73.4	110	56.2	84.5	120	181	93.9	141	79.6	120
	38	65.9	99.1	50.5	75.9	108	162	84.3	127	71.4	107
	40	59.5	89.4	45.6	68.5	97.3	146	76.0	114	64.5	96.9
	42	53.9	81.1	41.3	62.1	88.3	133	69.0	104	58.5	87.9
	44	49.2	73.9	37.7	56.6	80.4	121	62.8	94.4	53.3	80.1
	46	45.0	67.6	34.5	51.8	73.6	111	57.5	86.4	48.7	73.3
	48	41.3	62.1	31.6	47.6	67.6	102	52.8	79.4	44.8	67.3
	50	38.1	57.2	29.2	43.8	62.3	93.6	48.7	73.1	41.3	62.0
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		229	345	174	261	428	644	326	491	273	411
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		187	280	141	212	349	523	266	399	223	334
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		68.8	103	52.1	78.3	128	193	97.9	147	82.0	123
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		59.4	89.3	42.9	64.5	104	156	80.1	120	67.6	102
Properties											
Area, in.^{2}		7.66		5.80		14.3		10.9		9.13	
I, in.^{4}		91.1		69.8		150		117		99.1	
r, in.		3.45		3.47		3.23		3.27		3.29	

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Note: Confirm ASTM A1085 material availability before specifying.

HSS9.625–
HSS8.625

Table IV-9A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS9.625x				HSS8.625x					
		0.250		0.188[f]		0.625		0.500		0.375	
t_{des}, in.		0.250		0.188		0.625		0.500		0.375	
lb/ft		25.1		19.0		53.5		43.4		33.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	220	331	167	251	470	706	383	576	291	437
	1	220	331	167	250	469	706	383	575	291	437
	2	220	330	166	250	468	703	381	573	290	435
	3	218	328	165	249	465	698	379	569	288	433
	4	217	326	164	247	460	692	376	564	285	429
	5	215	323	163	245	455	684	371	558	282	424
	6	213	320	161	242	448	674	366	550	278	418
	7	210	316	159	239	441	663	360	541	274	412
	8	207	312	157	236	432	650	353	531	269	404
	9	204	307	154	232	423	636	346	520	263	396
	10	200	301	152	228	413	620	338	507	257	387
	11	196	295	149	224	401	603	329	494	251	377
	12	192	289	146	219	390	585	319	480	244	366
	13	188	282	142	214	377	567	309	465	236	355
	14	183	275	139	208	364	547	299	449	228	343
	15	178	267	135	203	350	527	288	433	220	331
	16	173	259	131	197	337	506	277	416	212	319
	17	167	251	127	191	322	484	266	399	204	306
	18	162	243	123	185	308	463	254	382	195	293
	19	156	235	119	178	293	441	242	364	186	280
	20	150	226	114	172	279	419	231	347	178	267
	22	139	209	106	159	250	376	207	312	160	241
	24	127	191	96.8	146	222	333	184	277	143	215
	26	116	174	88.1	132	194	292	162	244	126	190
	28	104	157	79.6	120	169	253	141	212	110	166
	30	93.3	140	71.3	107	147	221	123	185	96.1	144
	32	82.7	124	63.3	95.2	129	194	108	163	84.5	127
	34	73.3	110	56.1	84.3	114	172	95.9	144	74.8	112
	36	65.3	98.2	50.0	75.2	102	153	85.5	129	66.7	100
	38	58.6	88.1	44.9	67.5	91.5	138	76.7	115	59.9	90.0
	40	52.9	79.5	40.5	60.9	82.6	124	69.3	104	54.1	81.3
	42	48.0	72.1	36.8	55.3	74.9	113	62.8	94.4	49.0	73.7
	44	43.7	65.7	33.5	50.4	68.3	103	57.2	86.0	44.7	67.2
	46	40.0	60.1	30.7	46.1	62.5	93.9	52.4	78.7	40.9	61.4
	48	36.8	55.2	28.2	42.3			48.1	72.3	37.5	56.4
	50	33.9	50.9	25.9	39.0						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		220	331	167	251	470	707	383	576	291	437
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		179	269	136	204	383	574	312	468	237	355
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		66.1	99.4	50.0	75.2	141	212	115	173	87.3	131
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		54.9	82.5	39.8	59.9	100	150	82.3	124	63.6	95.6
Properties											
Area, in.^{2}		7.36		5.57		15.7		12.8		9.72	
I, in.^{4}		81.0		62.1		126		106		82.9	
r, in.		3.32		3.34		2.84		2.88		2.92	

[f] Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A1085 material availability before specifying.

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HSS8.625–
HSS7.625

Table IV-9A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS8.625x						HSS7.625x			
		0.322		0.250		0.188^f		0.375		0.328	
t_{des}, in.		0.322		0.250		0.188		0.375		0.328	
lb/ft		28.6		22.4		17.0		29.1		25.6	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	251	378	197	296	149	224	256	384	225	338
	1	251	378	197	296	149	224	255	384	225	338
	2	250	376	196	295	148	223	254	382	224	336
	3	249	374	195	293	148	222	252	379	222	334
	4	247	371	193	290	146	220	249	375	220	330
	5	244	367	191	287	145	218	246	369	216	325
	6	241	362	189	284	143	215	241	363	213	320
	7	237	356	186	279	141	211	236	355	208	313
	8	233	350	182	274	138	208	231	347	203	306
	9	228	342	179	269	135	204	225	338	198	298
	10	223	335	175	263	132	199	218	328	192	289
	11	217	326	170	256	129	194	211	317	186	279
	12	211	317	166	249	126	189	203	305	179	269
	13	205	308	161	242	122	183	195	294	172	259
	14	198	298	156	234	118	178	187	281	165	248
	15	191	287	150	226	114	172	179	268	158	237
	16	184	277	145	218	110	165	170	256	150	226
	17	177	266	139	209	106	159	161	242	143	214
	18	169	255	133	201	102	153	153	229	135	203
	19	162	244	128	192	97.2	146	144	216	127	191
	20	154	232	122	183	92.8	139	135	203	120	180
	22	139	210	110	166	84.0	126	118	178	105	157
	24	125	187	98.6	148	75.3	113	102	153	90.5	136
	26	110	166	87.4	131	66.9	101	87.1	131	77.3	116
	28	96.7	145	76.8	115	58.9	88.5	75.1	113	66.6	100
	30	84.2	127	66.9	100	51.3	77.1	65.4	98.3	58.1	87.3
	32	74.0	111	58.8	88.3	45.1	67.8	57.5	86.4	51.0	76.7
	34	65.6	98.5	52.1	78.2	39.9	60.0	50.9	76.5	45.2	67.9
	36	58.5	87.9	46.4	69.8	35.6	53.5	45.4	68.3	40.3	60.6
	38	52.5	78.9	41.7	62.6	32.0	48.0	40.8	61.3	36.2	54.4
	40	47.4	71.2	37.6	56.5	28.9	43.4	36.8	55.3	32.7	49.1
	42	43.0	64.6	34.1	51.3	26.2	39.3	33.4	50.2	29.6	44.5
	44	39.1	58.8	31.1	46.7	23.8	35.8				
	46	35.8	53.8	28.4	42.7	21.8	32.8				
	48	32.9	49.4	26.1	39.3	20.0	30.1				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		251	378	197	296	149	224	256	384	225	338
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		205	307	160	241	121	182	208	312	183	275
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		75.4	113	59.1	88.8	44.7	67.2	76.7	115	67.5	102
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		55.4	83.3	43.7	65.6	32.5	48.9	49.2	73.9	43.7	65.6

Properties

Area, in.^{2}	8.40	6.58	4.98	8.54	7.52
I, in.^{4}	72.5	57.7	44.4	56.3	50.1
r, in.	2.94	2.96	2.98	2.57	2.58

^{f} Shape exceeds the compact limit for flexure for F_y = 50 ksi.

Notes: Heavy line indicates L_c/r equal to or greater than 200.

Confirm ASTM A1085 material availability before specifying.

HSS7.500

Table IV-9A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS7.500x									
		0.500		0.375		0.312		0.250		0.188	
t_{des}, in.		0.500		0.375		0.312		0.250		0.188	
lb/ft		37.4		28.6		24.0		19.4		14.7	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	329	495	251	378	211	317	170	256	129	194
	1	329	494	251	377	211	317	170	256	129	194
	2	327	492	250	375	210	315	169	254	129	193
	3	324	487	247	372	208	313	168	252	128	192
	4	320	482	245	368	206	309	166	250	126	190
	5	316	474	241	362	203	305	164	246	124	187
	6	310	465	237	356	199	299	161	242	122	184
	7	303	455	232	348	195	293	157	237	120	180
	8	295	444	226	340	190	286	154	231	117	176
	9	287	431	220	330	185	278	150	225	114	171
	10	278	417	213	320	179	269	145	218	111	166
	11	268	402	206	309	173	260	140	211	107	161
	12	257	387	198	297	167	251	135	203	103	155
	13	247	371	190	285	160	241	130	195	99.2	149
	14	235	354	182	273	153	230	124	187	95.1	143
	15	224	337	173	260	146	220	119	178	90.9	137
	16	212	319	164	247	139	209	113	170	86.5	130
	17	201	302	156	234	132	198	107	161	82.2	124
	18	189	284	147	221	124	187	101	152	77.8	117
	19	178	267	138	208	117	176	95.4	143	73.4	110
	20	166	250	129	195	110	165	89.6	135	69.0	104
	22	144	216	113	169	95.8	144	78.3	118	60.5	90.9
	24	123	184	96.6	145	82.5	124	67.5	101	52.4	78.7
	26	104	157	82.3	124	70.2	106	57.6	86.5	44.7	67.3
	28	90.1	135	70.9	107	60.6	91.0	49.6	74.6	38.6	58.0
	30	78.5	118	61.8	92.9	52.8	79.3	43.2	65.0	33.6	50.5
	32	69.0	104	54.3	81.6	46.4	69.7	38.0	57.1	29.5	44.4
	34	61.1	91.8	48.1	72.3	41.1	61.7	33.7	50.6	26.2	39.3
	36	54.5	81.9	42.9	64.5	36.6	55.1	30.0	45.1	23.3	35.1
	38	48.9	73.5	38.5	57.9	32.9	49.4	27.0	40.5	20.9	31.5
	40	44.1	66.3	34.8	52.2	29.7	44.6	24.3	36.6	18.9	28.4
	42			31.5	47.4	26.9	40.5	22.1	33.2	17.1	25.8
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		329	495	251	378	211	317	170	256	129	194
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		268	402	205	307	172	258	139	208	105	158
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		98.8	149	75.4	113	63.3	95.2	51.1	76.8	38.8	58.3
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		61.1	91.9	47.7	71.6	40.2	60.4	32.7	49.1	25.2	37.9
Properties											
Area, in.^{2}		11.0		8.39		7.05		5.69		4.32	
I, in.^{4}		67.7		53.4		45.6		37.5		28.9	
r, in.		2.48		2.52		2.54		2.56		2.59	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A1085 material availability before specifying.

HSS7.000

Table IV-9A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS7.000x									
		0.500		0.375		0.312		0.250		0.188	
t_{des}, in.		0.500		0.375		0.312		0.250		0.188	
lb/ft		34.7		26.6		22.3		18.0		13.7	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	305	459	234	351	196	295	159	238	120	181
	1	305	458	233	350	196	295	158	238	120	181
	2	303	455	232	348	195	293	158	237	119	180
	3	300	451	230	345	193	290	156	235	118	178
	4	296	445	227	340	191	286	154	232	117	176
	5	291	437	223	335	187	282	152	228	115	173
	6	284	427	218	328	184	276	148	223	113	169
	7	277	416	213	320	179	269	145	218	110	166
	8	269	404	207	311	174	262	141	212	107	161
	9	260	391	200	301	169	254	137	205	104	156
	10	250	376	193	290	163	245	132	198	100	151
	11	240	361	185	279	157	235	127	191	96.7	145
	12	229	345	177	267	150	225	122	183	92.7	139
	13	218	328	169	254	143	215	116	175	88.6	133
	14	207	311	161	242	136	204	111	166	84.4	127
	15	195	293	152	229	129	194	105	158	80.0	120
	16	183	276	143	215	122	183	99.0	149	75.7	114
	17	172	258	135	202	114	172	93.1	140	71.3	107
	18	160	241	126	189	107	161	87.3	131	66.9	101
	19	149	224	117	176	99.8	150	81.6	123	62.6	94.0
	20	138	207	109	164	92.8	139	75.9	114	58.3	87.6
	22	116	175	92.8	139	79.3	119	65.0	97.7	50.1	75.2
	24	97.8	147	78.1	117	66.8	100	54.9	82.5	42.3	63.6
	26	83.3	125	66.5	100	56.9	85.5	46.7	70.3	36.1	54.2
	28	71.8	108	57.3	86.2	49.1	73.7	40.3	60.6	31.1	46.7
	30	62.6	94.1	50.0	75.1	42.7	64.2	35.1	52.8	27.1	40.7
	32	55.0	82.7	43.9	66.0	37.6	56.5	30.9	46.4	23.8	35.8
	34	48.7	73.2	38.9	58.5	33.3	50.0	27.3	41.1	21.1	31.7
	36	43.5	65.3	34.7	52.1	29.7	44.6	24.4	36.6	18.8	28.3
	38	39.0	58.6	31.1	46.8	26.6	40.0	21.9	32.9	16.9	25.4
	40									15.2	22.9
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		305	459	234	351	196	295	159	239	120	181
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		249	373	190	285	160	240	129	194	98.0	147
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		91.6	138	70.1	105	58.9	88.6	47.6	71.6	36.1	54.3
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		52.9	79.5	41.2	61.9	34.9	52.5	28.4	42.8	21.8	32.7
Properties											
Area, in.^{2}		10.2		7.80		6.56		5.30		4.02	
I, in.^{4}		54.2		43.0		36.7		30.2		23.4	
r, in.		2.30		2.35		2.37		2.39		2.41	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A1085 material availability before specifying.

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Table IV-9A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape	HSS6.875x									
	0.500		0.375		0.312		0.250		0.188	
t_{des}, in.	0.500		0.375		0.312		0.250		0.188	
lb/ft	34.1		26.1		21.9		17.7		13.4	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r: 0	299	450	229	345	193	289	156	234	118	178
1	299	449	229	344	192	289	155	234	118	177
2	297	446	228	342	191	287	154	232	117	176
3	294	442	225	339	189	284	153	230	116	175
4	290	435	222	334	187	280	151	227	115	172
5	284	427	218	328	183	276	148	223	113	170
6	278	418	213	321	179	270	145	218	111	166
7	271	407	208	313	175	263	142	213	108	162
8	262	394	202	303	170	255	138	207	105	158
9	253	381	195	293	164	247	133	200	102	153
10	244	366	188	282	158	238	128	193	98.0	147
11	233	351	180	271	152	228	123	185	94.3	142
12	223	334	172	259	145	218	118	177	90.3	136
13	211	318	164	246	138	208	112	169	86.2	129
14	200	300	155	233	131	197	107	161	81.9	123
15	188	283	147	220	124	186	101	152	77.6	117
16	177	265	138	207	117	175	95.2	143	73.2	110
17	165	248	129	194	109	164	89.3	134	68.8	103
18	154	231	120	181	102	154	83.5	126	64.4	96.8
19	142	214	112	168	95.0	143	77.8	117	60.1	90.3
20	131	197	103	155	88.0	132	72.1	108	55.9	84.0
22	110	166	87.4	131	74.6	112	61.4	92.3	47.7	71.7
24	92.6	139	73.4	110	62.7	94.3	51.6	77.6	40.2	60.4
26	78.9	119	62.6	94.0	53.4	80.3	44.0	66.1	34.3	51.5
28	68.0	102	53.9	81.1	46.1	69.3	37.9	57.0	29.5	44.4
30	59.2	89.0	47.0	70.6	40.1	60.3	33.0	49.6	25.7	38.7
32	52.1	78.3	41.3	62.1	35.3	53.0	29.0	43.6	22.6	34.0
34	46.1	69.3	36.6	55.0	31.2	47.0	25.7	38.6	20.0	30.1
36	41.1	61.8	32.6	49.1	27.9	41.9	22.9	34.5	17.9	26.9
38			29.3	44.0	25.0	37.6	20.6	30.9	16.0	24.1
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	299	450	229	345	193	289	156	234	118	178
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	244	366	187	280	157	235	127	190	96.3	144
Available Strength in Shear, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	89.8	135	68.8	103	57.8	86.8	46.7	70.2	35.5	53.3
Available Strength in Flexure, kip-ft	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
	50.9	76.5	39.7	59.6	33.4	50.3	27.4	41.3	21.0	31.5
Properties										
Area, in.^{2}	10.0		7.66		6.43		5.20		3.95	
I, in.^{4}	51.2		40.6		34.7		28.6		22.1	
r, in.	2.26		2.30		2.32		2.34		2.37	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A1085 material availability before specifying.

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HSS6.625

Table IV-9A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS6.625x									
		0.500		0.432		0.375		0.312		0.280	
t_{des}, in.		0.500		0.432		0.375		0.312		0.280	
lb/ft		32.7		28.6		25.1		21.1		19.0	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	288	433	251	378	220	331	185	279	167	251
	1	287	432	251	377	220	330	185	278	167	251
	2	285	429	249	375	218	328	184	276	166	249
	3	282	424	247	371	216	325	182	273	164	246
	4	278	418	243	365	213	320	179	269	162	243
	5	272	409	238	358	209	314	176	264	159	238
	6	266	399	232	349	204	306	172	258	155	233
	7	258	388	226	339	198	298	167	251	151	227
	8	250	375	219	328	192	289	162	243	146	220
	9	240	361	211	316	185	278	156	235	141	212
	10	230	346	202	303	178	267	150	225	136	204
	11	220	330	193	290	170	255	143	216	130	195
	12	209	314	183	276	162	243	137	205	124	186
	13	197	297	174	261	153	230	130	195	118	177
	14	186	279	164	246	144	217	122	184	111	167
	15	174	262	153	231	136	204	115	173	105	157
	16	162	244	143	215	127	191	108	162	98.1	147
	17	151	227	133	200	118	178	101	151	91.6	138
	18	140	210	123	186	110	165	93.3	140	85.2	128
	19	128	193	114	171	101	152	86.3	130	78.9	119
	20	118	177	105	157	93.0	140	79.5	119	72.7	109
	22	97.7	147	86.9	131	77.5	117	66.4	99.8	60.9	91.6
	24	82.1	123	73.0	110	65.1	97.9	55.8	83.8	51.2	76.9
	26	69.9	105	62.2	93.5	55.5	83.4	47.5	71.4	43.6	65.6
	28	60.3	90.6	53.6	80.6	47.9	71.9	41.0	61.6	37.6	56.5
	30	52.5	79.0	46.7	70.2	41.7	62.7	35.7	53.7	32.8	49.2
	32	46.2	69.4	41.1	61.7	36.6	55.1	31.4	47.2	28.8	43.3
	34	40.9	61.5	36.4	54.7	32.5	48.8	27.8	41.8	25.5	38.3
	36	36.5	54.8	32.4	48.8	29.0	43.5	24.8	37.3	22.8	34.2
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		288	433	251	378	220	331	185	279	167	251
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		234	352	205	307	179	269	151	226	136	204
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		86.4	130	75.4	113	66.1	99.4	55.6	83.6	50.1	75.3
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		46.9	70.5	41.4	62.3	36.7	55.1	30.9	46.5	28.2	42.4
Properties											
Area, in.^{2}		9.62		8.40		7.36		6.19		5.58	
I, in.^{4}		45.4		40.5		36.1		30.9		28.1	
r, in.		2.17		2.19		2.21		2.23		2.25	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A1085 material availability before specifying.

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HSS6.625–
HSS6.000

Table IV-9A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS6.625x				HSS6.000x					
		0.250		0.188		0.500		0.375		0.312	
t_{des}, in.		0.250		0.188		0.500		0.375		0.312	
lb/ft		17.0		12.9		29.4		22.5		19.0	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	**0**	150	225	114	171	259	389	199	298	167	251
	1	150	225	114	171	258	388	198	298	167	250
	2	149	224	113	170	256	385	196	295	165	248
	3	147	221	112	168	252	379	194	291	163	245
	4	145	218	110	166	247	372	190	286	160	241
	5	142	214	108	163	241	363	186	279	157	235
	6	139	209	106	159	234	352	180	271	152	229
	7	136	204	103	155	226	339	174	262	147	221
	8	131	198	99.9	150	217	326	167	252	141	213
	9	127	191	96.6	145	207	311	160	241	135	203
	10	122	183	92.9	140	196	295	152	229	129	193
	11	117	176	89.0	134	185	278	144	216	122	183
	12	111	168	85.0	128	174	261	135	203	115	173
	13	106	159	80.8	121	162	243	127	190	108	162
	14	100	151	76.5	115	150	226	118	177	100	151
	15	94.3	142	72.1	108	139	209	109	164	92.9	140
	16	88.5	133	67.7	102	127	191	101	151	85.7	129
	17	82.7	124	63.4	95.2	116	175	92.1	138	78.7	118
	18	76.9	116	59.0	88.7	105	159	83.9	126	71.8	108
	19	71.3	107	54.8	82.3	95.0	143	75.9	114	65.2	98.0
	20	65.8	98.8	50.6	76.1	85.7	129	68.5	103	58.8	88.4
	22	55.2	82.9	42.6	64.0	70.9	106	56.6	85.1	48.6	73.1
	24	46.4	69.7	35.8	53.8	59.5	89.5	47.6	71.5	40.9	61.4
	26	39.5	59.4	30.5	45.8	50.7	76.2	40.5	60.9	34.8	52.3
	28	34.1	51.2	26.3	39.5	43.7	65.7	35.0	52.5	30.0	45.1
	30	29.7	44.6	22.9	34.4	38.1	57.3	30.5	45.8	26.1	39.3
	32	26.1	39.2	20.1	30.3	33.5	50.3	26.8	40.2	23.0	34.5
	34	23.1	34.7	17.8	26.8						
	36	20.6	31.0	15.9	23.9						
	38			14.3	21.5						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		150	225	114	171	259	389	199	298	167	251
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		122	183	92.6	139	211	316	162	242	136	204
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		45.0	67.6	34.1	51.3	77.6	117	59.6	89.5	50.1	75.3
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		25.4	38.3	19.4	29.2	37.9	57.0	29.7	44.6	25.2	37.9
Properties											
Area, in.^{2}		5.01		3.80		8.64		6.63		5.58	
I, in.^{4}		25.5		19.7		32.9		26.3		22.6	
r, in.		2.26		2.28		1.95		1.99		2.01	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A1085 material availability before specifying.

HSS6.000–
HSS5.563

Table IV-9A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS6.000x						HSS5.563x			
		0.280		0.250		0.188		0.500		0.375	
t_{des}, in.		0.280		0.250		0.188		0.500		0.375	
lb/ft		17.1		15.4		11.7		27.1		20.8	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	151	226	135	203	103	154	238	358	183	275
	1	150	226	135	203	102	154	237	357	182	274
	2	149	224	134	201	102	153	235	353	181	272
	3	147	221	132	199	100	151	231	347	178	267
	4	145	217	130	195	98.7	148	226	340	174	262
	5	141	212	127	191	96.5	145	219	330	169	254
	6	137	206	123	186	93.9	141	212	318	164	246
	7	133	199	119	179	90.9	137	203	305	157	236
	8	128	192	115	173	87.6	132	193	291	150	225
	9	122	184	110	165	84.0	126	183	275	142	214
	10	116	175	105	158	80.1	120	172	258	134	201
	11	110	166	99.3	149	76.1	114	161	241	126	189
	12	104	156	93.7	141	71.8	108	149	224	117	176
	13	97.4	146	87.9	132	67.5	101	137	207	108	163
	14	90.8	137	82.0	123	63.1	94.9	126	189	99.4	149
	15	84.3	127	76.2	114	58.8	88.3	115	172	90.9	137
	16	77.8	117	70.4	106	54.4	81.8	104	156	82.5	124
	17	71.4	107	64.7	97.2	50.1	75.4	93.1	140	74.5	112
	18	65.3	98.1	59.1	88.9	46.0	69.1	83.0	125	66.6	100
	19	59.3	89.2	53.8	80.9	41.9	63.0	74.5	112	59.8	89.9
	20	53.6	80.5	48.6	73.1	38.0	57.1	67.2	101	54.0	81.1
	22	44.3	66.5	40.2	60.4	31.4	47.2	55.6	83.5	44.6	67.1
	24	37.2	55.9	33.8	50.7	26.4	39.6	46.7	70.2	37.5	56.3
	26	31.7	47.6	28.8	43.2	22.5	33.8	39.8	59.8	31.9	48.0
	28	27.3	41.1	24.8	37.3	19.4	29.1	34.3	51.5	27.5	41.4
	30	23.8	35.8	21.6	32.5	16.9	25.4	29.9	44.9	24.0	36.1
	32	20.9	31.4	19.0	28.5	14.8	22.3				
	34					13.1	19.8				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		151	226	135	203	103	154	238	358	183	275
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		123	184	110	165	83.6	125	194	291	149	223
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		45.2	67.9	40.6	61.0	30.8	46.3	71.4	107	54.9	82.5
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		22.9	34.4	20.6	31.0	15.8	23.8	32.2	48.4	25.2	37.9

Properties

	0.280	0.250	0.188	0.500	0.375
Area, in.^{2}	5.03	4.52	3.43	7.95	6.11
I, in.^{4}	20.6	18.7	14.5	25.7	20.7
r, in.	2.02	2.03	2.06	1.80	1.84

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A1085 material availability before specifying.

HSS5.563–
HSS5.500

Table IV-9A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS5.563x				HSS5.500x					
		0.258		0.188		0.500		0.375		0.258	
t_{des}, in.		0.258		0.188		0.500		0.375		0.258	
lb/ft		14.6		10.8		26.7		20.5		14.5	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	129	193	94.9	143	235	353	181	272	127	191
	1	128	193	94.6	142	234	352	180	271	127	191
	2	127	191	93.8	141	232	349	179	268	126	189
	3	125	188	92.5	139	228	343	176	264	124	186
	4	123	184	90.6	136	223	335	172	258	121	182
	5	120	180	88.2	133	216	325	167	251	118	177
	6	116	174	85.5	128	209	313	161	242	114	171
	7	111	167	82.3	124	200	300	155	233	110	165
	8	106	160	78.7	118	190	286	148	222	105	157
	9	101	152	74.9	113	180	270	140	210	99.4	149
	10	95.6	144	70.9	107	169	253	132	198	93.9	141
	11	89.8	135	66.7	100	157	236	123	185	88.0	132
	12	83.8	126	62.4	93.7	146	219	114	172	82.1	123
	13	77.8	117	58.0	87.1	134	201	106	159	76.1	114
	14	71.8	108	53.6	80.5	123	184	97.0	146	70.1	105
	15	65.9	99.0	49.2	74.0	111	167	88.4	133	64.2	96.4
	16	60.1	90.3	45.0	67.6	100	151	80.1	120	58.4	87.7
	17	54.4	81.8	40.9	61.4	89.8	135	72.2	108	52.8	79.4
	18	49.0	73.6	36.9	55.4	80.1	120	64.5	96.9	47.4	71.2
	19	43.9	66.0	33.1	49.7	71.9	108	57.8	86.9	42.5	63.9
	20	39.7	59.6	29.9	44.9	64.9	97.5	52.2	78.5	38.4	57.7
	22	32.8	49.3	24.7	37.1	53.6	80.6	43.1	64.9	31.7	47.7
	24	27.5	41.4	20.7	31.2	45.1	67.7	36.3	54.5	26.6	40.0
	26	23.5	35.3	17.7	26.6	38.4	57.7	30.9	46.4	22.7	34.1
	28	20.2	30.4	15.2	22.9	33.1	49.8	26.6	40.0	19.6	29.4
	30	17.6	26.5	13.3	19.9			23.2	34.9	17.1	25.6
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		129	194	94.9	143	235	353	181	272	127	191
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		105	157	77.3	116	191	287	147	221	104	155
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		38.6	58.1	28.5	42.8	70.5	106	54.3	81.5	38.2	57.4
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		18.1	27.3	13.5	20.4	31.2	46.9	24.6	37.0	17.7	26.6
Properties											
Area, in.^{2}		4.30		3.17		7.85		6.04		4.25	
I, in.^{4}		15.2		11.5		24.8		19.9		14.6	
r, in.		1.88		1.90		1.78		1.82		1.86	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A1085 material availability before specifying.

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HSS5.000

Table IV-9A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS5.000x									
		0.500		0.375		0.312		0.258		0.250	
t_{des}, in.		0.500		0.375		0.312		0.258		0.250	
lb/ft		24.1		18.5		15.6		13.1		12.7	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	212	318	163	245	138	207	115	173	112	168
	1	211	317	163	244	137	206	115	172	111	167
	2	208	313	161	241	136	204	113	170	110	165
	3	204	307	158	237	133	200	111	167	108	162
	4	198	298	153	230	130	195	108	163	105	158
	5	191	287	148	222	125	188	105	157	102	153
	6	183	274	142	213	120	180	101	151	97.6	147
	7	173	260	135	202	114	172	95.8	144	93.0	140
	8	163	245	127	191	108	162	90.6	136	88.0	132
	9	152	228	119	179	101	152	85.0	128	82.6	124
	10	140	211	110	166	94.0	141	79.2	119	76.9	116
	11	129	193	102	153	86.7	130	73.2	110	71.1	107
	12	117	176	92.9	140	79.4	119	67.2	101	65.3	98.1
	13	106	159	84.2	127	72.2	109	61.2	92.0	59.5	89.4
	14	94.5	142	75.8	114	65.1	97.9	55.3	83.2	53.8	80.8
	15	83.9	126	67.6	102	58.3	87.6	49.7	74.6	48.2	72.5
	16	73.8	111	59.8	89.8	51.7	77.7	44.2	66.4	42.9	64.5
	17	65.4	98.3	52.9	79.6	45.8	68.8	39.1	58.8	38.0	57.1
	18	58.3	87.6	47.2	71.0	40.8	61.4	34.9	52.5	33.9	51.0
	19	52.3	78.7	42.4	63.7	36.7	55.1	31.3	47.1	30.4	45.8
	20	47.2	71.0	38.3	57.5	33.1	49.7	28.3	42.5	27.5	41.3
	22	39.0	58.7	31.6	47.5	27.3	41.1	23.4	35.1	22.7	34.1
	24	32.8	49.3	26.6	39.9	23.0	34.5	19.6	29.5	19.1	28.7
	26	27.9	42.0	22.6	34.0	19.6	29.4	16.7	25.2	16.3	24.4
	28							14.4	21.7	14.0	21.1
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		212	318	163	245	138	207	115	173	112	168
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		172	258	133	199	112	168	93.6	140	90.9	136
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		63.5	95.4	49.0	73.6	41.3	62.1	34.5	51.8	33.5	50.4
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		25.4	38.3	20.1	30.2	17.1	25.8	14.5	21.8	14.1	21.2
Properties											
Area, in.^{2}		7.07		5.45		4.60		3.84		3.73	
I, in.^{4}		18.1		14.7		12.7		10.8		10.6	
r, in.		1.60		1.64		1.66		1.68		1.68	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A1085 material availability before specifying.

HSS5.000–HSS4.500

Table IV-9A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS5.000x		HSS4.500x							
		0.188		0.375		0.337		0.237		0.188	
t_{des}, in.		0.188		0.375		0.337		0.237		0.188	
lb/ft		9.67		16.5		15.0		10.8		8.67	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	85.0	128	146	219	132	198	94.9	143	76.3	115
	1	84.7	127	145	218	131	197	94.5	142	76.0	114
	2	83.8	126	143	214	130	195	93.2	140	75.0	113
	3	82.3	124	139	209	126	190	91.0	137	73.3	110
	4	80.2	121	134	202	122	184	88.2	132	71.0	107
	5	77.6	117	129	193	117	176	84.6	127	68.2	103
	6	74.6	112	122	183	111	167	80.4	121	64.9	97.6
	7	71.1	107	114	172	104	157	75.7	114	61.2	92.1
	8	67.3	101	106	159	97.1	146	70.6	106	57.3	86.0
	9	63.3	95.1	97.5	147	89.5	134	65.3	98.1	53.0	79.7
	10	59.1	88.8	88.8	133	81.6	123	59.8	89.9	48.7	73.2
	11	54.7	82.2	80.0	120	73.8	111	54.3	81.6	44.3	66.6
	12	50.3	75.6	71.4	107	66.1	99.3	48.8	73.4	39.9	60.0
	13	45.9	69.0	63.1	94.9	58.6	88.1	43.5	65.4	35.7	53.7
	14	41.6	62.6	55.2	82.9	51.4	77.3	38.4	57.7	31.6	47.5
	15	37.5	56.3	48.1	72.2	44.8	67.4	33.5	50.4	27.7	41.6
	16	33.5	50.3	42.2	63.5	39.4	59.2	29.5	44.3	24.3	36.6
	17	29.6	44.6	37.4	56.2	34.9	52.4	26.1	39.2	21.6	32.4
	18	26.4	39.7	33.4	50.2	31.1	46.8	23.3	35.0	19.2	28.9
	19	23.7	35.7	30.0	45.0	27.9	42.0	20.9	31.4	17.3	25.9
	20	21.4	32.2	27.0	40.6	25.2	37.9	18.9	28.3	15.6	23.4
	22	17.7	26.6	22.3	33.6	20.8	31.3	15.6	23.4	12.9	19.3
	24	14.9	22.4	18.8	28.2	17.5	26.3	13.1	19.7	10.8	16.3
	26	12.7	19.0								
	28	10.9	16.4								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		85.0	128	146	219	132	198	94.9	143	76.3	115
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		69.2	104	118	178	107	161	77.3	116	62.2	93.2
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		25.5	38.3	43.7	65.6	39.6	59.5	28.5	42.8	22.9	34.4
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		10.9	16.4	16.0	24.0	14.6	21.9	10.8	16.2	8.73	13.1
Properties											
Area, in.^{2}		2.84		4.86		4.41		3.17		2.55	
I, in.^{4}		8.24		10.4		9.61		7.23		5.93	
r, in.		1.70		1.46		1.48		1.51		1.53	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A1085 material availability before specifying.

HSS4.000

Table IV-9A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS4.000x									
		0.313		0.250		0.237		0.226		0.220	
t_{des}, in.		0.313		0.250		0.237		0.226		0.220	
lb/ft		12.3		10.0		9.53		9.12		8.89	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	109	163	88.3	133	83.8	126	80.2	121	78.1	117
	1	108	162	87.8	132	83.3	125	79.8	120	77.7	117
	2	106	159	86.2	130	81.9	123	78.4	118	76.3	115
	3	103	155	83.7	126	79.5	119	76.1	114	74.1	111
	4	98.5	148	80.3	121	76.2	115	73.1	110	71.1	107
	5	93.2	140	76.1	114	72.2	109	69.3	104	67.5	101
	6	87.1	131	71.3	107	67.7	102	65.0	97.6	63.3	95.1
	7	80.5	121	66.0	99.2	62.6	94.1	60.2	90.5	58.6	88.1
	8	73.4	110	60.3	90.7	57.3	86.1	55.1	82.9	53.7	80.7
	9	66.1	99.4	54.5	82.0	51.8	77.8	49.9	75.0	48.6	73.0
	10	58.8	88.4	48.7	73.2	46.2	69.5	44.6	67.1	43.5	65.3
	11	51.7	77.8	43.0	64.6	40.8	61.3	39.5	59.3	38.4	57.8
	12	44.9	67.5	37.5	56.3	35.6	53.5	34.5	51.8	33.6	50.5
	13	38.5	57.8	32.2	48.4	30.6	46.0	29.7	44.7	28.9	43.5
	14	33.2	49.9	27.8	41.8	26.4	39.6	25.6	38.5	25.0	37.5
	15	28.9	43.4	24.2	36.4	23.0	34.5	22.3	33.6	21.7	32.7
	16	25.4	38.2	21.3	32.0	20.2	30.4	19.6	29.5	19.1	28.7
	17	22.5	33.8	18.8	28.3	17.9	26.9	17.4	26.1	16.9	25.4
	18	20.1	30.2	16.8	25.3	16.0	24.0	15.5	23.3	15.1	22.7
	19	18.0	27.1	15.1	22.7	14.3	21.5	13.9	20.9	13.6	20.4
	20	16.3	24.4	13.6	20.5	12.9	19.4	12.6	18.9	12.2	18.4
	22			11.3	16.9	10.7	16.1	10.4	15.6	10.1	15.2
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		109	163	88.3	133	83.8	126	80.2	121	78.1	117
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		88.5	133	71.9	108	68.3	102	65.3	98.0	63.6	95.4
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		32.6	49.0	26.5	39.8	25.1	37.8	24.1	36.2	23.4	35.2
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		10.7	16.0	8.78	13.2	8.38	12.6	8.03	12.1	7.86	11.8
Properties											
Area, in.^{2}		3.63		2.95		2.80		2.68		2.61	
I, in.^{4}		6.21		5.20		4.98		4.79		4.68	
r, in.		1.31		1.33		1.33		1.34		1.34	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A1085 material availability before specifying.

HSS4.000–
HSS3.500

Table IV-9A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS4.000x		HSS3.500x							
		0.188		0.313		0.300		0.250		0.216	
t_{des}, in.		0.188		0.313		0.300		0.250		0.216	
lb/ft		7.66		10.7		10.3		8.69		7.58	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	67.4	101	93.7	141	90.4	136	76.3	115	66.8	100
	1	67.0	101	92.9	140	89.7	135	75.7	114	66.2	99.6
	2	65.8	98.9	90.7	136	87.5	132	74.0	111	64.7	97.3
	3	64.0	96.1	87.0	131	84.1	126	71.1	107	62.2	93.5
	4	61.4	92.3	82.1	123	79.4	119	67.2	101	58.9	88.5
	5	58.3	87.6	76.3	115	73.8	111	62.6	94.0	54.9	82.5
	6	54.7	82.2	69.6	105	67.5	102	57.3	86.2	50.4	75.7
	7	50.8	76.3	62.6	94.0	60.8	91.4	51.7	77.7	45.5	68.4
	8	46.5	70.0	55.3	83.1	53.8	80.9	45.9	68.9	40.5	60.8
	9	42.2	63.4	48.1	72.2	46.9	70.5	40.1	60.2	35.4	53.2
	10	37.8	56.8	41.1	61.8	40.2	60.4	34.4	51.8	30.5	45.9
	11	33.5	50.3	34.5	51.8	33.9	50.9	29.1	43.7	25.9	38.9
	12	29.3	44.1	29.0	43.5	28.4	42.8	24.4	36.7	21.8	32.7
	13	25.3	38.1	24.7	37.1	24.2	36.4	20.8	31.3	18.5	27.9
	14	21.8	32.8	21.3	32.0	20.9	31.4	18.0	27.0	16.0	24.0
	15	19.0	28.6	18.5	27.9	18.2	27.4	15.6	23.5	13.9	20.9
	16	16.7	25.1	16.3	24.5	16.0	24.1	13.8	20.7	12.2	18.4
	17	14.8	22.3	14.4	21.7	14.2	21.3	12.2	18.3	10.8	16.3
	18	13.2	19.9	12.9	19.4	12.6	19.0	10.9	16.3	9.67	14.5
	19	11.9	17.8			11.3	17.1	9.75	14.7	8.68	13.0
	20	10.7	16.1								
	22	8.84	13.3								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		67.4	101	93.7	141	90.4	136	76.3	115	66.8	100
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		54.8	82.3	76.3	114	73.6	110	62.2	93.2	54.4	81.5
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		20.2	30.4	28.1	42.3	27.1	40.8	22.9	34.4	20.0	30.1
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		6.81	10.2	7.96	12.0	7.68	11.6	6.61	9.94	5.81	8.74
Properties											
Area, in.^{2}		2.25		3.13		3.02		2.55		2.23	
I, in.^{4}		4.10		4.02		3.89		3.39		3.02	
r, in.		1.35		1.13		1.14		1.15		1.16	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A1085 material availability before specifying.

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HSS3.500–
HSS3.000

Table IV-9A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape	HSS3.500x				HSS3.000x					
	0.203		0.188		0.250		0.216		0.203	
t_{des}, in.	0.203		0.188		0.250		0.216		0.203	
lb/ft	7.15		6.66		7.35		6.43		6.07	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r: 0	62.9	94.5	58.7	88.2	64.7	97.2	56.6	85.0	53.3	80.1
1	62.4	93.8	58.2	87.5	64.0	96.1	56.0	84.1	52.7	79.2
2	61.0	91.6	56.9	85.5	61.9	93.0	54.2	81.5	51.1	76.7
3	58.7	88.2	54.8	82.3	58.5	88.0	51.3	77.2	48.4	72.7
4	55.6	83.6	51.9	78.0	54.2	81.4	47.6	71.5	44.9	67.5
5	51.9	78.0	48.4	72.8	49.1	73.7	43.2	64.9	40.8	61.3
6	47.7	71.6	44.5	66.9	43.4	65.3	38.3	57.6	36.2	54.5
7	43.1	64.8	40.3	60.5	37.6	56.6	33.3	50.1	31.5	47.4
8	38.4	57.8	35.9	53.9	31.9	47.9	28.3	42.6	26.8	40.3
9	33.7	50.7	31.5	47.3	26.4	39.7	23.6	35.4	22.4	33.6
10	29.1	43.8	27.2	40.9	21.5	32.3	19.2	28.9	18.2	27.4
11	24.8	37.3	23.1	34.8	17.7	26.7	15.9	23.9	15.1	22.7
12	20.8	31.3	19.4	29.2	14.9	22.4	13.3	20.1	12.7	19.0
13	17.8	26.7	16.6	24.9	12.7	19.1	11.4	17.1	10.8	16.2
14	15.3	23.0	14.3	21.5	11.0	16.5	9.81	14.7	9.31	14.0
15	13.3	20.0	12.4	18.7	9.55	14.3	8.54	12.8	8.11	12.2
16	11.7	17.6	10.9	16.4	8.39	12.6	7.51	11.3	7.13	10.7
17	10.4	15.6	9.69	14.6						
18	9.26	13.9	8.64	13.0						
19	8.31	12.5	7.76	11.7						
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	62.9	94.5	58.7	88.2	64.7	97.2	56.6	85.1	53.3	80.1
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	51.2	76.8	47.8	71.7	52.7	79.0	46.1	69.1	43.4	65.1
Available Strength in Shear, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	18.9	28.4	17.6	26.5	19.4	29.2	17.0	25.5	16.0	24.0
Available Strength in Flexure, kip-ft	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
	5.51	8.29	5.14	7.73	4.74	7.13	4.19	6.30	3.97	5.96
Properties										
Area, in.^{2}	2.10		1.96		2.16		1.89		1.78	
I, in.^{4}	2.87		2.69		2.06		1.84		1.75	
r, in.	1.17		1.17		0.976		0.987		0.991	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A1085 material availability before specifying.

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HSS3.000–
HSS2.875

Table IV-9A (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS3.000x				HSS2.875x					
		0.188		0.152		0.250		0.203		0.188	
t_{des}, in.		0.188		0.152		0.250		0.203		0.188	
lb/ft		5.65		4.63		7.02		5.80		5.40	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	49.7	74.7	40.7	61.2	61.7	92.7	50.9	76.5	47.6	71.5
	1	49.2	73.9	40.3	60.6	60.9	91.6	50.3	75.6	47.1	70.7
	2	47.6	71.6	39.1	58.7	58.8	88.3	48.6	73.0	45.4	68.3
	3	45.2	67.9	37.1	55.8	55.3	83.1	45.8	68.8	42.9	64.4
	4	41.9	63.0	34.5	51.9	50.8	76.4	42.2	63.4	39.5	59.4
	5	38.1	57.3	31.5	47.3	45.6	68.5	38.0	57.0	35.6	53.5
	6	33.9	51.0	28.1	42.2	39.9	59.9	33.4	50.1	31.3	47.1
	7	29.5	44.4	24.6	36.9	34.1	51.2	28.6	43.0	26.9	40.5
	8	25.2	37.9	21.0	31.6	28.4	42.7	24.0	36.1	22.6	34.0
	9	21.0	31.6	17.6	26.5	23.1	34.7	19.6	29.5	18.6	27.9
	10	17.2	25.8	14.5	21.8	18.7	28.1	15.9	23.9	15.0	22.6
	11	14.2	21.4	12.0	18.0	15.4	23.2	13.2	19.8	12.4	18.7
	12	11.9	17.9	10.1	15.1	13.0	19.5	11.1	16.6	10.4	15.7
	13	10.2	15.3	8.57	12.9	11.1	16.6	9.42	14.2	8.90	13.4
	14	8.77	13.2	7.39	11.1	9.53	14.3	8.12	12.2	7.67	11.5
	15	7.64	11.5	6.44	9.67	8.30	12.5	7.07	10.6	6.69	10.0
	16	6.71	10.1	5.66	8.50						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		49.7	74.7	40.7	61.2	61.7	92.7	50.9	76.5	47.6	71.6
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		40.5	60.7	33.2	49.7	50.2	75.3	41.4	62.2	38.8	58.1
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		14.9	22.4	12.2	18.4	18.5	27.8	15.3	23.0	14.3	21.5
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		3.72	5.59	3.07	4.61	4.32	6.49	3.62	5.44	3.39	5.10
Properties											
Area, in.^{2}		1.66		1.36		2.06		1.70		1.59	
I, in.^{4}		1.65		1.38		1.79		1.53		1.44	
r, in.		0.996		1.01		0.932		0.947		0.952	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A1085 material availability before specifying.

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HSS2.500–
HSS2.375

Table IV-9A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS2.500x				HSS2.375x					
		0.250		0.188		0.250		0.218		0.188	
t_{des}, in.		0.250		0.188		0.250		0.218		0.188	
lb/ft		6.01		4.65		5.68		5.03		4.40	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	53.0	79.6	41.0	61.6	50.0	75.1	44.3	66.6	38.6	58.0
	1	52.1	78.4	40.4	60.7	49.1	73.8	43.5	65.4	38.0	57.0
	2	49.6	74.6	38.5	57.9	46.4	69.8	41.2	62.0	36.0	54.1
	3	45.7	68.7	35.6	53.5	42.4	63.7	37.7	56.7	33.0	49.6
	4	40.7	61.2	31.9	48.0	37.2	56.0	33.3	50.0	29.2	43.9
	5	35.1	52.8	27.7	41.7	31.5	47.4	28.3	42.5	24.9	37.5
	6	29.3	44.1	23.3	35.1	25.8	38.7	23.2	34.9	20.6	30.9
	7	23.7	35.6	19.0	28.6	20.3	30.5	18.4	27.6	16.4	24.6
	8	18.5	27.8	15.0	22.6	15.6	23.4	14.2	21.3	12.7	19.0
	9	14.6	21.9	11.9	17.8	12.3	18.5	11.2	16.8	10.0	15.0
	10	11.8	17.8	9.62	14.5	9.96	15.0	9.06	13.6	8.11	12.2
	11	9.77	14.7	7.95	11.9	8.23	12.4	7.49	11.3	6.70	10.1
	12	8.21	12.3	6.68	10.0	6.92	10.4	6.29	9.46	5.63	8.46
	13	7.00	10.5	5.69	8.55						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		53.0	79.7	41.0	61.7	50.0	75.2	44.3	66.6	38.6	58.1
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		43.1	64.7	33.4	50.1	40.7	61.1	36.1	54.1	31.4	47.2
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		15.9	23.9	12.3	18.5	15.0	22.5	13.3	20.0	11.6	17.4
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		3.17	4.76	2.52	3.79	2.82	4.24	2.54	3.83	2.25	3.38
Properties											
Area, in.^{2}		1.77		1.37		1.67		1.48		1.29	
I, in.^{4}		1.13		0.918		0.955		0.868		0.778	
r, in.		0.800		0.820		0.756		0.766		0.776	

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A1085 material availability before specifying.

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HSS2.375–
HSS1.900

Table IV-9A (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A1085 Gr. A

F_y = 50 ksi
F_u = 65 ksi

Shape		HSS2.375x 0.154		HSS1.900x 0.188	
t_{des}, in.		0.154		0.188	
lb/ft		3.66		3.44	
Design		ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	32.0	48.1	30.2	45.4
	1	31.5	47.3	29.4	44.2
	2	29.9	45.0	27.0	40.6
	3	27.5	41.3	23.4	35.2
	4	24.4	36.7	19.2	28.9
	5	20.9	31.5	14.9	22.4
	6	17.4	26.1	10.9	16.3
	7	13.9	20.9	7.98	12.0
	8	10.8	16.2	6.11	9.18
	9	8.54	12.8	4.83	7.26
	10	6.92	10.4	3.91	5.88
	11	5.72	8.59		
	12	4.80	7.22		
	13	4.09	6.15		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		32.0	48.2	30.2	45.5
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		26.1	39.1	24.6	36.9
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		9.61	14.4	9.07	13.6
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		1.90	2.85	1.38	2.07

Properties

	HSS2.375x0.154	HSS1.900x0.188
Area, in.^{2}	1.07	1.01
I, in.^{4}	0.666	0.375
r, in.	0.787	0.609

Notes: Heavy line indicates L_c/r equal to or greater than 200.
Confirm ASTM A1085 material availability before specifying.

HSS20.000–
HSS16.000

Table IV-9B
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS20.000x				HSS18.000x				HSS16.000x	
		0.500		0.375^f		0.500		0.375^f		0.625	
t_{des}, in.		0.465		0.349		0.465		0.349		0.581	
lb/ft		104		78.7		93.5		70.7		103	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	785	1180	592	890	705	1060	534	803	774	1160
	1	785	1180	592	890	705	1060	534	803	774	1160
	2	784	1180	592	889	704	1060	534	802	773	1160
	3	784	1180	591	888	704	1060	533	801	772	1160
	4	782	1180	590	887	702	1060	532	800	770	1160
	5	781	1170	589	886	701	1050	531	798	768	1150
	6	779	1170	588	884	699	1050	530	796	765	1150
	7	777	1170	586	881	696	1050	528	793	762	1140
	8	775	1160	585	879	694	1040	526	790	758	1140
	9	772	1160	583	876	691	1040	524	787	754	1130
	10	769	1160	580	872	688	1030	521	783	749	1130
	11	766	1150	578	869	684	1030	519	779	744	1120
	12	762	1150	575	865	680	1020	516	775	739	1110
	13	759	1140	572	860	676	1020	512	770	733	1100
	14	754	1130	569	856	671	1010	509	765	726	1090
	15	750	1130	566	851	666	1000	505	759	719	1080
	16	745	1120	563	846	661	994	501	754	712	1070
	17	740	1110	559	840	656	985	497	747	705	1060
	18	735	1100	555	834	650	977	493	741	697	1050
	19	730	1100	551	828	644	968	488	734	688	1030
	20	724	1090	547	821	638	958	484	727	680	1020
	22	712	1070	537	808	624	938	474	712	661	994
	24	698	1050	528	793	610	917	463	696	642	965
	26	684	1030	517	777	595	894	452	679	621	934
	28	670	1010	506	761	579	870	440	661	600	902
	30	654	983	494	743	562	845	427	642	578	868
	32	638	959	482	725	545	819	414	623	555	834
	34	621	933	470	706	527	792	401	602	532	799
	36	604	907	457	686	509	765	387	582	508	764
	38	586	880	443	666	490	737	373	561	484	728
	40	567	853	430	646	471	708	359	539	460	692
	42	549	825	416	625	452	680	345	518	436	656
	44	530	797	402	604	433	651	330	496	413	620
	46	511	768	387	582	414	622	316	474	389	585
	48	492	739	373	561	395	593	301	453	366	550
	50	473	711	359	539	376	564	287	431	344	516
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		785	1180	592	890	705	1060	534	803	774	1160
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		663	994	500	750	595	893	451	677	653	980
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		236	354	178	267	212	318	160	241	232	349
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		406	611	294	442	328	493	242	363	317	476
Properties											
Area, in.^{2}		28.5		21.5		25.6		19.4		28.1	
I, in.^{4}		1360		1040		985		754		838	
r, in.		6.91		6.95		6.20		6.24		5.46	

[f] Shape exceeds the compact limit for flexure for F_y = 46 ksi.

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Table IV-9B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

HSS16.000

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS16.000x									
		0.500		0.438		0.375^f		0.312^f		0.250^f	
t_{des}, in.		0.465		0.407		0.349		0.291		0.233	
lb/ft		82.9		72.9		62.6		52.3		42.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	625	940	548	824	474	712	397	596	317	476
	1	625	939	548	824	474	712	397	596	317	476
	2	624	939	547	823	473	711	396	595	316	476
	3	623	937	547	821	472	710	396	594	316	475
	4	622	935	545	820	471	708	395	593	315	474
	5	620	932	544	817	470	706	394	591	314	472
	6	618	929	542	814	468	704	392	589	313	471
	7	615	925	540	811	466	701	391	587	312	469
	8	613	921	537	807	464	698	389	584	311	467
	9	609	916	534	803	462	694	387	581	309	464
	10	605	910	531	798	459	690	384	578	307	462
	11	601	904	527	793	456	685	382	574	305	459
	12	597	897	524	787	453	680	379	570	303	455
	13	592	890	519	781	449	675	376	565	301	452
	14	587	882	515	774	445	669	373	561	298	448
	15	582	874	510	767	441	663	370	555	295	444
	16	576	866	505	759	437	657	366	550	293	440
	17	570	856	500	751	432	650	362	544	290	435
	18	563	847	494	743	428	643	358	538	286	430
	19	557	837	489	734	423	635	354	532	283	426
	20	550	826	482	725	417	627	350	526	280	420
	22	535	804	470	706	406	611	341	512	272	410
	24	520	781	456	686	395	593	331	497	265	398
	26	503	756	442	664	382	575	321	482	257	386
	28	486	730	427	642	370	555	310	466	248	373
	30	468	704	411	618	356	535	299	449	239	360
	32	450	676	395	594	343	515	287	432	230	346
	34	431	648	379	570	329	494	276	414	221	332
	36	412	620	363	545	314	472	264	397	212	318
	38	393	591	346	520	300	451	252	379	202	304
	40	374	562	329	494	285	429	240	360	193	289
	42	355	533	312	469	271	407	228	342	183	275
	44	336	504	296	444	257	386	216	324	173	261
	46	317	476	279	419	242	364	204	306	164	246
	48	298	448	263	395	228	343	192	289	155	232
	50	280	421	247	371	215	323	181	272	146	219
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		625	940	548	824	474	712	397	596	317	476
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		528	792	463	694	400	600	335	502	267	401
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		188	282	164	247	142	214	119	179	95.0	143
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		257	386	227	342	194	292	158	237	123	184
Properties											
Area, in.^{2}		22.7		19.9		17.2		14.4		11.5	
I, in.^{4}		685		606		526		443		359	
r, in.		5.49		5.51		5.53		5.55		5.58	

^{f} Shape exceeds the compact limit for flexure for F_y = 46 ksi.

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HSS14.000

Table IV-9B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS14.000x									
		0.625		0.500		0.375		0.312[f]		0.250[f]	
t_{des}, in.		0.581		0.465		0.349		0.291		0.233	
lb/ft		89.4		72.2		54.6		45.7		36.8	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	675	1010	545	820	413	621	344	517	278	418
	1	675	1010	545	819	413	621	344	517	278	418
	2	674	1010	544	818	412	620	344	517	278	417
	3	672	1010	543	817	412	619	343	516	277	417
	4	670	1010	542	814	410	617	342	514	276	415
	5	668	1000	540	811	409	615	341	512	275	414
	6	665	999	537	807	407	612	339	510	274	412
	7	661	993	534	803	405	608	337	507	273	410
	8	657	987	531	798	402	605	335	504	271	407
	9	652	980	527	792	400	600	333	501	269	405
	10	646	972	523	786	396	596	330	497	267	401
	11	641	963	518	779	393	591	328	492	265	398
	12	634	953	513	771	389	585	324	488	262	394
	13	628	943	508	763	385	579	321	483	260	390
	14	620	932	502	755	381	572	318	477	257	386
	15	613	921	496	745	376	566	314	472	254	381
	16	605	909	490	736	372	558	310	466	251	377
	17	596	896	483	726	366	551	306	459	247	372
	18	587	883	476	715	361	543	301	453	244	366
	19	578	869	468	704	356	535	297	446	240	361
	20	568	854	461	692	350	526	292	439	236	355
	22	548	824	445	668	338	508	282	424	228	343
	24	527	792	428	643	325	489	272	408	220	330
	26	505	759	410	616	312	469	261	392	211	317
	28	482	724	392	589	298	448	249	375	202	304
	30	459	689	373	561	284	427	238	357	193	290
	32	435	653	354	532	270	406	226	339	183	275
	34	411	617	335	503	256	384	214	321	174	261
	36	387	581	316	474	241	363	202	303	164	246
	38	363	546	296	446	227	341	190	286	154	232
	40	340	510	278	417	213	320	178	268	145	218
	42	316	476	259	389	199	299	167	250	135	203
	44	294	442	241	362	185	278	155	233	126	190
	46	272	409	223	336	172	258	144	217	117	176
	48	250	376	206	309	159	238	133	200	109	163
	50	231	347	190	285	146	220	123	185	100	150
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		675	1010	545	820	413	621	344	518	278	418
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		570	854	460	691	349	523	291	436	235	352
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		202	304	164	246	124	186	103	155	83.5	125
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		241	362	196	294	149	225	123	185	95.5	144
Properties											
Area, in.^{2}		24.5		19.8		15.0		12.5		10.1	
I, in.^{4}		552		453		349		295		239	
r, in.		4.75		4.79		4.83		4.85		4.87	

[f] Shape exceeds the compact limit for flexure for F_y = 46 ksi.

Table IV-9B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

HSS12.750–
HSS10.750

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS12.750x						HSS10.750x			
		0.500		0.375		0.250[f]		0.500		0.375	
t_{des}, in.		0.465		0.349		0.233		0.465		0.349	
lb/ft		65.5		49.6		33.4		54.8		41.6	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	493	741	375	563	252	379	413	621	314	472
	1	493	741	374	563	252	379	413	621	314	472
	2	492	740	374	562	252	378	412	619	313	471
	3	491	738	373	560	251	378	410	617	312	469
	4	489	735	372	559	250	376	408	614	310	467
	5	487	732	370	556	249	375	406	610	308	464
	6	484	728	368	553	248	373	402	605	306	460
	7	481	723	365	549	246	370	399	599	303	456
	8	477	717	363	545	244	367	394	593	300	451
	9	473	711	360	541	242	364	389	585	296	445
	10	468	704	356	535	240	361	384	577	292	439
	11	463	697	353	530	238	357	378	568	288	433
	12	458	688	348	524	235	353	372	559	283	426
	13	452	680	344	517	232	349	365	549	278	418
	14	446	670	339	510	229	344	358	538	273	410
	15	439	660	335	503	226	339	351	527	267	402
	16	432	650	329	495	222	334	343	515	261	393
	17	425	639	324	487	219	329	334	503	255	384
	18	418	628	318	478	215	323	326	490	249	374
	19	410	616	312	470	211	317	317	477	243	365
	20	402	604	306	460	207	311	308	464	236	355
	22	385	578	294	441	199	299	290	436	222	334
	24	367	552	280	422	190	285	271	408	208	313
	26	349	524	267	401	181	272	252	379	194	291
	28	330	496	253	380	171	258	233	350	179	269
	30	311	467	238	358	162	243	214	322	165	248
	32	292	439	224	337	152	229	195	294	151	227
	34	273	410	210	315	143	214	177	267	137	206
	36	254	382	195	294	133	200	160	241	124	187
	38	235	354	181	272	124	186	144	216	112	168
	40	217	327	168	252	115	172	130	195	101	151
	42	200	300	154	232	106	159	118	177	91.4	137
	44	183	274	141	212	96.9	146	107	161	83.2	125
	46	167	251	129	194	88.7	133	98.0	147	76.2	114
	48	153	231	119	178	81.4	122	90.0	135	69.9	105
	50	141	213	109	164	75.1	113	83.0	125	64.5	96.9
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		493	741	375	563	252	379	413	621	314	472
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		416	624	316	474	213	319	349	523	265	398
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		148	222	112	169	75.7	114	124	186	94.2	142
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		161	242	123	185	80.4	121	113	170	86.8	130
Properties											
Area, in.^{2}		17.9		13.6		9.16		15.0		11.4	
I, in.^{4}		339		262		180		199		154	
r, in.		4.35		4.39		4.43		3.64		3.68	

[f] Shape exceeds the compact limit for flexure for F_y = 46 ksi.

Table IV-9B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

HSS10.750–
HSS10.000

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS10.750x		HSS10.000x							
		0.250^f		0.625		0.500		0.375		0.312	
t_{des}, in.		0.233		0.581		0.465		0.349		0.291	
lb/ft		28.1		62.6		50.8		38.6		32.3	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	212	319	474	712	383	575	292	439	245	368
	1	212	319	473	711	383	575	292	438	244	367
	2	212	318	472	710	382	574	291	437	244	366
	3	211	317	470	707	380	571	290	436	243	365
	4	210	315	467	702	378	568	288	433	241	363
	5	208	313	464	697	375	563	286	430	240	360
	6	207	311	459	690	371	558	283	426	237	357
	7	205	308	454	682	367	552	280	421	235	353
	8	203	305	448	674	363	545	277	416	232	349
	9	200	301	442	664	357	537	273	410	229	344
	10	198	297	434	653	352	529	269	404	225	339
	11	195	293	427	641	346	519	264	397	221	333
	12	192	288	418	628	339	509	259	389	217	327
	13	188	283	409	615	332	499	254	381	213	320
	14	185	278	400	601	324	487	248	373	208	313
	15	181	272	390	586	316	476	242	364	203	305
	16	177	266	379	570	308	463	236	355	198	298
	17	173	260	369	554	300	450	230	345	193	290
	18	169	254	358	537	291	437	223	335	187	282
	19	165	248	346	520	282	424	216	325	182	273
	20	160	241	335	503	273	410	209	314	176	264
	22	151	227	311	468	254	382	195	293	164	247
	24	142	213	287	432	235	353	181	272	152	229
	26	132	199	263	396	216	324	166	250	140	211
	28	123	184	240	360	197	296	152	228	128	193
	30	113	170	217	326	179	268	138	207	117	175
	32	104	156	195	293	161	242	124	187	105	158
	34	94.4	142	173	260	143	216	111	167	94.3	142
	36	85.6	129	155	232	128	192	99.3	149	84.1	126
	38	77.0	116	139	208	115	173	89.1	134	75.5	114
	40	69.5	104	125	188	104	156	80.4	121	68.2	102
	42	63.1	94.8	114	171	94.0	141	72.9	110	61.8	92.9
	44	57.4	86.3	103	155	85.6	129	66.5	99.9	56.3	84.7
	46	52.6	79.0	94.7	142	78.3	118	60.8	91.4	51.5	77.5
	48	48.3	72.6	86.9	131	71.9	108	55.8	83.9	47.3	71.1
	50	44.5	66.9	80.1	120	66.3	99.7	51.5	77.3	43.6	65.6
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		212	319	474	712	383	575	292	439	245	368
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		179	269	400	600	323	485	246	370	206	310
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		63.6	95.6	142	214	115	173	87.6	132	73.4	110
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		58.5	87.9	118	178	97.1	146	74.6	112	62.9	94.5
Properties											
Area, in.^{2}		7.70		17.2		13.9		10.6		8.88	
I, in.^{4}		106		191		159		123		105	
r, in.		3.72		3.34		3.38		3.41		3.43	

[f] Shape exceeds the compact limit for flexure for F_y = 46 ksi.

HSS10.000–
HSS9.625

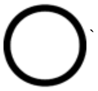
Table IV-9B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS10.000x				HSS9.625x					
		0.250		0.188[f]		0.500		0.375		0.312	
t_{des}, in.		0.233		0.174		0.465		0.349		0.291	
lb/ft		26.1		19.7		48.8		37.1		31.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	197	296	148	222	369	555	281	422	235	353
	1	197	296	148	222	369	554	281	422	235	353
	2	196	295	147	222	368	553	280	421	234	352
	3	196	294	147	221	366	550	279	419	233	350
	4	194	292	146	219	364	547	277	416	232	348
	5	193	290	145	218	361	542	275	413	230	345
	6	191	287	144	216	357	537	272	409	228	342
	7	189	284	142	214	353	530	269	404	225	338
	8	187	281	140	211	348	523	265	399	222	334
	9	184	277	139	208	343	515	261	393	219	329
	10	182	273	136	205	337	506	257	386	215	323
	11	178	268	134	202	330	496	252	379	211	317
	12	175	263	132	198	323	486	247	371	207	311
	13	172	258	129	194	316	475	241	363	202	304
	14	168	252	126	190	308	463	236	354	197	297
	15	164	246	123	186	300	451	229	345	192	289
	16	160	240	120	181	291	438	223	335	187	281
	17	156	234	117	176	283	425	217	326	182	273
	18	151	227	114	171	274	411	210	315	176	265
	19	147	221	111	166	265	398	203	305	170	256
	20	142	214	107	161	255	384	196	295	165	247
	22	133	200	100	151	236	355	182	273	153	230
	24	123	185	93.1	140	217	326	167	251	141	212
	26	114	171	85.9	129	198	297	153	230	129	194
	28	104	156	78.7	118	179	269	139	208	117	176
	30	94.7	142	71.7	108	161	242	125	188	106	159
	32	85.6	129	64.9	97.5	143	216	112	168	94.5	142
	34	76.9	116	58.4	87.7	127	191	99.1	149	83.9	126
	36	68.5	103	52.1	78.3	113	170	88.4	133	74.8	112
	38	61.5	92.5	46.7	70.2	102	153	79.3	119	67.1	101
	40	55.5	83.4	42.2	63.4	91.8	138	71.6	108	60.6	91.1
	42	50.4	75.7	38.3	57.5	83.2	125	64.9	97.6	55.0	82.6
	44	45.9	69.0	34.9	52.4	75.8	114	59.2	88.9	50.1	75.3
	46	42.0	63.1	31.9	47.9	69.4	104	54.1	81.4	45.8	68.9
	48	38.6	57.9	29.3	44.0	63.7	95.8	49.7	74.7	42.1	63.3
	50	35.5	53.4	27.0	40.6	58.7	88.3	45.8	68.9	38.8	58.3
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		197	296	148	222	369	555	281	422	235	353
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		166	249	125	187	312	467	237	356	198	297
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		59.1	88.8	44.4	66.7	111	166	84.3	127	70.5	106
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		51.0	76.6	36.7	55.2	89.5	135	68.9	104	58.3	87.6
Properties											
Area, in.^{2}		7.15		5.37		13.4		10.2		8.53	
I, in.^{4}		85.3		64.8		141		110		93.0	
r, in.		3.45		3.47		3.24		3.28		3.30	

[f] Shape exceeds the compact limit for flexure for F_y = 46 ksi.



HSS9.625–
HSS8.625

Table IV-9B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

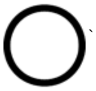
Shape		HSS9.625x				HSS8.625x					
		0.250		0.188[f]		0.625		0.500		0.375	
t_{des}, in.		0.233		0.174		0.581		0.465		0.349	
lb/ft		25.1		19.0		53.5		43.4		33.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	189	284	142	214	405	609	328	493	250	375
	1	189	284	142	214	404	608	327	492	250	375
	2	189	283	142	213	403	606	326	490	249	374
	3	188	282	141	212	401	602	324	488	247	372
	4	187	280	140	211	397	597	322	484	245	369
	5	185	278	139	209	393	591	318	479	243	365
	6	183	276	138	207	388	583	314	473	240	361
	7	181	272	136	205	382	574	310	465	236	355
	8	179	269	135	202	375	564	304	457	232	349
	9	176	265	133	200	368	553	298	448	228	343
	10	173	260	131	196	359	540	292	439	223	335
	11	170	256	128	193	351	527	285	428	218	328
	12	167	251	126	189	341	513	277	417	212	319
	13	163	245	123	185	331	497	269	405	206	310
	14	159	239	120	181	321	482	261	392	200	301
	15	155	233	117	176	310	465	252	380	194	291
	16	151	227	114	171	298	448	244	366	187	281
	17	147	221	111	167	287	431	234	352	180	271
	18	142	214	107	162	275	414	225	338	173	261
	19	138	207	104	156	263	396	216	324	166	250
	20	133	200	101	151	251	378	206	310	159	239
	22	124	186	93.5	141	227	342	187	281	145	217
	24	114	171	86.4	130	204	306	168	253	130	196
	26	104	157	79.2	119	181	272	150	225	117	175
	28	95.0	143	72.1	108	159	239	132	198	103	155
	30	85.8	129	65.2	98.0	138	208	115	173	90.3	136
	32	76.9	116	58.5	88.0	122	183	101	152	79.4	119
	34	68.4	103	52.1	78.3	108	162	89.7	135	70.3	106
	36	61.0	91.7	46.5	69.8	96.2	145	80.0	120	62.7	94.3
	38	54.7	82.3	41.7	62.7	86.3	130	71.8	108	56.3	84.6
	40	49.4	74.2	37.6	56.6	77.9	117	64.8	97.5	50.8	76.3
	42	44.8	67.3	34.1	51.3	70.7	106	58.8	88.4	46.1	69.3
	44	40.8	61.4	31.1	46.7	64.4	96.8	53.6	80.5	42.0	63.1
	46	37.4	56.1	28.5	42.8	58.9	88.5	49.0	73.7	38.4	57.7
	48	34.3	51.6	26.1	39.3			45.0	67.7	35.3	53.0
	50	31.6	47.5	24.1	36.2						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		189	284	142	214	405	609	328	493	250	375
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		160	240	120	180	342	513	277	415	211	316
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		56.8	85.3	42.7	64.2	121	183	98.3	148	74.9	113
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		47.3	71.1	34.1	51.3	86.5	130	71.2	107	54.9	82.5

Properties

Area, in.^{2}	6.87	5.17	14.7	11.9	9.07
I, in.^{4}	75.9	57.7	119	100	77.8
r, in.	3.32	3.34	2.85	2.89	2.93

[f] Shape exceeds the compact limit for flexure for F_y = 46 ksi.

Note: Heavy line indicates L_c/r equal to or greater than 200.



HSS8.625–
HSS7.625

Table IV-9B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS8.625x						HSS7.625x			
		0.322		0.250		0.188[f]		0.375		0.328	
t_{des}, in.		0.300		0.233		0.174		0.349		0.305	
lb/ft		28.6		22.4		17.0		29.1		25.6	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	216	325	169	254	127	191	220	330	193	290
	1	216	325	169	254	127	191	219	330	193	290
	2	215	324	168	253	127	190	219	328	192	289
	3	214	322	167	252	126	189	217	326	191	286
	4	212	319	166	250	125	188	215	323	189	284
	5	210	316	165	247	124	186	212	319	186	280
	6	208	312	163	244	122	184	209	314	183	276
	7	205	308	160	241	121	181	205	308	180	270
	8	201	303	158	237	119	178	200	301	176	265
	9	198	297	155	233	117	175	195	294	172	258
	10	193	291	152	228	114	172	190	286	167	251
	11	189	284	148	223	112	168	184	277	162	244
	12	184	277	144	217	109	164	178	268	157	236
	13	179	269	140	211	106	159	172	258	151	227
	14	174	261	136	205	103	155	165	248	145	219
	15	168	253	132	199	99.7	150	158	238	140	210
	16	163	244	128	192	96.4	145	151	228	133	201
	17	157	236	123	185	93.0	140	144	217	127	191
	18	151	227	118	178	89.6	135	137	206	121	182
	19	145	217	114	171	86.1	129	130	195	115	172
	20	139	208	109	164	82.5	124	123	185	108	163
	22	126	190	99.4	149	75.3	113	109	163	96.0	144
	24	114	171	89.8	135	68.2	102	95.1	143	84.0	126
	26	102	153	80.5	121	61.2	91.9	82.0	123	72.6	109
	28	90.3	136	71.5	107	54.4	81.8	70.7	106	62.6	94.1
	30	79.2	119	62.8	94.4	47.9	72.0	61.6	92.6	54.5	82.0
	32	69.6	105	55.2	83.0	42.1	63.3	54.1	81.4	47.9	72.0
	34	61.7	92.7	48.9	73.5	37.3	56.1	48.0	72.1	42.5	63.8
	36	55.0	82.7	43.6	65.6	33.3	50.0	42.8	64.3	37.9	56.9
	38	49.4	74.2	39.2	58.8	29.9	44.9	38.4	57.7	34.0	51.1
	40	44.6	67.0	35.3	53.1	26.9	40.5	34.7	52.1	30.7	46.1
	42	40.4	60.8	32.0	48.2	24.4	36.7	31.4	47.2	27.8	41.8
	44	36.8	55.4	29.2	43.9	22.3	33.5				
	46	33.7	50.6	26.7	40.2	20.4	30.6				
	48	30.9	46.5	24.5	36.9	18.7	28.1				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		216	325	169	254	127	191	220	330	193	290
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		183	274	143	214	107	161	186	278	163	244
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		64.9	97.5	50.7	76.3	38.2	57.4	65.9	99.1	57.9	87.1
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		47.7	71.8	37.6	56.6	27.8	41.8	42.5	63.8	37.6	56.6
Properties											
Area, in.^{2}		7.85		6.14		4.62		7.98		7.01	
I, in.^{4}		68.1		54.1		41.3		52.9		47.1	
r, in.		2.95		2.97		2.99		2.58		2.59	

[f] Shape exceeds the compact limit for flexure for F_y = 46 ksi.

Note: Heavy line indicates L_c/r equal to or greater than 200.

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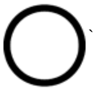


HSS7.500

Table IV-9B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape	HSS7.500x									
	0.500		0.375		0.312		0.250		0.188	
t_{des}, in.	0.465		0.349		0.291		0.233		0.174	
lb/ft	37.4		28.6		24.0		19.4		14.7	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r: 0	284	426	216	325	182	273	147	220	110	166
1	283	426	216	324	181	272	146	220	110	165
2	282	424	215	323	180	271	146	219	110	165
3	280	420	213	320	179	269	145	217	109	163
4	277	416	211	317	177	266	143	215	108	162
5	273	410	208	313	175	263	141	212	106	160
6	268	403	205	307	172	259	139	209	105	157
7	263	395	201	301	169	254	136	205	103	154
8	257	386	196	295	165	248	133	201	100	151
9	250	376	191	287	161	242	130	196	98.0	147
10	243	365	186	279	156	235	127	190	95.4	143
11	235	353	180	270	152	228	123	184	92.5	139
12	227	341	174	261	146	220	119	178	89.5	135
13	218	327	167	251	141	212	114	172	86.3	130
14	209	314	161	241	136	204	110	165	83.0	125
15	200	300	154	231	130	195	105	158	79.6	120
16	190	286	147	220	124	186	101	151	76.1	114
17	181	271	139	210	118	177	95.9	144	72.6	109
18	171	257	132	199	112	168	91.1	137	69.0	104
19	161	243	125	188	106	159	86.3	130	65.4	98.3
20	152	228	118	177	100	150	81.5	123	61.8	92.9
22	133	200	104	156	88.3	133	72.1	108	54.8	82.3
24	115	173	90.3	136	77.0	116	63.0	94.6	48.0	72.1
26	98.6	148	77.5	116	66.2	99.4	54.3	81.5	41.4	62.3
28	85.0	128	66.8	100	57.1	85.7	46.8	70.3	35.7	53.7
30	74.1	111	58.2	87.5	49.7	74.7	40.8	61.3	31.1	46.8
32	65.1	97.8	51.2	76.9	43.7	65.7	35.8	53.8	27.4	41.1
34	57.7	86.7	45.3	68.1	38.7	58.2	31.7	47.7	24.2	36.4
36	51.4	77.3	40.4	60.7	34.5	51.9	28.3	42.5	21.6	32.5
38	46.2	69.4	36.3	54.5	31.0	46.6	25.4	38.2	19.4	29.2
40	41.7	62.6	32.7	49.2	28.0	42.0	22.9	34.5	17.5	26.3
42			29.7	44.6	25.4	38.1	20.8	31.3	15.9	23.9
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	284	426	216	325	182	273	147	220	110	166
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	239	359	182	273	153	230	124	186	93.0	140
Available Strength in Shear, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	85.1	128	64.8	97.4	54.5	81.8	44.0	66.1	33.1	49.7
Available Strength in Flexure, kip-ft	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
	52.8	79.4	41.1	61.8	34.7	52.1	28.2	42.4	21.4	32.2
Properties										
Area, in.^{2}	10.3		7.84		6.59		5.32		4.00	
I, in.^{4}	63.9		50.2		42.9		35.2		26.9	
r, in.	2.49		2.53		2.55		2.57		2.59	

Note: Heavy line indicates L_c/r equal to or greater than 200.

HSS7.000

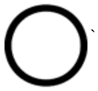
Table IV-9B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS7.000x									
		0.500		0.375		0.312		0.250		0.188	
t_{des}, in.		0.465		0.349		0.291		0.233		0.174	
lb/ft		34.7		26.6		22.3		18.0		13.7	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	263	395	201	302	169	254	136	205	103	154
	1	263	395	200	301	169	253	136	205	103	154
	2	261	393	199	300	168	252	135	204	102	153
	3	259	389	198	297	166	250	134	202	101	152
	4	256	384	195	293	164	247	133	199	100	150
	5	251	378	192	289	162	243	131	196	98.5	148
	6	247	371	189	283	159	239	128	193	96.8	145
	7	241	362	184	277	155	233	125	189	94.7	142
	8	234	352	179	270	151	227	122	184	92.3	139
	9	227	342	174	262	147	221	119	179	89.8	135
	10	220	330	168	253	142	214	115	173	87.0	131
	11	212	318	162	244	137	206	111	167	84.0	126
	12	203	305	156	234	132	198	107	161	80.8	121
	13	194	292	149	224	126	190	102	154	77.5	116
	14	185	278	142	214	120	181	97.8	147	74.1	111
	15	175	264	135	203	115	172	93.1	140	70.6	106
	16	166	249	128	193	109	163	88.3	133	67.0	101
	17	156	235	121	182	103	154	83.5	126	63.4	95.4
	18	147	221	114	171	96.6	145	78.7	118	59.9	90.0
	19	137	206	107	160	90.6	136	73.9	111	56.3	84.6
	20	128	192	99.6	150	84.7	127	69.2	104	52.7	79.2
	22	110	165	85.9	129	73.3	110	60.0	90.2	45.8	68.9
	24	93.1	140	73.0	110	62.4	93.8	51.2	77.0	39.3	59.0
	26	79.4	119	62.2	93.4	53.2	79.9	43.7	65.6	33.5	50.3
	28	68.4	103	53.6	80.6	45.8	68.9	37.6	56.6	28.8	43.4
	30	59.6	89.6	46.7	70.2	39.9	60.0	32.8	49.3	25.1	37.8
	32	52.4	78.8	41.0	61.7	35.1	52.8	28.8	43.3	22.1	33.2
	34	46.4	69.8	36.4	54.6	31.1	46.7	25.5	38.4	19.6	29.4
	36	41.4	62.2	32.4	48.7	27.7	41.7	22.8	34.2	17.4	26.2
	38	37.2	55.8	29.1	43.7	24.9	37.4	20.4	30.7	15.7	23.5
	40									14.1	21.2
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		263	395	201	302	169	254	136	205	103	154
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		222	333	169	254	143	214	115	173	86.7	130
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		78.9	119	60.2	90.5	50.7	76.1	40.9	61.5	30.8	46.3
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		45.7	68.7	35.6	53.5	30.1	45.2	24.6	36.9	18.6	28.0
Properties											
Area, in.^{2}		9.55		7.29		6.13		4.95		3.73	
I, in.^{4}		51.2		40.4		34.6		28.4		21.7	
r, in.		2.32		2.35		2.37		2.39		2.41	

Note: Heavy line indicates L_c/r equal to or greater than 200.



HSS7.000–
HSS6.875

Table IV-9B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS7.000x		HSS6.875x							
		0.125^f		0.500		0.375		0.312		0.250	
t_{des}, in.		0.116		0.465		0.349		0.291		0.233	
lb/ft		9.19		34.1		26.1		21.9		17.7	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	69.1	104	258	388	197	296	166	249	134	201
	1	69.0	104	257	387	197	296	166	249	134	201
	2	68.7	103	256	385	196	294	165	247	133	200
	3	68.1	102	253	381	194	292	163	245	132	198
	4	67.3	101	250	376	192	288	161	242	130	196
	5	66.4	99.7	246	370	188	283	159	238	128	193
	6	65.2	98.0	241	362	185	278	156	234	126	189
	7	63.8	95.9	235	353	180	271	152	228	123	185
	8	62.2	93.6	229	344	176	264	148	222	120	180
	9	60.5	91.0	221	333	170	256	144	216	116	175
	10	58.7	88.2	214	321	164	247	139	209	112	169
	11	56.7	85.2	205	309	158	238	134	201	108	163
	12	54.6	82.1	197	296	152	228	128	193	104	156
	13	52.4	78.8	188	282	145	218	123	184	99.5	150
	14	50.1	75.3	178	268	138	208	117	176	94.9	143
	15	47.8	71.8	169	254	131	197	111	167	90.2	136
	16	45.4	68.3	159	239	124	186	105	158	85.4	128
	17	43.0	64.7	150	225	117	175	99.0	149	80.6	121
	18	40.6	61.1	140	211	110	165	93.0	140	75.8	114
	19	38.2	57.5	131	197	102	154	87.1	131	71.1	107
	20	35.9	53.9	122	183	95.4	143	81.2	122	66.4	99.8
	22	31.3	47.0	104	156	81.9	123	69.9	105	57.3	86.1
	24	26.9	40.4	87.4	131	69.2	104	59.2	89.0	48.6	73.1
	26	22.9	34.4	74.5	112	59.0	88.7	50.5	75.8	41.4	62.3
	28	19.7	29.7	64.2	96.5	50.9	76.5	43.5	65.4	35.7	53.7
	30	17.2	25.8	55.9	84.1	44.3	66.6	37.9	57.0	31.1	46.8
	32	15.1	22.7	49.2	73.9	38.9	58.5	33.3	50.1	27.4	41.1
	34	13.4	20.1	43.5	65.5	34.5	51.9	29.5	44.4	24.2	36.4
	36	11.9	17.9	38.8	58.4	30.8	46.2	26.3	39.6	21.6	32.5
	38	10.7	16.1			27.6	41.5	23.6	35.5	19.4	29.2
	40	9.67	14.5								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		69.1	104	258	388	197	296	166	249	134	201
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		58.4	87.5	218	326	166	250	140	210	113	169
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		20.7	31.2	77.3	116	59.2	88.9	49.7	74.8	40.2	60.4
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		11.9	17.9	43.8	65.9	34.2	51.4	28.9	43.5	23.6	35.5
Properties											
Area, in.^{2}		2.51		9.36		7.16		6.02		4.86	
I, in.^{4}		14.9		48.3		38.2		32.7		26.8	
r, in.		2.43		2.27		2.31		2.33		2.35	

^{f} Shape exceeds the compact limit for flexure for F_y = 46 ksi.

Note: Heavy line indicates L_c/r equal to or greater than 200.

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HSS6.875–
HSS6.625

Table IV-9B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A500 Gr. C
F_y = 46 ksi
F_u = 62 ksi

Shape		HSS6.875x		HSS6.625x							
		0.188		0.500		0.432		0.375		0.312	
t_{des}, in.		0.174		0.465		0.402		0.349		0.291	
lb/ft		13.4		32.7		28.6		25.1		21.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	101	152	248	373	217	325	190	285	159	240
	1	101	151	247	372	216	325	189	284	159	239
	2	100	150	246	370	215	323	188	283	158	238
	3	99.3	149	243	366	213	320	186	280	157	236
	4	98.1	147	240	361	210	315	184	276	155	232
	5	96.6	145	236	354	206	310	180	271	152	228
	6	94.7	142	230	346	201	303	177	265	149	224
	7	92.6	139	224	337	196	295	172	259	145	218
	8	90.3	136	218	327	190	286	167	251	141	212
	9	87.7	132	210	316	184	277	162	243	136	205
	10	84.8	128	202	304	177	266	156	234	131	198
	11	81.8	123	194	291	170	255	149	225	126	190
	12	78.6	118	185	278	162	244	143	215	121	182
	13	75.3	113	176	264	154	232	136	204	115	173
	14	71.9	108	166	250	146	220	129	194	109	164
	15	68.4	103	157	236	138	207	122	183	103	155
	16	64.8	97.4	147	221	130	195	115	172	97.3	146
	17	61.2	92.1	138	207	121	182	107	161	91.3	137
	18	57.7	86.7	128	193	113	170	100	151	85.3	128
	19	54.1	81.3	119	179	105	158	93.2	140	79.4	119
	20	50.6	76.0	110	165	97.2	146	86.3	130	73.7	111
	22	43.8	65.8	92.2	139	82.0	123	73.1	110	62.7	94.2
	24	37.3	56.0	77.5	116	68.9	104	61.4	92.4	52.6	79.1
	26	31.7	47.7	66.0	99.3	58.7	88.3	52.4	78.7	44.9	67.4
	28	27.4	41.1	56.9	85.6	50.6	76.1	45.1	67.9	38.7	58.1
	30	23.8	35.8	49.6	74.6	44.1	66.3	39.3	59.1	33.7	50.6
	32	21.0	31.5	43.6	65.5	38.8	58.3	34.6	51.9	29.6	44.5
	34	18.6	27.9	38.6	58.0	34.4	51.6	30.6	46.0	26.2	39.4
	36	16.6	24.9	34.4	51.8	30.6	46.1	27.3	41.0	23.4	35.2
	38	14.9	22.3								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		101	152	248	373	217	325	190	285	159	240
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		85.1	128	209	314	183	274	160	240	135	202
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		30.2	45.5	74.4	112	65.0	97.6	56.9	85.4	47.8	71.9
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		17.9	26.9	40.6	61.1	35.8	53.8	31.7	47.6	26.9	40.4
Properties											
Area, in.^{2}		3.66		9.00		7.86		6.88		5.79	
I, in.^{4}		20.6		42.9		38.2		34.0		29.1	
r, in.		2.37		2.18		2.20		2.22		2.24	

Note: Heavy line indicates L_c/r equal to or greater than 200.

HSS6.625–
HSS6.000

Table IV-9B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS6.625x								HSS6.000x	
		0.280		0.250		0.188		0.125^f		0.500	
t_{des}, in.		0.260		0.233		0.174		0.116		0.465	
lb/ft		19.0		17.0		12.9		8.69		29.4	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	143	215	129	194	97.2	146	65.3	98.1	223	335
	1	143	215	129	193	97.1	146	65.2	97.9	222	334
	2	142	214	128	192	96.5	145	64.8	97.4	221	332
	3	141	212	127	190	95.6	144	64.2	96.5	218	327
	4	139	209	125	188	94.4	142	63.4	95.3	214	322
	5	137	205	123	185	92.8	139	62.4	93.7	209	314
	6	134	201	120	181	90.9	137	61.1	91.9	204	306
	7	130	196	117	177	88.7	133	59.7	89.7	197	296
	8	127	190	114	172	86.3	130	58.1	87.3	190	285
	9	123	184	111	166	83.6	126	56.3	84.6	182	273
	10	118	178	107	160	80.7	121	54.4	81.7	173	260
	11	114	171	102	154	77.6	117	52.3	78.6	164	247
	12	109	163	98.1	147	74.4	112	50.2	75.4	155	233
	13	104	156	93.6	141	71.0	107	47.9	72.0	146	219
	14	98.4	148	88.9	134	67.5	101	45.6	68.5	136	204
	15	93.1	140	84.1	126	63.9	96.1	43.2	65.0	126	190
	16	87.8	132	79.3	119	60.3	90.7	40.9	61.4	117	176
	17	82.4	124	74.5	112	56.7	85.3	38.5	57.8	108	162
	18	77.1	116	69.7	105	53.2	79.9	36.1	54.2	98.4	148
	19	71.8	108	65.0	97.7	49.6	74.6	33.7	50.7	89.7	135
	20	66.6	100	60.4	90.7	46.1	69.4	31.4	47.2	81.1	122
	22	56.7	85.3	51.5	77.4	39.5	59.3	26.9	40.4	67.0	101
	24	47.7	71.7	43.3	65.1	33.3	50.0	22.7	34.1	56.3	84.6
	26	40.6	61.1	36.9	55.5	28.3	42.6	19.4	29.1	48.0	72.1
	28	35.0	52.7	31.8	47.8	24.4	36.7	16.7	25.1	41.4	62.2
	30	30.5	45.9	27.7	41.7	21.3	32.0	14.5	21.9	36.0	54.2
	32	26.8	40.3	24.4	36.6	18.7	28.1	12.8	19.2	31.7	47.6
	34	23.8	35.7	21.6	32.4	16.6	24.9	11.3	17.0		
	36	21.2	31.9	19.3	28.9	14.8	22.2	10.1	15.2		
	38					13.3	19.9	9.06	13.6		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		143	215	129	194	97.2	146	65.3	98.1	223	335
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		121	181	109	163	82.1	123	55.1	82.7	188	282
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		43.0	64.6	38.7	58.1	29.2	43.8	19.6	29.4	66.9	100
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		24.1	36.2	21.9	32.8	16.6	25.0	10.7	16.1	32.8	49.3
Properties											
Area, in.^{2}		5.20		4.68		3.53		2.37		8.09	
I, in.^{4}		26.4		23.9		18.4		12.6		31.2	
r, in.		2.25		2.26		2.28		2.30		1.96	

^{f} Shape exceeds the compact limit for flexure for F_y = 46 ksi.

Note: Heavy line indicates L_c/r equal to or greater than 200.

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HSS6.000

Table IV-9B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS6.000x									
		0.375		0.312		0.280		0.250		0.188	
t_{des}, in.		0.349		0.291		0.260		0.233		0.174	
lb/ft		22.6		19.0		17.1		15.4		11.7	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	171	257	144	216	129	194	116	175	87.6	132
	1	170	256	143	216	129	194	116	174	87.4	131
	2	169	254	142	214	128	192	115	173	86.8	130
	3	167	251	141	212	126	190	114	171	85.8	129
	4	164	247	138	208	124	187	112	168	84.5	127
	5	161	242	135	204	122	183	110	165	82.7	124
	6	157	235	132	198	119	178	107	161	80.7	121
	7	152	228	128	192	115	173	104	156	78.3	118
	8	146	220	124	186	111	167	100	151	75.7	114
	9	140	211	119	178	107	161	96.3	145	72.8	109
	10	134	201	113	170	102	153	92.1	138	69.7	105
	11	127	191	108	162	97.2	146	87.7	132	66.5	99.9
	12	121	181	102	154	92.1	138	83.1	125	63.1	94.8
	13	113	170	96.3	145	86.8	131	78.4	118	59.6	89.5
	14	106	160	90.3	136	81.5	122	73.7	111	56.0	84.2
	15	99.0	149	84.3	127	76.1	114	68.9	103	52.4	78.8
	16	91.9	138	78.3	118	70.8	106	64.1	96.3	48.8	73.4
	17	84.8	127	72.4	109	65.5	98.4	59.3	89.2	45.3	68.1
	18	77.9	117	66.6	100	60.3	90.7	54.7	82.2	41.8	62.8
	19	71.2	107	61.0	91.7	55.3	83.1	50.2	75.4	38.4	57.8
	20	64.7	97.3	55.6	83.5	50.5	75.8	45.8	68.9	35.2	52.8
	22	53.5	80.4	45.9	69.0	41.7	62.6	37.9	56.9	29.1	43.7
	24	44.9	67.5	38.6	58.0	35.0	52.6	31.8	47.8	24.5	36.8
	26	38.3	57.6	32.9	49.4	29.8	44.9	27.1	40.8	20.8	31.3
	28	33.0	49.6	28.4	42.6	25.7	38.7	23.4	35.1	18.0	27.0
	30	28.8	43.2	24.7	37.1	22.4	33.7	20.4	30.6	15.7	23.5
	32	25.3	38.0	21.7	32.6	19.7	29.6	17.9	26.9	13.8	20.7
	34							15.9	23.8	12.2	18.3
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		171	257	144	216	129	194	116	175	87.6	132
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		144	216	121	182	109	164	98.1	147	73.9	111
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		51.2	77.0	43.1	64.8	38.8	58.2	34.9	52.4	26.3	39.5
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		25.7	38.6	21.8	32.7	19.7	29.6	17.8	26.7	13.6	20.4
Properties											
Area, in.^{2}		6.20		5.22		4.69		4.22		3.18	
I, in.^{4}		24.8		21.3		19.3		17.6		13.5	
r, in.		2.00		2.02		2.03		2.04		2.06	

Note: Heavy line indicates L_c/r equal to or greater than 200.

HSS6.000–
HSS5.563

Table IV-9B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS6.000x		HSS5.563x							
		0.125^f		0.500		0.375		0.258		0.188	
t_{des}, in.		0.116		0.465		0.349		0.240		0.174	
lb/ft		7.85		27.1		20.8		14.6		10.8	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	58.9	88.6	205	308	158	237	110	166	81.3	122
	1	58.8	88.4	205	308	157	236	110	166	81.0	122
	2	58.4	87.8	203	305	156	234	109	164	80.4	121
	3	57.8	86.8	200	300	154	231	108	162	79.3	119
	4	56.9	85.5	196	294	151	226	106	159	77.9	117
	5	55.7	83.8	191	286	147	221	103	155	76.0	114
	6	54.4	81.7	184	277	142	214	100	150	73.8	111
	7	52.8	79.4	178	267	137	206	96.6	145	71.3	107
	8	51.1	76.8	170	255	131	198	92.7	139	68.6	103
	9	49.2	73.9	162	243	125	188	88.5	133	65.5	98.5
	10	47.1	70.8	153	229	119	178	84.0	126	62.3	93.6
	11	45.0	67.6	143	216	112	168	79.3	119	58.9	88.6
	12	42.7	64.2	134	201	105	158	74.4	112	55.4	83.3
	13	40.4	60.7	125	187	97.7	147	69.5	104	51.9	78.0
	14	38.0	57.1	115	173	90.5	136	64.6	97.0	48.3	72.6
	15	35.6	53.5	106	159	83.3	125	59.6	89.6	44.7	67.2
	16	33.2	49.9	96.3	145	76.3	115	54.8	82.3	41.2	61.9
	17	30.9	46.4	87.3	131	69.5	105	50.0	75.2	37.7	56.7
	18	28.5	42.9	78.6	118	63.0	94.7	45.5	68.3	34.4	51.7
	19	26.3	39.5	70.6	106	56.6	85.1	41.0	61.6	31.1	46.8
	20	24.1	36.2	63.7	95.7	51.1	76.8	37.0	55.6	28.1	42.2
	22	20.0	30.0	52.6	79.1	42.2	63.5	30.6	45.9	23.2	34.9
	24	16.8	25.2	44.2	66.5	35.5	53.3	25.7	38.6	19.5	29.3
	26	14.3	21.5	37.7	56.6	30.2	45.4	21.9	32.9	16.6	25.0
	28	12.3	18.5	32.5	48.8	26.1	39.2	18.9	28.4	14.3	21.5
	30	10.7	16.1	28.3	42.5	22.7	34.1	16.4	24.7	12.5	18.8
	32	9.44	14.2								
	34	8.36	12.6								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		58.9	88.6	205	308	158	237	110	166	81.3	122
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		49.8	74.6	173	260	133	199	93.2	140	68.6	103
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		17.7	26.6	61.6	92.5	47.3	71.0	33.1	49.8	24.4	36.6
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		8.91	13.4	27.8	41.7	21.8	32.8	15.6	23.5	11.6	17.4
Properties											
Area, in.^{2}		2.14		7.45		5.72		4.01		2.95	
I, in.^{4}		9.28		24.4		19.5		14.2		10.7	
r, in.		2.08		1.81		1.85		1.88		1.91	

[f] Shape exceeds the compact limit for flexure for F_y = 46 ksi.

Note: Heavy line indicates L_c/r equal to or greater than 200.

HSS5.563–
HSS5.000

Table IV-9B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS5.563x		HSS5.500x						HSS5.000x	
		0.134[f]		0.500		0.375		0.258		0.500	
t_{des}, in.		0.124		0.465		0.349		0.240		0.465	
lb/ft		7.78		26.7		20.6		14.5		24.1	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	**0**	58.4	87.8	203	305	156	234	109	164	182	274
	1	58.2	87.5	202	304	155	233	109	164	182	273
	2	57.8	86.9	200	301	154	231	108	163	180	270
	3	57.0	85.7	197	297	152	228	107	160	176	265
	4	56.0	84.2	193	290	149	223	105	157	172	258
	5	54.7	82.2	188	283	145	218	102	153	166	250
	6	53.1	79.8	182	273	140	211	98.9	149	159	240
	7	51.3	77.2	175	263	135	203	95.3	143	152	228
	8	49.4	74.2	167	251	129	194	91.4	137	144	216
	9	47.2	70.9	159	239	123	185	87.2	131	135	202
	10	44.9	67.5	150	225	117	175	82.6	124	125	189
	11	42.5	63.9	141	211	110	165	77.9	117	116	174
	12	40.0	60.1	131	197	103	154	73.1	110	106	160
	13	37.5	56.3	122	183	95.5	143	68.1	102	97.0	146
	14	34.9	52.4	112	168	88.3	133	63.2	94.9	87.7	132
	15	32.3	48.6	103	154	81.2	122	58.2	87.5	78.7	118
	16	29.8	44.8	93.5	141	74.2	112	53.4	80.3	70.0	105
	17	27.3	41.1	84.6	127	67.5	101	48.7	73.2	62.0	93.2
	18	24.9	37.5	76.0	114	61.0	91.6	44.1	66.3	55.3	83.1
	19	22.6	34.0	68.2	102	54.7	82.2	39.7	59.7	49.6	74.6
	20	20.4	30.7	61.5	92.5	49.4	74.2	35.8	53.9	44.8	67.3
	22	16.9	25.3	50.9	76.4	40.8	61.3	29.6	44.5	37.0	55.6
	24	14.2	21.3	42.7	64.2	34.3	51.5	24.9	37.4	31.1	46.7
	26	12.1	18.1	36.4	54.7	29.2	43.9	21.2	31.9	26.5	39.8
	28	10.4	15.6	31.4	47.2	25.2	37.9	18.3	27.5		
	30	9.06	13.6			21.9	33.0	15.9	23.9		
	32	7.97	12.0								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		58.4	87.8	203	305	156	234	109	164	182	274
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		49.3	73.9	171	257	131	197	92.3	138	154	231
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		17.5	26.3	60.8	91.4	46.7	70.2	32.8	49.3	54.7	82.2
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		8.38	12.6	27.1	40.7	21.3	32.0	15.2	22.9	22.0	33.1
Properties											
Area, in.^{2}		2.12		7.36		5.65		3.97		6.62	
I, in.^{4}		7.84		23.5		18.8		13.7		17.2	
r, in.		1.92		1.79		1.83		1.86		1.61	

[f] Shape exceeds the compact limit for flexure for F_y = 46 ksi.

Note: Heavy line indicates L_c/r equal to or greater than 200.

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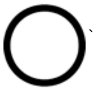

HSS5.000

Table IV-9B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS5.000x									
		0.375		0.312		0.258		0.250		0.188	
t_{des}, in.		0.349		0.291		0.240		0.233		0.174	
lb/ft		18.5		15.6		13.1		12.7		9.67	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	140	211	118	178	98.9	149	96.1	144	72.7	109
	1	140	210	118	177	98.6	148	95.8	144	72.5	109
	2	138	208	117	176	97.6	147	94.8	143	71.8	108
	3	136	204	115	173	95.9	144	93.2	140	70.6	106
	4	133	199	112	168	93.7	141	91.1	137	69.0	104
	5	129	193	109	163	90.8	137	88.3	133	66.9	101
	6	124	186	105	157	87.5	132	85.1	128	64.5	97.0
	7	118	177	99.9	150	83.7	126	81.4	122	61.8	92.9
	8	112	168	94.8	143	79.6	120	77.4	116	58.8	88.4
	9	105	158	89.4	134	75.1	113	73.0	110	55.6	83.6
	10	98.4	148	83.7	126	70.4	106	68.5	103	52.2	78.5
	11	91.3	137	77.8	117	65.6	98.6	63.8	95.9	48.7	73.2
	12	84.2	126	71.8	108	60.7	91.2	59.0	88.7	45.1	67.8
	13	77.0	116	65.9	99.0	55.7	83.8	54.2	81.5	41.5	62.4
	14	69.9	105	60.0	90.1	50.9	76.5	49.5	74.3	38.0	57.1
	15	63.1	94.8	54.2	81.5	46.1	69.3	44.8	67.4	34.5	51.9
	16	56.5	84.9	48.7	73.2	41.5	62.4	40.3	60.6	31.1	46.8
	17	50.1	75.4	43.3	65.1	37.0	55.7	36.0	54.1	27.9	41.9
	18	44.7	67.2	38.6	58.1	33.0	49.6	32.1	48.3	24.9	37.4
	19	40.1	60.3	34.7	52.1	29.6	44.6	28.8	43.3	22.3	33.5
	20	36.2	54.5	31.3	47.0	26.8	40.2	26.0	39.1	20.1	30.3
	22	29.9	45.0	25.9	38.9	22.1	33.2	21.5	32.3	16.6	25.0
	24	25.2	37.8	21.7	32.7	18.6	27.9	18.1	27.1	14.0	21.0
	26	21.4	32.2	18.5	27.8	15.8	23.8	15.4	23.1	11.9	17.9
	28					13.7	20.5	13.3	19.9	10.3	15.4
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		140	211	118	178	98.9	149	96.1	144	72.7	109
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		119	178	100	150	83.5	125	81.1	122	61.4	92.1
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		42.1	63.3	35.5	53.4	29.7	44.6	28.8	43.3	21.8	32.8
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		17.4	26.1	14.8	22.3	12.5	18.8	12.2	18.3	9.30	14.0
Properties											
Area, in.^{2}		5.10		4.30		3.59		3.49		2.64	
I, in.^{4}		13.9		12.0		10.2		9.94		7.69	
r, in.		1.65		1.67		1.69		1.69		1.71	

Note: Heavy line indicates L_c/r equal to or greater than 200.

HSS5.000–
HSS4.500

Table IV-9B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS5.000x		HSS4.500x							
		0.125		0.375		0.337		0.237		0.188	
t_{des}, in.		0.116		0.349		0.313		0.220		0.174	
lb/ft		6.51		16.5		15.0		10.8		8.67	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	**0**	49.0	73.7	125	188	113	171	81.5	123	65.0	97.7
	1	48.9	73.5	125	188	113	170	81.2	122	64.7	97.3
	2	48.4	72.7	123	185	111	168	80.2	121	63.9	96.1
	3	47.6	71.6	120	181	109	164	78.5	118	62.6	94.1
	4	46.6	70.0	117	175	106	159	76.2	115	60.8	91.4
	5	45.2	68.0	112	168	102	153	73.4	110	58.6	88.1
	6	43.6	65.6	107	160	96.8	145	70.1	105	56.0	84.2
	7	41.8	62.9	101	151	91.4	137	66.4	99.8	53.1	79.8
	8	39.9	59.9	94.1	141	85.5	129	62.3	93.7	49.9	75.0
	9	37.7	56.7	87.2	131	79.3	119	58.1	87.3	46.5	69.9
	10	35.5	53.3	80.1	120	72.9	110	53.6	80.6	43.0	64.6
	11	33.1	49.8	72.9	110	66.5	99.9	49.1	73.8	39.4	59.2
	12	30.8	46.2	65.7	98.8	60.0	90.2	44.6	67.0	35.8	53.8
	13	28.4	42.6	58.8	88.3	53.7	80.8	40.1	60.3	32.3	48.6
	14	26.0	39.1	52.1	78.2	47.7	71.7	35.8	53.9	28.9	43.4
	15	23.7	35.6	45.6	68.6	41.9	62.9	31.7	47.7	25.6	38.5
	16	21.4	32.2	40.1	60.3	36.8	55.3	27.9	41.9	22.5	33.9
	17	19.2	28.9	35.5	53.4	32.6	49.0	24.7	37.1	20.0	30.0
	18	17.2	25.8	31.7	47.6	29.1	43.7	22.0	33.1	17.8	26.8
	19	15.4	23.2	28.4	42.7	26.1	39.2	19.8	29.7	16.0	24.0
	20	13.9	20.9	25.7	38.6	23.5	35.4	17.8	26.8	14.4	21.7
	22	11.5	17.3	21.2	31.9	19.5	29.3	14.7	22.2	11.9	17.9
	24	9.65	14.5	17.8	26.8	16.4	24.6	12.4	18.6	10.0	15.0
	26	8.23	12.4								
	28	7.09	10.7								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		49.0	73.7	125	188	113	171	81.5	123	65.0	97.7
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		41.4	62.1	106	159	95.8	144	68.8	103	54.9	82.3
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		14.7	22.1	37.6	56.5	34.0	51.2	24.5	36.8	19.5	29.3
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		6.36	9.56	13.8	20.8	12.6	19.0	9.25	13.9	7.48	11.2
Properties											
Area, in.^{2}		1.78		4.55		4.12		2.96		2.36	
I, in.^{4}		5.31		9.87		9.07		6.79		5.54	
r, in.		1.73		1.47		1.48		1.52		1.53	

Note: Heavy line indicates L_c/r equal to or greater than 200.

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HSS4.500–
HSS4.000

Table IV-9B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A500 Gr. C

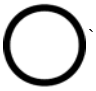
F_y = 46 ksi
F_u = 62 ksi

Shape		HSS4.500x		HSS4.000x							
		0.125		0.313		0.250		0.237		0.226	
t_{des}, in.		0.116		0.291		0.233		0.220		0.210	
lb/ft		5.85		12.3		10.0		9.53		9.12	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	44.1	66.2	93.4	140	76.0	114	71.9	108	68.9	103
	1	43.9	66.0	92.9	140	75.6	114	71.5	107	68.5	103
	2	43.4	65.2	91.3	137	74.4	112	70.4	106	67.4	101
	3	42.5	63.9	88.8	133	72.4	109	68.5	103	65.6	98.6
	4	41.3	62.1	85.4	128	69.6	105	65.9	99.1	63.2	94.9
	5	39.8	59.9	81.3	122	66.3	99.6	62.8	94.4	60.2	90.4
	6	38.1	57.3	76.4	115	62.4	93.8	59.2	89.0	56.7	85.2
	7	36.2	54.4	71.1	107	58.1	87.4	55.2	83.0	52.9	79.5
	8	34.0	51.2	65.4	98.3	53.5	80.5	50.9	76.5	48.8	73.3
	9	31.8	47.8	59.5	89.5	48.8	73.3	46.4	69.8	44.5	66.9
	10	29.4	44.3	53.6	80.5	44.0	66.1	41.9	63.0	40.2	60.3
	11	27.1	40.7	47.7	71.6	39.2	58.9	37.4	56.3	35.9	53.9
	12	24.7	37.1	41.9	63.0	34.6	51.9	33.1	49.7	31.7	47.6
	13	22.3	33.5	36.5	54.8	30.1	45.3	28.9	43.4	27.7	41.6
	14	20.0	30.1	31.5	47.3	26.0	39.1	25.0	37.5	23.9	35.9
	15	17.8	26.7	27.4	41.2	22.6	34.0	21.7	32.7	20.8	31.3
	16	15.7	23.6	24.1	36.2	19.9	29.9	19.1	28.7	18.3	27.5
	17	13.9	20.9	21.3	32.1	17.6	26.5	16.9	25.4	16.2	24.4
	18	12.4	18.6	19.0	28.6	15.7	23.6	15.1	22.7	14.5	21.7
	19	11.1	16.7	17.1	25.7	14.1	21.2	13.6	20.4	13.0	19.5
	20	10.0	15.1	15.4	23.2	12.7	19.1	12.2	18.4	11.7	17.6
	22	8.29	12.5	12.7	19.1	10.5	15.8	10.1	15.2	9.68	14.6
	24	6.97	10.5								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		44.1	66.2	93.4	140	76.0	114	71.9	108	68.9	104
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		37.2	55.8	78.8	118	64.2	96.3	60.7	91.0	58.1	87.2
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		13.2	19.9	28.0	42.1	22.8	34.3	21.6	32.4	20.7	31.1
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		5.12	7.69	9.20	13.8	7.60	11.4	7.23	10.9	6.93	10.4

Properties

	HSS4.500x0.125	HSS4.000x0.313	HSS4.000x0.250	HSS4.000x0.237	HSS4.000x0.226
Area, in.^{2}	1.60	3.39	2.76	2.61	2.50
I, in.^{4}	3.84	5.87	4.91	4.68	4.50
r, in.	1.55	1.32	1.33	1.34	1.34

Note: Heavy line indicates L_c/r equal to or greater than 200.



HSS4.000–
HSS3.500

Table IV-9B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS4.000x						HSS3.500x			
		0.220		0.188		0.125		0.313		0.300	
t_{des}, in.		0.205		0.174		0.116		0.291		0.279	
lb/ft		8.89		7.66		5.18		10.7		10.3	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	67.2	101	57.6	86.5	39.1	58.8	80.7	121	77.7	117
	1	66.8	100	57.3	86.1	38.9	58.5	80.1	120	77.1	116
	2	65.8	98.9	56.4	84.7	38.3	57.6	78.3	118	75.4	113
	3	64.0	96.2	54.9	82.5	37.3	56.1	75.5	113	72.6	109
	4	61.7	92.7	52.9	79.5	36.0	54.1	71.6	108	68.9	104
	5	58.7	88.3	50.4	75.8	34.4	51.7	67.0	101	64.5	96.9
	6	55.3	83.2	47.5	71.5	32.5	48.8	61.7	92.8	59.4	89.3
	7	51.6	77.6	44.4	66.7	30.4	45.7	56.0	84.2	53.9	81.0
	8	47.6	71.5	41.0	61.6	28.1	42.3	50.1	75.3	48.2	72.5
	9	43.4	65.3	37.4	56.3	25.8	38.7	44.1	66.3	42.5	63.8
	10	39.2	58.9	33.8	50.9	23.3	35.1	38.3	57.6	36.9	55.4
	11	35.0	52.6	30.3	45.5	20.9	31.5	32.8	49.2	31.5	47.4
	12	30.9	46.5	26.8	40.2	18.6	28.0	27.6	41.5	26.6	39.9
	13	27.0	40.6	23.4	35.2	16.4	24.6	23.5	35.3	22.6	34.0
	14	23.3	35.1	20.3	30.5	14.2	21.3	20.3	30.5	19.5	29.3
	15	20.3	30.5	17.7	26.6	12.4	18.6	17.7	26.6	17.0	25.6
	16	17.9	26.8	15.5	23.3	10.9	16.3	15.5	23.3	14.9	22.5
	17	15.8	23.8	13.8	20.7	9.63	14.5	13.8	20.7	13.2	19.9
	18	14.1	21.2	12.3	18.4	8.59	12.9	12.3	18.4	11.8	17.7
	19	12.7	19.0	11.0	16.6	7.71	11.6	11.0	16.5	10.6	15.9
	20	11.4	17.2	9.94	14.9	6.95	10.5				
	22	9.45	14.2	8.21	12.3	5.75	8.64				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		67.2	101	57.6	86.5	39.1	58.8	80.7	121	77.7	117
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		56.7	85.1	48.6	72.9	33.0	49.5	68.1	102	65.6	98.3
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		20.2	30.3	17.3	26.0	11.7	17.6	24.2	36.4	23.3	35.0
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		6.79	10.2	5.85	8.80	4.02	6.04	6.89	10.4	6.66	10.0
Properties											
Area, in.^{2}		2.44		2.09		1.42		2.93		2.82	
I, in.^{4}		4.41		3.83		2.67		3.81		3.69	
r, in.		1.34		1.35		1.37		1.14		1.14	

Note: Heavy line indicates L_c/r equal to or greater than 200.

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HSS3.500

Table IV-9B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

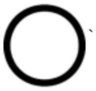
Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS3.500x									
		0.250		0.216		0.203		0.188		0.125	
t_{des}, in.		0.233		0.201		0.189		0.174		0.116	
lb/ft		8.69		7.58		7.15		6.66		4.51	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	65.8	98.9	57.3	86.1	54.3	81.6	50.1	75.3	33.9	50.9
	1	65.4	98.2	56.9	85.5	53.9	81.0	49.8	74.8	33.7	50.6
	2	64.0	96.1	55.7	83.7	52.7	79.3	48.8	73.3	33.0	49.6
	3	61.7	92.7	53.8	80.8	50.9	76.5	47.1	70.8	31.9	47.9
	4	58.7	88.2	51.2	76.9	48.5	72.8	44.9	67.4	30.4	45.7
	5	55.0	82.6	48.0	72.1	45.5	68.3	42.1	63.3	28.6	43.0
	6	50.8	76.4	44.4	66.7	42.1	63.2	39.0	58.7	26.6	40.0
	7	46.3	69.5	40.5	60.9	38.4	57.7	35.7	53.6	24.4	36.6
	8	41.5	62.4	36.4	54.7	34.5	51.9	32.1	48.3	22.0	33.1
	9	36.7	55.2	32.3	48.5	30.6	46.0	28.5	42.9	19.6	29.5
	10	32.0	48.2	28.2	42.4	26.7	40.2	25.0	37.6	17.3	26.0
	11	27.6	41.4	24.3	36.6	23.0	34.6	21.6	32.5	15.0	22.6
	12	23.3	35.0	20.6	31.0	19.5	29.4	18.4	27.6	12.8	19.3
	13	19.9	29.9	17.6	26.4	16.7	25.0	15.7	23.5	10.9	16.4
	14	17.1	25.7	15.2	22.8	14.4	21.6	13.5	20.3	9.43	14.2
	15	14.9	22.4	13.2	19.9	12.5	18.8	11.8	17.7	8.22	12.3
	16	13.1	19.7	11.6	17.4	11.0	16.5	10.3	15.5	7.22	10.9
	17	11.6	17.5	10.3	15.5	9.74	14.6	9.15	13.8	6.40	9.61
	18	10.4	15.6	9.17	13.8	8.69	13.1	8.16	12.3	5.71	8.58
	19	9.30	14.0	8.23	12.4	7.80	11.7	7.33	11.0	5.12	7.70
	20									4.62	6.95
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		65.8	98.9	57.3	86.1	54.3	81.6	50.1	75.3	33.9	50.9
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		55.6	83.4	48.4	72.5	45.8	68.7	42.3	63.5	28.6	42.9
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		19.7	29.7	17.2	25.8	16.3	24.5	15.0	22.6	10.2	15.3
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		5.72	8.59	5.03	7.56	4.75	7.14	4.43	6.66	3.05	4.59
Properties											
Area, in.^{2}		2.39		2.08		1.97		1.82		1.23	
I, in.^{4}		3.21		2.84		2.70		2.52		1.77	
r, in.		1.16		1.17		1.17		1.18		1.20	

Note: Heavy line indicates L_c/r equal to or greater than 200.



HSS3.000

Table IV-9B (continued)

Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS3.000x									
		0.250		0.216		0.203		0.188		0.152	
t_{des}, in.		0.233		0.201		0.189		0.174		0.141	
lb/ft		7.35		6.43		6.07		5.65		4.63	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	55.9	84.0	48.8	73.3	46.0	69.1	42.4	63.8	35.0	52.6
	1	55.4	83.2	48.3	72.6	45.6	68.5	42.0	63.1	34.7	52.1
	2	53.7	80.7	46.9	70.4	44.2	66.5	40.8	61.3	33.7	50.6
	3	51.1	76.8	44.6	67.1	42.1	63.3	38.9	58.4	32.1	48.3
	4	47.6	71.6	41.7	62.6	39.3	59.1	36.3	54.6	30.1	45.2
	5	43.5	65.4	38.1	57.3	36.0	54.2	33.3	50.0	27.6	41.5
	6	38.9	58.5	34.2	51.4	32.4	48.6	29.9	45.0	24.9	37.4
	7	34.2	51.4	30.1	45.2	28.5	42.8	26.4	39.7	22.0	33.0
	8	29.4	44.2	26.0	39.0	24.6	37.0	22.8	34.3	19.1	28.6
	9	24.8	37.3	22.0	33.0	20.9	31.3	19.4	29.1	16.2	24.4
	10	20.4	30.7	18.2	27.3	17.3	26.0	16.1	24.2	13.5	20.3
	11	16.9	25.4	15.0	22.6	14.3	21.5	13.3	20.0	11.2	16.8
	12	14.2	21.3	12.6	19.0	12.0	18.0	11.2	16.8	9.39	14.1
	13	12.1	18.2	10.8	16.2	10.2	15.4	9.51	14.3	8.00	12.0
	14	10.4	15.7	9.28	13.9	8.82	13.3	8.20	12.3	6.90	10.4
	15	9.08	13.6	8.08	12.1	7.69	11.6	7.14	10.7	6.01	9.03
	16	7.98	12.0	7.10	10.7	6.75	10.2	6.28	9.44	5.28	7.94
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		55.9	84.0	48.8	73.3	46.0	69.1	42.4	63.8	35.0	52.6
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		47.2	70.8	41.2	61.7	38.8	58.2	35.8	53.7	29.5	44.3
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		16.8	25.2	14.6	22.0	13.8	20.7	12.7	19.1	10.5	15.8
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		4.11	6.18	3.63	5.45	3.44	5.18	3.19	4.80	2.64	3.97
Properties											
Area, in.^{2}		2.03		1.77		1.67		1.54		1.27	
I, in.^{4}		1.95		1.74		1.66		1.55		1.30	
r, in.		0.982		0.992		0.996		1.00		1.01	

Note: Heavy line indicates L_c/r equal to or greater than 200.

HSS3.000–
HSS2.875

Table IV-9B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS3.000x				HSS2.875x					
		0.134		0.125		0.250		0.203		0.188	
t_{des}, in.		0.124		0.116		0.233		0.189		0.174	
lb/ft		4.11		3.84		7.02		5.80		5.40	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	30.9	46.4	28.9	43.5	53.2	79.9	43.8	65.8	40.8	61.3
	1	30.6	45.9	28.7	43.1	52.6	79.0	43.3	65.1	40.3	60.6
	2	29.7	44.7	27.9	41.9	50.9	76.5	42.0	63.1	39.1	58.7
	3	28.4	42.6	26.6	40.0	48.1	72.4	39.8	59.8	37.1	55.7
	4	26.6	40.0	24.9	37.5	44.6	67.0	36.9	55.5	34.4	51.7
	5	24.4	36.7	22.9	34.4	40.4	60.7	33.5	50.4	31.3	47.0
	6	22.1	33.2	20.7	31.1	35.8	53.8	29.8	44.8	27.9	41.9
	7	19.5	29.4	18.3	27.5	31.0	46.6	25.9	39.0	24.3	36.5
	8	17.0	25.6	15.9	24.0	26.3	39.5	22.1	33.2	20.7	31.1
	9	14.5	21.8	13.6	20.4	21.8	32.8	18.4	27.7	17.3	26.0
	10	12.2	18.3	11.4	17.1	17.7	26.6	15.0	22.6	14.1	21.3
	11	10.1	15.1	9.42	14.2	14.6	22.0	12.4	18.7	11.7	17.6
	12	8.45	12.7	7.92	11.9	12.3	18.5	10.4	15.7	9.83	14.8
	13	7.20	10.8	6.75	10.1	10.5	15.8	8.90	13.4	8.37	12.6
	14	6.21	9.33	5.82	8.74	9.04	13.6	7.67	11.5	7.22	10.8
	15	5.41	8.12	5.07	7.62	7.88	11.8	6.69	10.0	6.29	9.45
	16	4.75	7.14	4.45	6.69						
	17	4.21	6.33	3.95	5.93						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		30.9	46.4	28.9	43.5	53.2	79.9	43.8	65.8	40.8	61.3
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		26.0	39.1	24.4	36.6	44.9	67.3	37.0	55.5	34.4	51.6
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		9.26	13.9	8.68	13.0	15.9	24.0	13.1	19.7	12.2	18.4
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		2.36	3.55	2.22	3.33	3.74	5.62	3.14	4.73	2.92	4.38
Properties											
Area, in.^{2}		1.12		1.05		1.93		1.59		1.48	
I, in.^{4}		1.16		1.09		1.70		1.45		1.35	
r, in.		1.02		1.02		0.938		0.952		0.957	

Note: Heavy line indicates L_c/r equal to or greater than 200.

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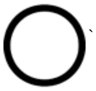

HSS2.875–
HSS2.375

Table IV-9B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS2.875x		HSS2.500x						HSS2.375x	
		0.125		0.250		0.188		0.125		0.250	
t_{des}, in.		0.116		0.233		0.174		0.116		0.233	
lb/ft		3.67		6.01		4.65		3.17		5.68	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	27.8	41.8	45.7	68.7	35.0	52.6	23.9	36.0	43.2	65.0
	1	27.5	41.4	45.0	67.7	34.5	51.8	23.6	35.5	42.5	63.9
	2	26.7	40.1	43.1	64.7	33.0	49.7	22.7	34.1	40.5	60.8
	3	25.4	38.2	40.0	60.1	30.8	46.3	21.2	31.8	37.2	55.9
	4	23.6	35.5	36.0	54.1	27.9	41.9	19.3	28.9	33.1	49.8
	5	21.6	32.4	31.5	47.3	24.5	36.8	17.0	25.6	28.5	42.8
	6	19.3	29.0	26.7	40.2	21.0	31.5	14.7	22.1	23.7	35.7
	7	16.9	25.4	22.0	33.1	17.4	26.2	12.3	18.5	19.1	28.7
	8	14.5	21.8	17.6	26.4	14.1	21.1	10.0	15.1	14.9	22.3
	9	12.2	18.3	13.9	20.9	11.1	16.7	7.98	12.0	11.7	17.7
	10	10.0	15.1	11.3	16.9	9.02	13.6	6.46	9.71	9.52	14.3
	11	8.30	12.5	9.30	14.0	7.46	11.2	5.34	8.03	7.86	11.8
	12	6.97	10.5	7.82	11.7	6.27	9.42	4.49	6.74	6.61	9.93
	13	5.94	8.93	6.66	10.0	5.34	8.02	3.82	5.75		
	14	5.12	7.70					3.30	4.95		
	15	4.46	6.71								
	16	3.92	5.90								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		27.8	41.8	45.7	68.7	35.0	52.6	23.9	36.0	43.2	65.0
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		23.5	35.2	38.6	57.9	29.5	44.3	20.2	30.3	36.5	54.8
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		8.35	12.5	13.7	20.6	10.5	15.8	7.18	10.8	13.0	19.5
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		2.03	3.05	2.75	4.14	2.16	3.25	1.51	2.28	2.46	3.69
Properties											
Area, in.^{2}		1.01		1.66		1.27		0.869		1.57	
I, in.^{4}		0.958		1.08		0.865		0.619		0.910	
r, in.		0.976		0.806		0.825		0.844		0.762	

Note: Heavy line indicates L_c/r equal to or greater than 200.

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HSS2.375–
HSS1.900

Table IV-9B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS2.375x								HSS1.900x	
		0.218		0.188		0.154		0.125		0.188	
t_{des}, in.		0.203		0.174		0.143		0.116		0.174	
lb/ft		5.03		4.40		3.66		3.01		3.44	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	38.3	57.5	33.1	49.7	27.5	41.4	22.7	34.1	26.0	39.0
	1	37.7	56.6	32.5	48.9	27.1	40.8	22.3	33.6	25.3	38.0
	2	35.9	53.9	31.0	46.6	25.9	38.9	21.3	32.1	23.4	35.2
	3	33.1	49.7	28.7	43.1	24.0	36.0	19.8	29.7	20.6	31.0
	4	29.5	44.3	25.6	38.5	21.5	32.3	17.8	26.7	17.2	25.8
	5	25.5	38.3	22.2	33.4	18.7	28.1	15.5	23.3	13.6	20.5
	6	21.3	32.0	18.7	28.0	15.8	23.7	13.1	19.8	10.3	15.4
	7	17.2	25.9	15.2	22.8	12.9	19.4	10.8	16.2	7.55	11.3
	8	13.5	20.3	11.9	17.9	10.2	15.3	8.59	12.9	5.78	8.69
	9	10.6	16.0	9.43	14.2	8.06	12.1	6.79	10.2	4.57	6.86
	10	8.62	13.0	7.64	11.5	6.53	9.82	5.50	8.26	3.70	5.56
	11	7.13	10.7	6.31	9.49	5.40	8.11	4.54	6.83		
	12	5.99	9.00	5.31	7.97	4.54	6.82	3.82	5.74		
	13			4.52	6.79	3.86	5.81	3.25	4.89		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		38.3	57.5	33.1	49.7	27.5	41.4	22.7	34.1	26.0	39.0
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		32.3	48.5	27.9	41.9	23.3	34.9	19.1	28.7	21.9	32.9
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		11.5	17.3	9.92	14.9	8.26	12.4	6.80	10.2	7.79	11.7
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		2.20	3.31	1.94	2.92	1.64	2.46	1.36	2.04	1.19	1.79
Properties											
Area, in.^{2}		1.39		1.20		1.00		0.823		0.943	
I, in.^{4}		0.824		0.733		0.627		0.527		0.355	
r, in.		0.771		0.781		0.791		0.800		0.613	

Note: Heavy line indicates L_c/r equal to or greater than 200.

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HSS1.900–
HSS1.660

Table IV-9B (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Round HSS

A500 Gr. C

F_y = 46 ksi
F_u = 62 ksi

Shape		HSS1.900x				HSS1.660x	
		0.145		0.120		0.140	
t_{des}, in.		0.135		0.111		0.130	
lb/ft		2.72		2.28		2.27	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	20.6	31.0	17.2	25.8	17.2	25.9
	1	20.1	30.3	16.8	25.2	16.7	25.0
	2	18.7	28.1	15.6	23.5	15.1	22.7
	3	16.5	24.8	13.8	20.8	12.8	19.3
	4	13.9	20.9	11.7	17.6	10.2	15.3
	5	11.1	16.7	9.41	14.1	7.57	11.4
	6	8.47	12.7	7.22	10.8	5.34	8.03
	7	6.25	9.40	5.34	8.03	3.93	5.90
	8	4.79	7.19	4.09	6.15	3.01	4.52
	9	3.78	5.68	3.23	4.86	2.37	3.57
	10	3.06	4.60	2.62	3.93		
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		20.6	31.0	17.2	25.8	17.2	25.9
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		17.4	26.1	14.5	21.8	14.5	21.8
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		6.19	9.30	5.16	7.75	5.16	7.76
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		0.966	1.45	0.817	1.23	0.700	1.05

Properties

	HSS1.900x0.145	HSS1.900x0.120	HSS1.660x0.140
Area, in.^{2}	0.749	0.624	0.625
I, in.^{4}	0.293	0.251	0.184
r, in.	0.626	0.634	0.543

Note: Heavy line indicates L_c/r equal to or greater than 200.

PIPE 26–
PIPE 20

Table IV-10
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Pipe

F_y = 35 ksi

Shape		Pipe 26				Pipe 24				Pipe 20	
		x-Strong		Std^f		x-Strong		Std^f		x-Strong	
t_{des}, in.		0.465		0.349		0.465		0.349		0.465	
lb/ft		136		103		126		94.7		104	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	757	1140	591	888	698	1050	545	819	578	869
	1	757	1140	591	888	698	1050	545	819	578	869
	2	756	1140	591	888	698	1050	545	819	578	869
	3	756	1140	591	888	697	1050	544	818	578	868
	4	755	1140	590	887	697	1050	544	818	577	867
	5	755	1130	590	886	696	1050	543	817	576	866
	6	754	1130	589	885	695	1040	543	816	575	865
	7	753	1130	588	884	694	1040	542	815	574	863
	8	752	1130	588	883	693	1040	541	813	573	861
	9	751	1130	587	882	692	1040	540	812	571	859
	10	750	1130	586	880	691	1040	539	810	570	856
	11	748	1120	585	879	689	1040	538	809	568	853
	12	747	1120	583	877	687	1030	537	807	566	850
	13	745	1120	582	875	685	1030	535	805	564	847
	14	743	1120	581	873	684	1030	534	802	561	843
	15	741	1110	579	871	681	1020	532	800	559	840
	16	739	1110	578	868	679	1020	530	797	556	836
	17	737	1110	576	866	677	1020	529	794	553	831
	18	735	1100	574	863	674	1010	527	791	550	827
	19	732	1100	572	860	672	1010	525	788	547	822
	20	730	1100	570	857	669	1010	522	785	544	817
	22	724	1090	566	851	663	996	518	778	537	807
	24	718	1080	561	844	656	987	513	771	529	795
	26	712	1070	556	836	650	976	507	763	521	783
	28	705	1060	551	828	642	965	502	754	513	770
	30	697	1050	545	819	634	953	496	745	503	757
	32	690	1040	539	810	626	941	489	735	494	742
	34	682	1020	533	801	617	928	482	725	484	727
	36	673	1010	526	791	608	914	475	714	474	712
	38	664	998	519	781	599	900	468	703	463	696
	40	655	984	512	770	589	885	460	692	452	679
	42	645	970	505	758	579	870	452	680	441	662
	44	635	955	497	747	568	854	444	668	429	645
	46	625	939	489	735	557	838	436	655	417	627
	48	614	923	481	723	546	821	427	642	405	609
	50	604	907	472	710	535	804	419	629	393	591
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		757	1140	591	888	698	1050	545	819	578	869
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		812	1220	635	952	749	1120	585	878	621	932
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		227	341	177	266	209	315	163	246	174	261
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		513	772	383	576	437	656	333	500	300	452
Properties											
Area, in.^{2}		36.1		28.2		33.3		26.0		27.6	
I, in.^{4}		2950		2320		2310		1820		1320	
r, in.		9.03		9.07		8.33		8.36		6.91	

^{f} Shape exceeds the compact limit for flexure for F_y = 35 ksi.

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PIPE 20–
PIPE 16

Table IV-10
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Pipe

F_y = 35 ksi

Shape		Pipe 20		Pipe 18				Pipe 16			
		Std		x-Strong		Std		x-Strong		Std	
t_{des}, in.		0.349		0.465		0.349		0.465		0.349	
lb/ft		78.7		93.5		70.7		82.9		62.6	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	453	680	520	781	407	611	461	693	360	542
	1	453	680	520	781	407	611	461	693	360	542
	2	452	680	519	781	406	611	461	692	360	541
	3	452	679	519	780	406	610	460	691	360	541
	4	452	679	518	779	405	609	459	690	359	540
	5	451	678	517	777	405	608	458	689	358	539
	6	450	677	516	776	404	607	457	687	357	537
	7	449	675	515	774	403	605	456	685	356	535
	8	448	674	513	772	402	604	454	682	355	534
	9	447	672	512	769	400	602	452	679	354	531
	10	446	670	510	766	399	600	450	676	352	529
	11	444	668	508	763	397	597	448	673	350	526
	12	443	666	506	760	396	595	445	669	348	523
	13	441	663	503	756	394	592	442	665	346	520
	14	439	660	501	752	392	589	440	661	344	517
	15	437	657	498	748	390	586	436	656	341	513
	16	435	654	495	744	387	582	433	651	339	509
	17	433	651	492	739	385	579	430	646	336	505
	18	431	648	489	734	382	575	426	640	333	501
	19	428	644	485	729	380	571	422	635	330	497
	20	426	640	482	724	377	567	418	629	327	492
	22	420	632	474	712	371	558	410	616	321	482
	24	415	623	466	700	365	548	401	602	314	472
	26	408	614	457	687	358	538	391	588	306	460
	28	402	604	447	672	351	527	381	573	298	449
	30	395	593	438	658	343	515	370	557	290	436
	32	387	582	427	642	335	503	359	540	282	423
	34	379	570	417	626	327	491	348	523	273	410
	36	371	558	406	610	318	478	336	505	264	396
	38	363	546	394	593	309	465	324	487	255	383
	40	355	533	383	575	300	451	312	469	245	368
	42	346	520	371	558	291	438	300	451	236	354
	44	337	506	359	540	282	424	288	432	226	340
	46	328	493	347	521	272	409	275	414	216	325
	48	319	479	335	503	263	395	263	395	207	311
	50	309	465	322	484	253	381	251	377	197	297
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		453	680	520	781	407	611	461	693	360	542
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		486	729	558	837	437	655	495	743	387	581
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		136	204	156	234	122	183	138	208	108	163
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		236	354	243	365	190	286	190	286	150	225
Properties											
Area, in.^{2}		21.6		24.8		19.4		22.0		17.2	
I, in.^{4}		1040		956		756		665		527	
r, in.		6.95		6.21		6.24		5.50		5.53	

PIPE 14–
PIPE 12

Table IV-10
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Pipe

F_y = 35 ksi

Shape		Pipe 14				Pipe 12					
		x-Strong		Std		xx-Strong		x-Strong		Std	
t_{des}, in.		0.465		0.349		0.930		0.465		0.349	
lb/ft		72.2		54.6		126		65.5		49.6	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	402	605	314	472	742	1120	367	551	287	432
	1	402	605	314	472	742	1110	367	551	287	431
	2	402	604	314	472	741	1110	366	550	287	431
	3	401	603	313	471	739	1110	365	549	286	430
	4	400	602	313	470	737	1110	364	548	285	429
	5	399	600	312	469	734	1100	363	546	284	427
	6	398	598	311	467	731	1100	362	544	283	426
	7	396	595	310	465	727	1090	360	541	282	424
	8	394	592	308	463	722	1090	358	538	280	421
	9	392	589	306	461	717	1080	355	534	278	418
	10	390	586	305	458	712	1070	353	530	276	415
	11	387	582	303	455	705	1060	350	526	274	412
	12	384	577	300	451	699	1050	347	521	272	408
	13	381	573	298	448	691	1040	343	516	269	405
	14	378	568	295	444	684	1030	340	511	266	400
	15	374	563	293	440	675	1020	336	505	263	396
	16	371	557	290	436	667	1000	332	499	260	391
	17	367	551	287	431	658	988	328	493	257	386
	18	363	545	284	427	648	974	323	486	254	381
	19	358	539	280	422	638	959	319	479	250	376
	20	354	532	277	416	628	943	314	472	246	370
	22	344	518	270	406	606	911	304	457	239	359
	24	334	503	262	394	583	877	293	440	230	346
	26	324	487	254	382	559	841	282	424	222	333
	28	313	470	245	369	535	804	270	406	213	320
	30	301	453	237	356	509	766	258	388	204	306
	32	290	435	227	342	484	727	246	370	194	292
	34	278	417	218	328	458	688	234	351	185	277
	36	265	399	209	314	432	649	221	333	175	263
	38	253	380	199	299	406	610	209	314	165	248
	40	241	362	190	285	380	571	197	296	156	234
	42	228	343	180	271	355	534	185	277	146	220
	44	216	325	171	256	330	497	173	259	137	206
	46	204	306	161	242	306	461	161	242	128	192
	48	192	289	152	228	283	425	150	225	119	179
	50	180	271	143	214	261	392	138	208	110	166
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		402	605	314	473	742	1120	367	551	287	432
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		432	648	338	506	797	1190	394	591	308	462
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		121	181	94.3	142	223	335	110	165	86.1	129
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		144	217	114	171	234	352	123	184	93.8	141
Properties											
Area, in.^{2}		19.2		15.0		35.4		17.5		13.7	
I, in.^{4}		440		350		625		339		262	
r, in.		4.79		4.83		4.20		4.35		4.39	

PIPE 10–
PIPE 8

Table IV-10
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Pipe

F_y = 35 ksi

Shape		Pipe 10						Pipe 8			
		xx-Strong		x-Strong		Std		xx-Strong		x-Strong	
t_{des}, in.		0.930		0.465		0.340		0.816		0.465	
lb/ft		104		54.8		40.5		72.5		43.4	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	604	907	316	476	241	362	419	630	249	375
	1	603	907	316	475	241	362	419	629	249	375
	2	602	905	316	475	240	361	418	628	249	374
	3	600	902	315	473	240	360	416	625	247	372
	4	598	899	314	471	239	359	413	620	246	370
	5	595	894	312	469	238	357	409	615	244	367
	6	591	888	310	466	236	355	405	609	242	363
	7	586	881	308	463	235	353	400	601	239	359
	8	581	873	305	459	233	350	394	593	236	354
	9	575	864	303	455	231	347	388	583	232	349
	10	569	855	299	450	228	343	381	573	228	343
	11	561	844	296	445	226	339	373	561	224	337
	12	554	832	292	439	223	335	365	549	220	330
	13	546	820	288	433	220	330	357	536	215	323
	14	537	807	284	427	217	326	348	523	210	315
	15	528	793	279	420	213	320	338	508	204	307
	16	518	778	274	413	210	315	328	494	199	299
	17	508	763	269	405	206	310	318	478	193	290
	18	497	747	264	397	202	304	308	463	187	282
	19	486	731	259	389	198	298	297	447	181	273
	20	475	714	253	381	194	291	286	430	175	263
	22	452	679	242	363	185	278	264	397	163	245
	24	428	643	230	345	176	265	242	364	150	225
	26	403	605	217	327	167	251	220	331	137	206
	28	378	568	205	308	157	236	198	298	125	188
	30	352	530	192	288	148	222	178	267	113	169
	32	327	492	179	269	138	207	158	237	101	152
	34	302	454	166	250	128	193	140	210	89.7	135
	36	278	418	154	231	119	179	124	187	80.0	120
	38	254	382	142	213	110	165	112	168	71.8	108
	40	231	348	130	195	101	152	101	152	64.8	97.5
	42	210	316	118	178	92.2	139	91.5	137	58.8	88.4
	44	191	288	108	162	84.0	126	83.3	125	53.6	80.5
	46	175	263	98.7	148	76.8	115	76.2	115	49.0	73.7
	48	161	242	90.6	136	70.6	106			45.0	67.7
	50	148	223	83.5	126	65.0	97.7				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		604	907	316	476	241	362	419	630	249	375
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		648	972	340	510	259	388	450	675	268	402
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		181	272	94.9	143	72.3	109	126	189	74.8	112
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		159	239	85.9	129	64.4	96.9	87.2	131	54.1	81.4
Properties											
Area, in.^{2}		28.8		15.1		11.5		20.0		11.9	
I, in.^{4}		354		199		151		154		100	
r, in.		3.51		3.64		3.68		2.78		2.89	

Note: Heavy line indicates L_c/r equal to or greater than 200.

PIPE 8–
PIPE 5

Table IV-10
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Pipe

F_y = 35 ksi

Shape		Pipe 8		Pipe 6						Pipe 5	
		Std		xx-Strong		x-Strong		Std		xx-Strong	
t_{des}, in.		0.300		0.805		0.403		0.261		0.699	
lb/ft		28.6		53.2		28.6		19.0		38.6	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	165	247	308	463	164	247	109	164	224	337
	1	164	247	308	462	164	246	109	164	224	336
	2	164	246	306	460	163	245	108	163	222	334
	3	163	245	303	456	162	243	108	162	219	330
	4	162	244	300	451	160	241	106	160	216	324
	5	161	242	295	444	158	237	105	158	211	317
	6	160	240	290	436	155	233	103	155	205	309
	7	158	237	283	426	152	229	101	153	199	299
	8	156	234	276	415	149	224	99.3	149	192	288
	9	154	231	268	403	145	218	96.9	146	184	277
	10	151	227	260	391	141	212	94.2	142	176	264
	11	148	223	251	377	136	205	91.4	137	167	251
	12	146	219	241	362	132	198	88.4	133	158	237
	13	143	214	231	347	127	191	85.2	128	149	223
	14	139	209	221	332	122	183	81.9	123	139	209
	15	136	204	210	316	116	175	78.5	118	130	195
	16	132	199	199	299	111	167	75.1	113	120	181
	17	129	194	188	283	106	159	71.6	108	111	167
	18	125	188	177	267	100	151	68.0	102	102	153
	19	121	182	167	250	94.7	142	64.4	96.8	93.1	140
	20	117	176	156	234	89.2	134	60.9	91.5	84.5	127
	22	109	164	135	203	78.5	118	53.9	81.0	69.9	105
	24	101	152	115	173	68.3	103	47.1	70.8	58.7	88.2
	26	92.8	139	98.2	148	58.5	88.0	40.6	61.1	50.0	75.2
	28	84.7	127	84.7	127	50.5	75.8	35.0	52.7	43.1	64.8
	30	76.8	115	73.8	111	44.0	66.1	30.5	45.9		
	32	69.1	104	64.8	97.4	38.6	58.1	26.8	40.3		
	34	61.7	92.7	57.4	86.3	34.2	51.4	23.8	35.7		
	36	55.0	82.7			30.5	45.9	21.2	31.9		
	38	49.4	74.2								
	40	44.6	67.0								
	42	40.4	60.8								
	44	36.8	55.4								
	46	33.7	50.6								
	48	30.9	46.5								
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		165	247	308	463	164	247	109	164	224	337
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		177	265	331	496	176	264	117	176	241	361
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		49.4	74.2	92.4	139	49.2	74.0	32.7	49.1	67.3	101
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		36.3	54.6	47.9	71.9	27.2	41.0	18.5	27.8	29.2	43.8

Properties

	Pipe 8 Std	Pipe 6 xx-Strong	Pipe 6 x-Strong	Pipe 6 Std	Pipe 5 xx-Strong
Area, in.^{2}	7.85	14.7	7.83	5.20	10.7
I, in.^{4}	68.1	63.5	38.3	26.5	32.2
r, in.	2.95	2.08	2.20	2.25	1.74

Note: Heavy line indicates L_c/r equal to or greater than 200.

PIPE 5–
PIPE 4

Table IV-10
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Pipe

F_y = 35 ksi

Shape		Pipe 5				Pipe 4					
		x-Strong		Std		xx-Strong		x-Strong		Std	
t_{des}, in.		0.349		0.241		0.628		0.315		0.221	
lb/ft		20.8		14.6		27.6		15.0		10.8	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	120	180	84.0	126	161	241	86.8	130	62.0	93.2
	1	120	180	83.9	126	160	240	86.5	130	61.8	92.9
	2	119	179	83.3	125	158	238	85.6	129	61.2	92.0
	3	118	177	82.5	124	155	233	84.2	127	60.3	90.6
	4	116	174	81.3	122	151	227	82.2	124	58.9	88.5
	5	114	171	79.8	120	146	219	79.8	120	57.2	86.0
	6	111	167	78.0	117	140	210	76.9	116	55.2	83.0
	7	108	162	75.9	114	133	200	73.6	111	52.9	79.6
	8	105	157	73.5	111	126	189	70.0	105	50.4	75.8
	9	101	152	71.0	107	118	177	66.1	99.3	47.7	71.8
	10	96.8	146	68.2	103	110	165	62.0	93.1	44.9	67.5
	11	92.5	139	65.3	98.1	101	152	57.7	86.8	42.0	63.1
	12	88.1	132	62.2	93.6	92.7	139	53.4	80.3	38.9	58.5
	13	83.5	125	59.1	88.8	84.3	127	49.1	73.8	35.9	54.0
	14	78.7	118	55.8	83.9	76.0	114	44.9	67.4	32.9	49.5
	15	74.0	111	52.6	79.0	68.1	102	40.7	61.2	30.0	45.1
	16	69.2	104	49.3	74.1	60.3	90.7	36.7	55.1	27.1	40.8
	17	64.4	96.9	46.0	69.1	53.5	80.3	32.8	49.2	24.4	36.6
	18	59.8	89.8	42.8	64.3	47.7	71.7	29.2	43.9	21.7	32.7
	19	55.2	83.0	39.6	59.5	42.8	64.3	26.2	39.4	19.5	29.3
	20	50.7	76.3	36.5	54.9	38.6	58.0	23.7	35.6	17.6	26.5
	22	42.3	63.6	30.6	45.9	31.9	48.0	19.6	29.4	14.6	21.9
	24	35.5	53.4	25.7	38.6			16.4	24.7	12.2	18.4
	26	30.3	45.5	21.9	32.9						
	28	26.1	39.2	18.9	28.4						
	30	22.7	34.2	16.4	24.7						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		120	180	84.0	126	161	241	86.8	130	62.0	93.2
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		129	193	90.2	135	172	259	93.2	140	66.6	99.9
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		36.0	54.1	25.2	37.9	48.2	72.4	26.0	39.1	18.6	28.0
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		16.6	24.9	11.9	17.9	16.6	24.9	9.66	14.5	7.07	10.6
Properties											
Area, in.^{2}		5.73		4.01		7.66		4.14		2.96	
I, in.^{4}		19.5		14.3		14.7		9.12		6.82	
r, in.		1.85		1.88		1.39		1.48		1.51	

Note: Heavy line indicates L_c/r equal to or greater than 200.

PIPE 3½–
PIPE 3

Table IV-10
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Pipe

F_y = 35 ksi

Shape		Pipe 3½				Pipe 3					
		x-Strong		Std		xx-Strong		x-Strong		Std	
t_{des}, in.		0.296		0.211		0.559		0.280		0.201	
lb/ft		12.5		9.12		18.6		10.3		7.58	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	71.9	108	52.4	78.7	108	163	59.3	89.1	43.4	65.2
	1	71.6	108	52.2	78.4	108	162	59.0	88.6	43.2	64.9
	2	70.7	106	51.5	77.5	106	159	58.0	87.1	42.5	63.8
	3	69.2	104	50.5	75.9	102	154	56.4	84.7	41.3	62.1
	4	67.1	101	49.1	73.7	97.6	147	54.2	81.4	39.8	59.8
	5	64.6	97.0	47.3	71.1	92.0	138	51.5	77.4	37.9	57.0
	6	61.6	92.6	45.2	67.9	85.6	129	48.4	72.7	35.7	53.7
	7	58.2	87.5	42.8	64.4	78.6	118	44.9	67.5	33.3	50.1
	8	54.6	82.1	40.3	60.6	71.2	107	41.3	62.0	30.7	46.2
	9	50.8	76.3	37.6	56.5	63.7	95.7	37.5	56.3	28.0	42.2
	10	46.8	70.3	34.8	52.2	56.2	84.5	33.6	50.6	25.3	38.1
	11	42.8	64.3	31.9	47.9	49.0	73.6	29.9	44.9	22.6	34.0
	12	38.7	58.2	29.0	43.6	42.1	63.3	26.2	39.4	20.0	30.0
	13	34.8	52.3	26.2	39.4	35.9	53.9	22.7	34.1	17.5	26.2
	14	31.0	46.6	23.4	35.2	30.9	46.5	19.6	29.4	15.1	22.7
	15	27.3	41.0	20.8	31.3	26.9	40.5	17.1	25.6	13.1	19.8
	16	24.0	36.1	18.3	27.5	23.7	35.6	15.0	22.5	11.6	17.4
	17	21.3	32.0	16.2	24.4	21.0	31.5	13.3	20.0	10.2	15.4
	18	19.0	28.5	14.5	21.7			11.8	17.8	9.13	13.7
	19	17.0	25.6	13.0	19.5			10.6	16.0	8.19	12.3
	20	15.4	23.1	11.7	17.6						
	22			9.68	14.6						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		71.9	108	52.4	78.8	108	163	59.3	89.1	43.4	65.2
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		77.2	116	56.3	84.4	116	174	63.7	95.5	46.6	69.9
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		21.6	32.4	15.7	23.6	32.5	48.9	17.8	26.7	13.0	19.6
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		7.11	10.7	5.29	7.95	8.54	12.8	5.08	7.64	3.82	5.75
Properties											
Area, in.^{2}		3.43		2.50		5.17		2.83		2.07	
I, in.^{4}		5.94		4.52		5.79		3.70		2.85	
r, in.		1.31		1.34		1.06		1.14		1.17	

Note: Heavy line indicates L_c/r equal to or greater than 200.

PIPE 2½–
PIPE 2

Table IV-10
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Pipe

F_y = 35 ksi

Shape		Pipe 2½						Pipe 2			
		xx-Strong		x-Strong		Std		xx-Strong		x-Strong	
t_{des}, in.		0.514		0.257		0.189		0.406		0.204	
lb/ft		13.7		7.67		5.80		9.04		5.03	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	80.3	121	44.0	66.1	33.7	50.7	52.6	79.1	29.3	44.1
	1	79.5	119	43.6	65.6	33.5	50.3	51.8	77.9	29.0	43.6
	2	77.1	116	42.5	63.9	32.7	49.1	49.6	74.6	27.9	42.0
	3	73.3	110	40.8	61.3	31.4	47.1	46.1	69.3	26.2	39.4
	4	68.3	103	38.4	57.7	29.6	44.5	41.7	62.6	24.1	36.2
	5	62.3	93.7	35.6	53.5	27.5	41.4	36.5	54.9	21.5	32.3
	6	55.8	83.9	32.4	48.7	25.2	37.8	31.1	46.8	18.8	28.2
	7	48.9	73.5	29.0	43.6	22.7	34.0	25.7	38.7	16.0	24.0
	8	42.0	63.2	25.5	38.3	20.1	30.1	20.7	31.1	13.3	19.9
	9	35.4	53.2	22.1	33.2	17.5	26.2	16.4	24.6	10.7	16.1
	10	29.2	43.8	18.8	28.2	15.0	22.5	13.2	19.9	8.69	13.1
	11	24.1	36.2	15.7	23.5	12.6	18.9	10.9	16.5	7.18	10.8
	12	20.2	30.4	13.2	19.8	10.6	15.9			6.03	9.07
	13	17.3	25.9	11.2	16.9	9.01	13.5				
	14	14.9	22.4	9.67	14.5	7.77	11.7				
	15			8.43	12.7	6.77	10.2				
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		80.3	121	44.0	66.2	33.7	50.7	52.6	79.1	29.3	44.1
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		86.2	129	47.3	70.9	36.2	54.3	56.5	84.7	31.5	47.3
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		24.1	36.2	13.2	19.8	10.1	15.2	15.8	23.7	8.80	13.2
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		5.08	7.64	3.09	4.65	2.39	3.60	2.79	4.20	1.68	2.53
Properties											
Area, in.^{2}		3.83		2.10		1.61		2.51		1.40	
I, in.^{4}		2.78		1.83		1.45		1.27		0.827	
r, in.		0.854		0.930		0.952		0.711		0.771	

Note: Heavy line indicates L_c/r equal to or greater than 200.

PIPE 2–
PIPE 1¼

Table IV-10
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Pipe

F_y = 35 ksi

Shape	Pipe 2		Pipe 1½				Pipe 1¼			
	Std		x-Strong		Std		x-Strong		Std	
t_{des}, in.	0.143		0.186		0.135		0.178		0.130	
lb/ft	3.66		3.63		2.72		3.00		2.27	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r: 0	21.4	32.1	21.0	31.5	15.7	23.6	17.5	26.4	13.1	19.7
1	21.1	31.8	20.5	30.9	15.4	23.2	17.1	25.7	12.8	19.2
2	20.4	30.7	19.4	29.1	14.6	21.9	15.8	23.7	11.9	17.8
3	19.2	28.9	17.5	26.4	13.3	19.9	13.8	20.8	10.5	15.7
4	17.7	26.6	15.3	22.9	11.6	17.5	11.5	17.3	8.78	13.2
5	15.9	23.9	12.8	19.2	9.81	14.7	9.06	13.6	7.01	10.5
6	14.0	21.0	10.3	15.4	7.98	12.0	6.77	10.2	5.33	8.01
7	12.0	18.0	7.93	11.9	6.25	9.39	4.97	7.47	3.93	5.90
8	10.1	15.1	6.07	9.12	4.79	7.19	3.81	5.72	3.01	4.52
9	8.22	12.4	4.80	7.21	3.78	5.68			2.37	3.57
10	6.66	10.0	3.88	5.84	3.06	4.60				
11	5.51	8.27								
12	4.63	6.95								
13	3.94	5.92								
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	21.4	32.1	21.0	31.5	15.7	23.6	17.5	26.4	13.1	19.7
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	23.0	34.4	22.5	33.8	16.9	25.3	18.8	28.2	14.1	21.1
Available Strength in Shear, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	6.41	9.64	6.29	9.45	4.71	7.08	5.26	7.91	3.93	5.91
Available Strength in Flexure, kip-ft	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
	1.25	1.87	0.959	1.44	0.735	1.11	0.686	1.03	0.533	0.801
Properties										
Area, in.^{2}	1.02		1.00		0.749		0.837		0.625	
I, in.^{4}	0.627		0.372		0.293		0.231		0.184	
r, in.	0.791		0.610		0.626		0.528		0.543	

Note: Heavy line indicates L_c/r equal to or greater than 200.

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PIPE 1–
PIPE ½

Table IV-10
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Pipe

F_y = 35 ksi

Shape		Pipe 1				Pipe ¾				Pipe ½	
		x-Strong		Std		x-Strong		Std		x-Strong	
t_{des}, in.		0.166		0.124		0.143		0.105		0.137	
lb/ft		2.17		1.68		1.48		1.13		1.09	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	12.6	19.0	9.83	14.8	8.53	12.8	6.54	9.83	6.35	9.54
	1	12.1	18.1	9.43	14.2	7.96	12.0	6.13	9.21	5.66	8.51
	2	10.6	15.9	8.34	12.5	6.45	9.70	5.04	7.57	4.01	6.02
	3	8.50	12.8	6.78	10.2	4.55	6.84	3.63	5.46	2.25	3.38
	4	6.26	9.40	5.09	7.64	2.80	4.22	2.30	3.45	1.27	1.90
	5	4.23	6.35	3.50	5.27	1.79	2.70	1.47	2.21		
	6	2.93	4.41	2.43	3.66						
	7			1.79	2.69						
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		12.6	19.0	9.83	14.8	8.53	12.8	6.54	9.83	6.35	9.54
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
		13.5	20.3	10.6	15.8	9.16	13.7	7.02	10.5	6.82	10.2
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
		3.79	5.69	2.95	4.43	2.56	3.85	1.96	2.95	1.91	2.86
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		0.386	0.580	0.309	0.465	0.208	0.312	0.165	0.247	0.120	0.180
Properties											
Area, in.^{2}		0.602		0.469		0.407		0.312		0.303	
I, in.^{4}		0.101		0.0830		0.0430		0.0350		0.0190	
r, in.		0.410		0.423		0.325		0.336		0.253	

Note: Heavy line indicates L_c/r equal to or greater than 200.

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PIPE ½

Table IV-10
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
Pipe

F_y = 35 ksi

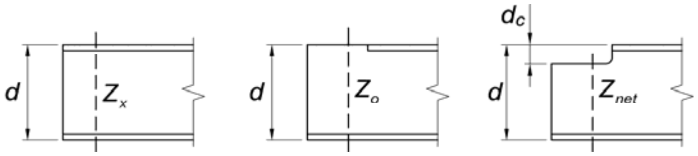
Shape		Pipe ½ Std	
t_{des}, in.		0.101	
lb/ft		0.850	
Design		ASD	LRFD
Available Compressive Strength, kips		P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to the radius of gyration, r	0	4.90	7.37
	1	4.41	6.63
	2	3.21	4.83
	3	1.89	2.84
	4	1.06	1.60
Available Strength in Tensile Yielding, kips		P_n/Ω_t	$\phi_t P_n$
		4.90	7.37
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips		P_n/Ω_t	$\phi_t P_n$
		5.27	7.90
Available Strength in Shear, kips		V_n/Ω_v	$\phi_v V_n$
		1.47	2.21
Available Strength in Flexure, kip-ft		M_n/Ω_b	$\phi_b M_n$
		0.0969	0.146

Properties

Area, in.^{2}	0.234
I, in.^{4}	0.0160
r, in.	0.264

Note: Heavy line indicates L_c/r equal to or greater than 200.

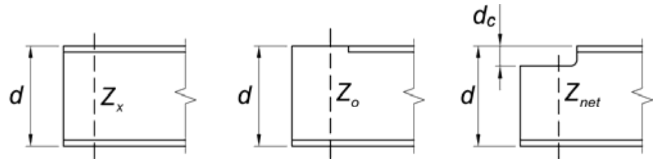
Table IV-11
Plastic Section Modulus for Coped W-Shapes



Shape	d, in.	t_f, in.	Z_x, in.^{3}	Z_o, in.^{3}	Z_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W44×335	44.0	1.77	1620	886	816	781	747	713	680	648	616	585	554
×290	43.6	1.58	1410	746	685	656	627	598	570	542	515	488	461
×262	43.3	1.42	1270	670	615	588	562	536	511	486	461	437	413
×230	42.9	1.22	1100	591	543	519	496	473	451	429	407	385	364
W40×655	43.6	3.54	3080	1640	–	–	1370	1310	1240	1180	1120	1060	996
×593	43.0	3.23	2760	1460	–	–	1220	1160	1100	1050	989	934	880
×503	42.1	2.76	2320	1220	–	1060	1010	961	912	864	817	771	725
×431	41.3	2.36	1960	1020	–	892	849	807	766	725	685	645	606
×397	41.0	2.20	1800	928	–	807	768	729	691	653	617	580	545
×372	40.6	2.05	1680	866	–	752	715	679	643	608	574	540	507
×362	40.6	2.01	1640	839	–	728	693	658	623	589	555	522	490
×324	40.2	1.81	1460	740	674	641	610	578	547	517	487	458	429
×297	39.8	1.65	1330	674	614	584	555	526	498	470	443	416	390
×277	39.7	1.58	1250	609	554	526	499	473	447	421	396	372	348
×249	39.4	1.42	1120	545	494	470	446	422	399	376	353	331	309
×215	39.0	1.22	964	465	422	401	380	359	339	319	300	281	263
×199	38.7	1.07	869	447	407	387	367	348	329	310	292	274	257
W40×392	41.6	2.52	1710	1020	–	893	852	812	773	734	696	659	622
×331	40.8	2.13	1430	848	–	741	706	673	639	607	575	543	513
×327	40.8	2.13	1410	826	–	722	688	655	623	591	559	529	499
×294	40.4	1.93	1270	734	671	640	610	580	551	523	495	467	440
×278	40.2	1.81	1190	702	641	612	583	555	527	500	473	447	421
×264	40.0	1.73	1130	653	597	569	542	516	490	464	439	414	390
×235	39.7	1.58	1010	568	519	494	471	447	424	402	380	358	337
×211	39.4	1.42	906	507	462	441	419	399	378	358	338	319	300
×183	39.0	1.20	774	431	393	375	356	338	321	304	287	270	254
×167	38.6	1.03	693	407	371	354	337	320	303	287	271	256	241
×149	38.2	0.830	598	369	336	320	305	290	275	260	246	232	218

– Indicates that cope depth is less than flange thickness

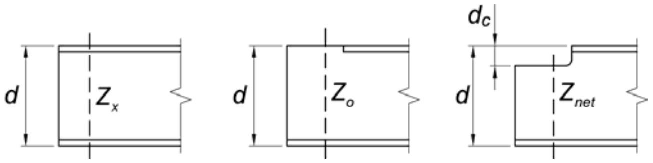
Table IV-11 (continued)
Plastic Section Modulus for Coped W-Shapes



Shape	d, in.	t_f, in.	Z_x, in.^{3}	Z_o, in.^{3}	Z_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W36×925	43.1	4.53	4130	2350	–	–	–	1870	1780	1690	1600	1510	1420
×853	43.1	4.53	3920	2040	–	–	–	1610	1520	1440	1360	1290	1210
×802	42.6	4.29	3660	1890	–	–	–	1490	1410	1330	1260	1190	1120
×723	41.8	3.90	3270	1670	–	–	1380	1310	1240	1170	1110	1040	979
×652	41.1	3.54	2910	1480	–	–	1220	1150	1090	1030	971	913	858
×529	39.8	2.91	2330	1150	–	994	942	891	841	792	745	699	654
×487	39.3	2.68	2130	1050	–	906	858	811	765	719	676	633	592
×441	38.9	2.44	1910	942	–	809	766	723	682	641	601	563	526
×395	38.4	2.20	1710	829	–	710	672	634	597	561	526	492	459
×361	38.0	2.01	1550	749	–	641	606	571	537	504	472	442	412
×330	37.7	1.85	1410	675	609	577	545	514	483	453	424	396	369
×302	37.3	1.68	1280	615	554	524	495	466	438	411	384	358	333
×282	37.1	1.57	1190	571	514	487	459	433	406	381	356	332	309
×262	36.9	1.44	1100	535	482	456	431	406	382	357	334	311	289
×247	36.7	1.35	1030	504	454	430	406	382	359	336	314	293	272
×231	36.5	1.26	963	473	426	404	381	359	337	316	295	275	255
W36×256	37.4	1.73	1040	584	530	503	477	452	427	402	378	354	331
×232	37.1	1.57	936	523	474	450	427	404	381	359	338	316	295
×210	36.7	1.36	833	481	436	414	392	371	350	330	310	291	272
×194	36.5	1.26	767	440	398	378	358	339	320	301	283	265	248
×182	36.3	1.18	718	412	373	354	336	318	300	282	265	248	232
×170	36.2	1.10	668	384	348	330	313	296	279	263	247	231	216
×160	36.0	1.02	624	362	327	311	294	278	262	247	232	217	203
×150	35.9	0.940	581	343	310	294	279	264	249	234	220	206	193
×135	35.6	0.790	509	313	283	269	255	241	227	214	201	189	176
W33×387	36.0	2.28	1560	752	–	636	599	562	526	492	459	427	396
×354	35.6	2.09	1420	681	–	574	540	507	474	443	412	383	355
×318	35.2	1.89	1270	601	537	506	475	445	416	388	361	336	311
×291	34.8	1.73	1160	544	486	457	429	402	375	350	325	302	279
×263	34.5	1.57	1040	487	434	409	384	359	335	312	290	268	248
×241	34.2	1.40	940	455	406	382	359	336	314	292	271	251	231
×221	33.9	1.28	857	417	372	351	329	308	287	267	248	229	211
×201	33.7	1.15	773	380	339	319	300	281	262	244	226	209	192

– Indicates that cope depth is less than flange thickness

Table IV-11 (continued)
Plastic Section Modulus for Coped W-Shapes



Shape	d, in.	t_f, in.	Z_x, in.^{3}	Z_o, in.^{3}	Z_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W33×169	33.8	1.22	629	341	306	289	272	256	240	224	209	194	179
×152	33.5	1.06	559	313	281	266	250	235	221	206	192	178	165
×141	33.3	0.960	514	292	262	247	233	219	205	192	179	166	154
×130	33.1	0.855	467	272	244	230	217	204	191	179	167	155	144
×118	32.9	0.740	415	249	224	211	199	187	176	164	153	143	132
W30×391	33.2	2.44	1450	687	–	570	533	498	463	430	398	367	338
×357	32.8	2.24	1320	616	–	510	476	444	413	383	354	326	300
×326	32.4	2.05	1190	555	–	459	428	399	370	343	316	291	267
×292	32.0	1.85	1060	488	430	402	375	349	323	299	276	254	232
×261	31.6	1.65	943	436	384	358	334	310	287	265	244	224	205
×235	31.3	1.50	847	384	338	315	293	272	252	232	214	196	179
×211	30.9	1.32	751	349	307	287	267	247	229	211	193	177	161
×191	30.7	1.19	675	316	278	260	242	224	207	190	175	160	145
×173	30.4	1.07	607	287	252	235	219	202	187	172	158	144	131
W30×148	30.7	1.18	500	273	242	227	212	198	184	170	157	144	131
×132	30.3	1.00	437	246	218	205	192	179	166	154	142	130	119
×124	30.2	0.930	408	232	206	193	181	168	157	145	134	123	112
×116	30.0	0.850	378	219	194	182	170	159	148	137	126	116	106
×108	29.8	0.760	346	205	181	170	159	148	138	128	118	108	99.2
×99	29.7	0.670	312	190	169	158	148	138	129	119	110	101	92.6
×90	29.5	0.610	283	170	151	141	132	123	115	106	98.1	90.1	82.4
W27×539	32.5	3.54	1890	921	–	–	709	661	614	569	526	485	445
×368	30.4	2.48	1240	582	–	474	440	407	376	346	318	290	264
×336	30.0	2.28	1130	522	–	423	392	363	335	308	282	257	234
×307	29.6	2.09	1030	470	–	380	352	325	299	275	251	229	208
×281	29.3	1.93	936	424	368	342	316	292	268	246	225	204	185
×258	29.0	1.77	852	385	334	310	287	264	243	222	203	184	167
×235	28.7	1.61	772	351	305	282	261	240	221	202	184	167	150
×217	28.4	1.50	711	316	273	253	233	215	197	180	164	148	133
×194	28.1	1.34	631	281	242	224	207	190	174	159	144	130	117
×178	27.8	1.19	570	264	229	212	195	179	164	149	135	122	110
×161	27.6	1.08	515	238	206	191	176	161	147	134	121	109	97.9
×146	27.4	0.975	464	216	187	173	159	146	133	121	109	98.3	88.0

– Indicates that cope depth is less than flange thickness

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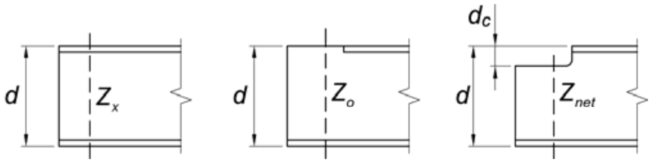
Table IV-11 (continued)
Plastic Section Modulus for Coped W-Shapes

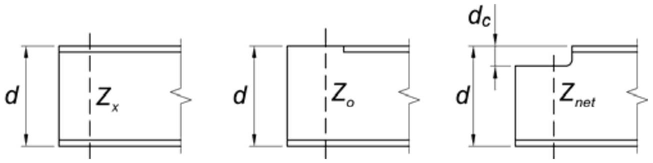


Shape	d, in.	t_f, in.	Z_x, in.^{3}	Z_o, in.^{3}	Z_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W27×129	27.6	1.10	395	209	183	170	157	145	133	122	110	99.5	89.1
×114	27.3	0.930	343	189	165	153	142	131	120	110	100	90.2	80.7
×102	27.1	0.830	305	168	147	136	126	117	107	97.8	88.8	80.1	71.6
×94	26.9	0.745	278	156	136	126	117	108	99.3	90.8	82.5	74.4	66.6
×84	26.7	0.640	244	141	123	115	106	98.2	90.3	82.6	75.1	67.8	60.8
W24×370	28.0	2.72	1130	536	–	428	394	362	332	303	275	249	225
×335	27.5	2.48	1020	473	–	376	346	317	290	264	240	216	194
×306	27.1	2.28	922	423	–	335	308	282	257	234	211	190	171
×279	26.7	2.09	835	380	–	300	275	252	229	208	188	169	151
×250	26.3	1.89	744	333	285	262	240	219	199	180	162	146	130
×229	26.0	1.73	675	302	258	237	217	198	179	162	146	130	116
×207	25.7	1.57	606	269	229	210	192	175	159	143	128	115	102
×192	25.5	1.46	559	248	211	193	176	160	145	131	117	105	92.6
×176	25.2	1.34	511	225	191	175	159	145	131	118	105	93.7	82.7
×162	25.0	1.22	468	209	177	162	148	134	121	109	97.0	86.1	75.9
×146	24.7	1.09	418	188	159	146	133	120	108	97.1	86.6	76.6	67.3
×131	24.5	0.960	370	172	146	134	121	110	98.9	88.6	78.8	69.6	61.1
×117	24.3	0.850	327	154	131	120	109	98.2	88.3	79.0	70.2	61.9	54.1
×104	24.1	0.750	289	138	117	107	97.2	87.8	78.8	70.4	62.4	55.0	48.0
W24×103	24.5	0.980	280	149	128	117	107	97.8	88.4	79.3	70.6	62.3	54.5
×94	24.3	0.875	254	136	117	108	98.6	89.7	81.1	72.8	64.7	57.0	49.9
×84	24.1	0.770	224	122	105	96.2	88.0	80.1	72.5	65.1	57.9	50.9	44.4
×76	23.9	0.680	200	111	95.2	87.6	80.2	73.1	66.1	59.4	52.8	46.6	40.5
×68	23.7	0.585	177	101	86.6	79.7	73.0	66.5	60.2	54.2	48.3	42.7	37.2
W24×62	23.7	0.590	153	96.6	82.9	76.4	70.1	64.1	58.2	52.6	47.1	41.9	36.9
×55	23.6	0.505	134	86.5	74.2	68.4	62.8	57.3	52.1	47.0	42.2	37.5	33.1

– Indicates that cope depth is less than flange thickness

Table IV-11 (continued)

Plastic Section Modulus for Coped W-Shapes



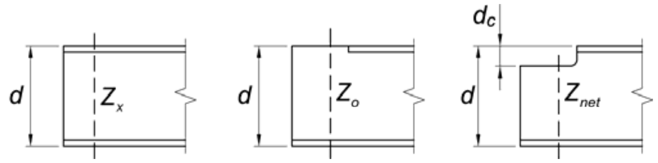
Shape	d, in.	t_f, in.	Z_x, in.^{3}	Z_o, in.^{3}	Z_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W21×275	24.1	2.19	749	324	–	249	226	204	184	165	146	130	114
×248	23.7	1.99	671	285	239	217	197	178	160	143	127	112	97.7
×223	23.4	1.79	601	254	212	193	175	158	141	126	111	97.8	85.3
×201	23.0	1.63	530	225	187	170	154	138	123	110	96.7	84.7	73.5
×182	22.7	1.48	476	201	167	151	136	122	109	96.7	85.1	74.3	64.2
×166	22.5	1.36	432	179	149	135	121	109	96.8	85.7	75.2	65.5	56.5
×147	22.1	1.15	373	166	138	124	112	99.7	88.5	77.9	68.1	59.0	50.6
×132	21.8	1.04	333	147	121	109	97.9	87.3	77.3	67.9	59.2	51.1	43.6
×122	21.7	0.960	307	135	111	100	89.7	79.9	70.7	62.1	54.0	46.6	39.7
×111	21.5	0.875	279	122	100	90.1	80.7	71.8	63.4	55.5	48.2	41.5	35.3
×101	21.4	0.800	253	110	90.4	81.4	72.8	64.7	57.1	50.0	43.4	37.2	31.6
W21×93	21.6	0.930	221	120	101	91.7	82.8	74.2	65.9	57.8	50.2	43.1	36.6
×83	21.4	0.835	196	105	88.4	80.3	72.4	64.8	57.4	50.3	43.6	37.3	31.6
×73	21.2	0.740	172	92.0	77.0	69.8	62.9	56.2	49.7	43.5	37.6	32.1	27.1
×68	21.1	0.685	160	85.9	71.9	65.2	58.7	52.5	46.5	40.6	35.1	29.9	25.2
×62	21.0	0.615	144	78.7	65.9	59.8	53.9	48.2	42.6	37.3	32.2	27.5	23.1
×55	20.8	0.522	126	70.9	59.4	53.9	48.6	43.5	38.6	33.9	29.3	25.0	20.9
×48	20.6	0.430	107	62.9	52.7	47.8	43.2	38.7	34.4	30.2	26.3	22.5	18.9
W21×57	21.1	0.650	129	76.1	64.0	58.2	52.6	47.3	42.1	37.2	32.4	27.8	23.5
×50	20.8	0.535	110	67.3	56.5	51.4	46.4	41.7	37.2	32.8	28.6	24.6	20.8
×44	20.7	0.450	95.4	60.0	50.4	45.8	41.4	37.2	33.2	29.3	25.6	22.1	18.7
W18×311	22.3	2.74	754	336	–	252	227	204	182	161	142	124	107
×283	21.9	2.50	676	300	–	224	202	180	160	142	124	108	93.3
×258	21.5	2.30	611	267	–	198	178	159	141	124	108	93.6	80.4
×234	21.1	2.11	549	235	–	174	155	138	122	107	93.1	80.3	68.6
×211	20.7	1.91	490	208	170	153	136	121	106	92.8	80.4	69.0	58.6
×192	20.4	1.75	442	184	150	135	120	106	93.0	81.0	69.9	59.7	50.5
×175	20.0	1.59	398	165	134	120	106	93.6	81.8	71.0	60.9	51.7	43.4
×158	19.7	1.44	356	147	119	106	93.6	82.2	71.7	61.9	52.9	44.7	
×143	19.5	1.32	322	130	105	93.6	82.7	72.6	63.1	54.4	46.4	39.1	
×130	19.3	1.20	290	118	94.7	84.2	74.3	65.1	56.5	48.5	41.2	34.6	
×119	19.0	1.06	262	112	89.5	79.3	69.8	60.9	52.7	45.0	38.0	31.7	
×106	18.7	0.940	230	98.0	78.2	69.1	60.7	52.8	45.5	38.7	32.5	26.9	
×97	18.6	0.870	211	88.3	70.4	62.2	54.5	47.4	40.7	34.7	29.1	24.0	
×86	18.4	0.770	186	77.9	61.9	54.6	47.8	41.4	35.6	30.1	25.2	20.7	
×76	18.2	0.680	163	67.8	53.7	47.3	41.3	35.7	30.6	25.9	21.5	17.6	

– Indicates that cope depth is less than flange thickness

Note: Values are omitted when cope depth exceeds $d/2$.

Table IV-11 (continued)

Plastic Section Modulus for Coped W-Shapes

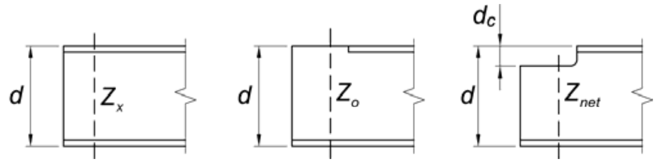


Shape	d, in.	t_f, in.	Z_x, in.^{3}	Z_o, in.^{3}	Z_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W18×71	18.5	0.810	146	76.6	62.2	55.3	48.7	42.4	36.3	30.8	25.7	21.1	
×65	18.4	0.750	133	69.4	56.2	50.0	43.9	38.1	32.7	27.6	23.0	18.9	
×60	18.2	0.695	123	63.0	50.9	45.1	39.6	34.3	29.3	24.7	20.5	16.7	
×55	18.1	0.630	112	58.3	47.1	41.8	36.7	31.8	27.1	22.9	18.9	15.4	
×50	18.0	0.570	101	52.6	42.5	37.7	33.1	28.6	24.4	20.5	17.0	13.8	
W18×46	18.1	0.605	90.7	51.4	41.8	37.3	32.9	28.7	24.7	20.9	17.3	14.1	
×40	17.9	0.525	78.4	44.1	35.8	31.9	28.1	24.5	21.1	17.8	14.7		
×35	17.7	0.425	66.5	39.5	32.1	28.6	25.3	22.1	19.0	16.1	13.4		
W16×100	17.0	0.985	198	80.0	62.4	54.4	47.0	40.2	33.9	28.2	23.1		
×89	16.8	0.875	175	70.5	54.8	47.7	41.1	35.0	29.5	24.4	19.9		
×77	16.5	0.760	150	59.3	45.9	39.8	34.2	29.0	24.3	20.0	16.2		
×67	16.3	0.665	130	50.5	38.9	33.7	28.9	24.4	20.4	16.7	13.4		
W16×57	16.4	0.715	105	53.1	41.7	36.3	31.2	26.4	22.0	18.0	14.4		
×50	16.3	0.630	92.0	46.6	36.5	31.8	27.3	23.0	19.1	15.6	12.5		
×45	16.1	0.565	82.3	41.4	32.4	28.1	24.1	20.3	16.8	13.7	10.9		
×40	16.0	0.505	73.0	36.3	28.4	24.6	21.0	17.7	14.6	11.9	9.40		
×36	15.9	0.430	64.0	33.9	26.6	23.2	19.9	16.8	13.9	11.2			
W16×31	15.9	0.440	54.0	30.4	24.0	21.0	18.1	15.4	12.8	10.4			
×26	15.7	0.345	44.2	26.0	20.5	18.0	15.6	13.3	11.1	9.02			

Note: Values are omitted when cope depth exceeds d/2.

Table IV-11 (continued)

Plastic Section Modulus for Coped W-Shapes



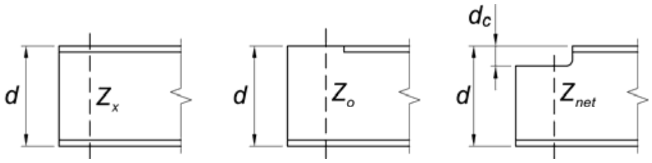
Shape	d, in.	t_f, in.	Z_x, in.^{3}	Z_o, in.^{3}	Z_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W14×873	23.6	5.51	2030	916	–	–	–	–	532	480	432	387	346
×808	22.8	5.12	1830	817	–	–	–	–	463	416	372	332	295
×730	22.4	4.91	1660	669	–	–	–	421	380	341	306	273	243
×665	21.6	4.52	1480	579	–	–	–	358	321	287	256	227	201
×605	20.9	4.16	1320	503	–	–	–	305	272	242	214	189	166
×550	20.2	3.82	1180	434	–	–	289	258	229	203	179	157	137
×500	19.6	3.50	1050	379	–	–	248	221	195	172	150	131	
×455	19.0	3.21	936	331	–	–	213	189	166	145	126	109	
×426	18.7	3.04	869	301	–	–	193	170	149	130	113	97.0	
×398	18.3	2.85	801	273	–	195	172	152	132	115	99.0	84.8	
×370	17.9	2.66	736	246	–	174	154	134	117	101	86.5		
×342	17.5	2.47	672	219	–	154	135	118	102	87.6	74.7		
×311	17.1	2.26	603	193	–	134	117	102	87.5	74.6	63.1		
×283	16.7	2.07	542	169	–	117	101	87.5	74.9	63.5	53.3		
×257	16.4	1.89	487	150	117	102	88.8	76.3	64.9	54.7	45.6		
×233	16.0	1.72	436	130	101	87.9	75.9	64.8	54.8	45.9	37.9		
×211	15.7	1.56	390	115	88.9	77.1	66.2	56.3	47.3	39.3			
×193	15.5	1.44	355	103	78.8	68.1	58.3	49.4	41.4	34.2			
×176	15.2	1.31	320	92.2	70.3	60.6	51.6	43.5	36.2	29.7			
×159	15.0	1.19	287	81.0	61.5	52.8	44.9	37.6	31.2	25.4			
×145	14.8	1.09	260	72.2	54.5	46.7	39.6	33.1	27.2	22.1			
W14×132	14.7	1.03	234	67.4	50.7	43.4	36.6	30.5	25.0	20.1			
×120	14.5	0.940	212	60.2	45.1	38.4	32.3	26.8	21.9	17.5			
×109	14.3	0.860	192	52.3	39.0	33.2	27.8	23.0	18.6	14.8			
×99	14.2	0.780	173	47.7	35.5	30.1	25.2	20.7	16.8	13.3			
×90	14.0	0.710	157	42.2	31.2	26.4	22.0	18.0	14.5	11.4			
×82	14.3	0.855	139	49.7	36.9	31.2	26.1	21.4	17.2	13.6			
×74	14.2	0.785	126	43.5	32.2	27.3	22.7	18.6	15.0	11.7			
×68	14.0	0.720	115	39.1	28.8	24.3	20.2	16.5	13.2	10.2			
×61	13.9	0.645	102	35.0	25.7	21.6	17.9	14.6	11.6				
W14×53	13.9	0.660	87.1	34.2	25.1	21.1	17.4	14.2	11.2				
×48	13.8	0.595	78.4	31.1	22.7	19.1	15.7	12.7	10.1				
×43	13.7	0.530	69.6	27.6	20.1	16.9	13.9	11.2	8.83				
W14×38	14.1	0.515	61.5	29.1	21.7	18.3	15.1	12.3	9.77	7.54			
×34	14.0	0.455	54.6	26.3	19.7	16.5	13.7	11.1	8.78	6.75			
×30	13.8	0.385	47.3	23.8	17.9	15.1	12.5	10.1	7.91				

– Indicates that cope depth is less than flange thickness

Note: Values are omitted when cope depth exceeds $d/2$.

Table IV-11 (continued)

Plastic Section Modulus for Coped W-Shapes



Shape	d, in.	t_f, in.	Z_x, in.^{3}	Z_o, in.^{3}	Z_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W14×26	13.9	0.420	40.2	21.9	16.6	14.1	11.8	9.61	7.58				
×22	13.7	0.335	33.2	18.7	14.2	12.1	10.1	8.26	6.52				
W12×336	16.8	2.96	603	225	–	156	136	118	101	86.2	73.1		
×305	16.3	2.71	537	195	–	133	116	99.6	85.1	72.0	60.5		
×279	15.9	2.47	481	175	–	118	102	87.1	73.8	61.9			
×252	15.4	2.25	428	152	–	101	86.3	73.3	61.6	51.2			
×230	15.1	2.07	386	135	–	88.9	75.8	64.1	53.5	44.1			
×210	14.7	1.90	348	118	89.3	76.6	65.0	54.5	45.2	37.0			
×190	14.4	1.74	311	103	77.2	65.9	55.7	46.5	38.3	31.1			
×170	14.0	1.56	275	88.4	65.8	55.9	46.9	38.8	31.6	25.4			
×152	13.7	1.40	243	77.1	56.9	48.1	40.1	32.9	26.6				
×136	13.4	1.25	214	67.3	49.3	41.4	34.3	27.9	22.3				
×120	13.1	1.11	186	58.1	42.1	35.2	28.9	23.4	18.5				
×106	12.9	0.990	164	48.8	35.2	29.3	24.0	19.3	15.2				
×96	12.7	0.900	147	42.8	30.7	25.5	20.8	16.6	12.9				
×87	12.5	0.810	132	38.9	27.7	22.8	18.5	14.7	11.3				
×79	12.4	0.735	119	35.0	24.8	20.4	16.5	13.0	10.0				
×72	12.3	0.670	108	31.6	22.3	18.3	14.7	11.6	8.85				
×65	12.1	0.605	96.8	27.8	19.5	15.9	12.8	9.96	7.54				
W12×58	12.2	0.640	86.4	26.0	18.3	15.0	12.0	9.39	7.13				
×53	12.1	0.575	77.9	24.5	17.2	14.0	11.2	8.69	6.55				
W12×50	12.2	0.640	71.9	26.4	18.5	15.1	12.1	9.38	7.07				
×45	12.1	0.575	64.2	23.6	16.5	13.4	10.7	8.28	6.20				
×40	11.9	0.515	57.0	20.2	14.0	11.3	8.99	6.92					
W12×35	12.5	0.520	51.2	22.4	15.8	13.0	10.4	8.13	6.15				
×30	12.3	0.440	43.1	18.9	13.3	10.8	8.64	6.70	5.03				
×26	12.2	0.380	37.2	16.5	11.6	9.41	7.48	5.79	4.31				
W12×22	12.3	0.425	29.3	16.9	12.3	10.3	8.32	6.50	4.85				
×19	12.2	0.350	24.7	14.7	10.8	8.96	7.28	5.71	4.27				
×16	12.0	0.265	20.1	12.6	9.23	7.68	6.25	4.93	3.71				
×14	11.9	0.225	17.4	11.1	8.10	6.74	5.48	4.31					

– Indicates that cope depth is less than flange thickness

Note: Values are omitted when cope depth exceeds $d/2$.

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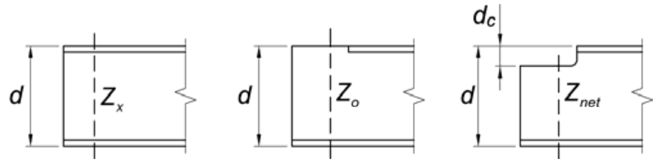
Table IV-11 (continued)

Plastic Section Modulus for Coped W-Shapes



Shape	d, in.	t_f, in.	Z_x, in.^{3}	Z_o, in.^{3}	Z_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W10×112	11.4	1.25	147	46.3	32.1	26.0	20.7	16.1					
×100	11.1	1.12	130	39.8	27.2	21.9	17.3	13.3					
×88	10.8	0.990	113	33.7	22.8	18.2	14.2	10.8					
×77	10.6	0.870	97.6	28.6	19.1	15.2	11.7	8.80					
×68	10.4	0.770	85.3	24.5	16.2	12.8	9.79	7.26					
×60	10.2	0.680	74.6	21.2	13.9	10.8	8.22	6.01					
×54	10.1	0.615	66.6	18.4	12.0	9.32	7.03	5.11					
×49	10.0	0.560	60.4	16.6	10.7	8.33	6.25	4.50					
W10×45	10.1	0.620	54.9	17.2	11.2	8.66	6.51	4.69					
×39	9.92	0.530	46.8	15.0	9.61	7.40	5.50						
×33	9.73	0.435	38.8	13.3	8.41	6.41	4.70						
W10×30	10.5	0.510	36.6	15.7	10.3	8.05	6.08	4.40					
×26	10.3	0.440	31.3	13.2	8.58	6.65	4.98	3.56					
×22	10.2	0.360	26.0	11.9	7.75	5.98	4.45	3.15					
W10×19	10.2	0.395	21.6	11.6	7.80	6.09	4.52	3.19					
×17	10.1	0.330	18.7	10.6	7.16	5.62	4.21	2.95					
×15	9.99	0.270	16.0	9.56	6.47	5.10	3.85						
×12	9.87	0.210	12.6	7.63	5.15	4.05	3.05						
W8×67	9.00	0.935	70.1	21.9	13.5	10.2	7.44						
×58	8.75	0.810	59.8	18.6	11.3	8.40	6.00						
×48	8.50	0.685	49.0	13.9	8.34	6.13	4.31						
×40	8.25	0.560	39.8	11.8	6.90	4.98	3.40						
×35	8.12	0.495	34.7	9.91	5.73	4.10	2.77						
×31	8.00	0.435	30.4	8.86	5.06	3.58	2.38						
W8×28	8.06	0.465	27.2	8.90	5.09	3.60	2.39						
×24	7.93	0.400	23.1	7.44	4.21	2.95							
W8×21	8.28	0.400	20.4	8.18	4.72	3.36	2.24						
×18	8.14	0.330	17.0	7.30	4.16	2.93	1.92						
W8×15	8.11	0.315	13.6	7.22	4.29	3.02	1.96						
×13	7.99	0.255	11.4	6.38	3.82	2.70							
×10	7.89	0.205	8.87	4.74	2.79	1.95							

Note: Values are omitted when cope depth exceeds $d/2$.