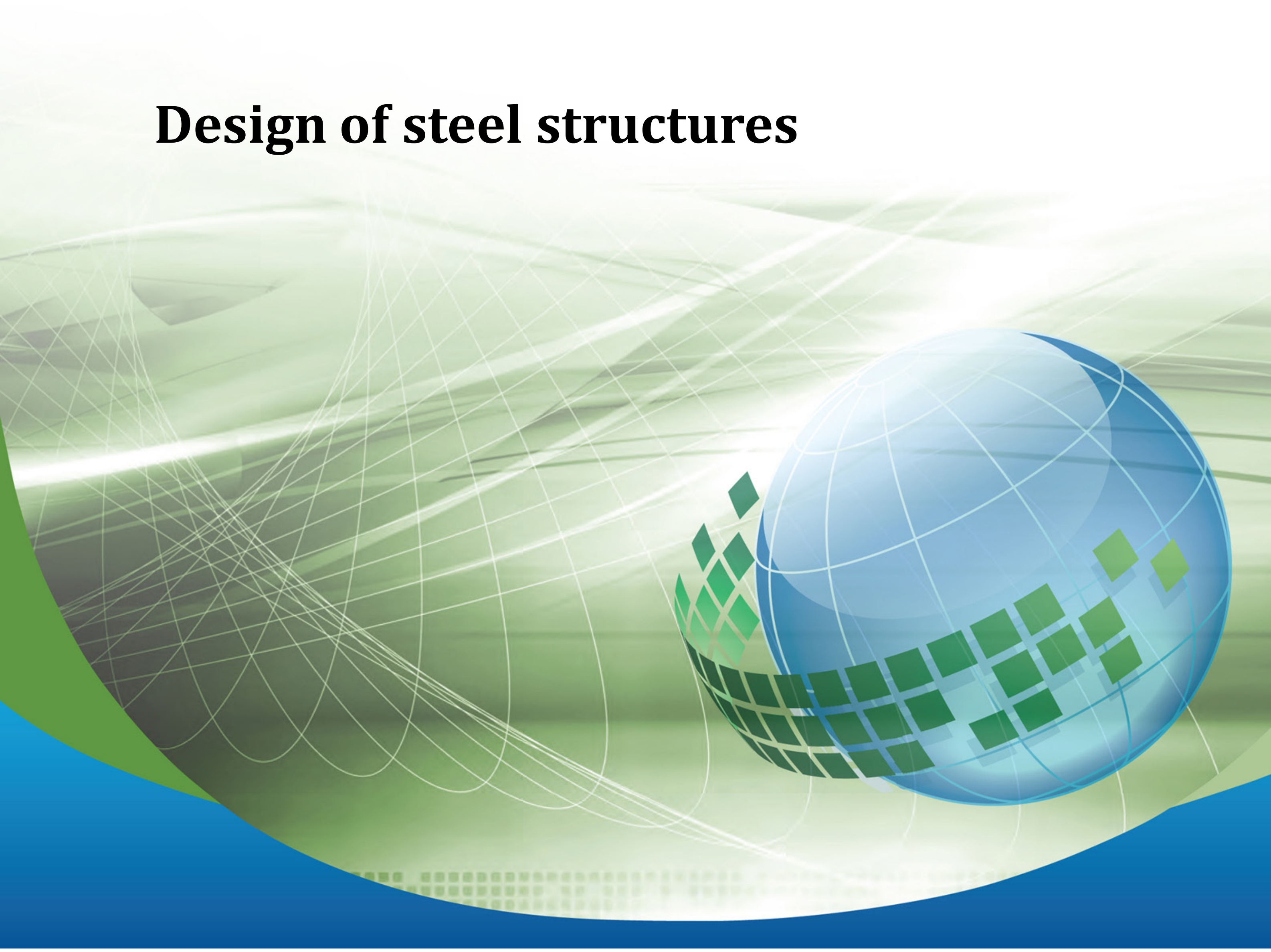
CSA Group

S16-14

Design of steel structures



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Design of steel structures

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Preface

This is the eighth edition of CSA S16, *Design of steel structures*. It supersedes the previous limit states editions published in 2009, 2001, 1994, 1989, 1984, 1978, and 1974. These limit states design editions were preceded by seven working stress design editions published in 1969, 1965, 1961, 1954, 1940, 1930, and 1924. The 1969 working stress design edition was withdrawn in 1984, from which point the design of steel structures in Canada has been carried out using limit states design principles.

This Standard is appropriate for the design of a broad range of structures. It sets out minimum requirements and is expected to be used only by engineers competent in the design of steel structures. The following is a list of some of the more important changes made in this edition:

a) Clause 1.4 specifically prohibits the use of other standards for fabrication, erection and inspection.
b) The definition of "snug-tightness" has been clarified.
c) Information required on design documents has been augmented.
d) ASTM grades A500/A500M, A1085 and A913/A913M have been added as permissible steel grades for design.
e) The fire endurance design requirements have been restated to be in compliance with the *NBCC*.
f) Requirements under impulse loading have been added.
g) The initial misalignment of members at brace points has been clarified.
h) A calculation for the net area of a slotted HSS member has been given.
i) The minimum *b*/*t* for bearing stiffeners has been added.
j) The clause permitting a joist manufacturer to determine the joist resistance by testing has been removed.
k) Provisions for column stiffeners opposite of a rigidly connected beam by bolting have been provided.
l) Requirements for zinc/aluminum coated assemblies have been incorporated.
m) The use of plate washers in lieu of hardened washers is permitted in oversize or slotted holes.
n) The use of non-matching electrodes is permitted with reference to W59 for locations where this is permitted.
o) Clause 24 that referred to joint surface conditions for field welding in the previous edition has been removed and is now covered in CSA W47.1.
p) The factored resistance of anchor rods in bearing has been referred to CSA A23.3 to be consistent with other Canadian design standards.
q) A clarification on fatigue calculations has been made to include bending moments due to joint eccentricities.
r) An upper limit on the design force of single-storey buildings' roof diaphragms has been provided.
s) A minimum Charpy V-notch value has been specified for weld of primary members and connections.
t) A maximum sulfur content for ASTM A913 used in seismic resisting systems is specified.
u) Additional criteria for joint connections has been added to ductile moment-resisting frames, limited ductility moment-resisting frames, and moderately ductile concentrically braced frames.
v) The design of link beams for ductile eccentrically braced frames has been expanded.
w) Detailing information for limited ductility plate walls have been given.
x) Annex K Structural design for fire conditions has been updated.
y) The clauses related to pin-connected members have been revised to clarify the net section and resistance requirements.

A commentary on this Standard, prepared by the Canadian Institute of Steel Construction with contributions from many members of the Technical Committee, comprises Part 2 of the Institute's *Handbook of Steel Construction*.

This Standard is intended to be used with the provisions of the 2015 edition of the *National Building Code of Canada* (*NBCC*), specifically Clause 7, which references the *NBCC* for load factors, load combinations, and other loading provisions..

This Standard was prepared by the Technical Committee on Steel Structures for Buildings, under the jurisdiction of the Strategic Steering Committee for Construction and Civil Infrastructure, and has been formally approved by the Technical Committee.

This edition of the CSA S16 is dedicated to the memories of Laurie Kennedy, André Picard, and Richard Redwood, three distinguished designers, researchers, and devoted educators committed to the advancement of steel standards.

Notes:

1) *Use of the singular does not exclude the plural (and vice versa) when the sense allows.*

2) *Although the intended primary application of this Standard is stated in its Scope, it is important to note that it remains the responsibility of the users of the Standard to judge its suitability for their particular purpose.*

3) *This Standard was developed by consensus, which is defined by CSA Policy governing standardization — Code of good practice for standardization as "substantial agreement. Consensus implies much more than a simple majority, but not necessarily unanimity". It is consistent with this definition that a member may be included in the Technical Committee list and yet not be in full agreement with all clauses of this Standard.*

4) *To submit a request for interpretation of this Standard, please send the following information to* ***inquiries@csagroup.org*** *and include "Request for interpretation" in the subject line:*

   a) *define the problem, making reference to the specific clause, and, where appropriate, include an illustrative sketch;*

   b) *provide an explanation of circumstances surrounding the actual field condition; and*

   c) *where possible, phrase the request in such a way that a specific "yes" or "no" answer will address the issue.*

   Committee interpretations are processed in accordance with the CSA Directives and guidelines governing standardization and are available on the Current Standards Activities page at standardsactivities.csa.ca.

5) *This Standard is subject to review five years from the date of publication. Suggestions for its improvement will be referred to the appropriate committee. To submit a proposal for change, please send the following information to* ***inquiries@csagroup.org*** *and include "Proposal for change" in the subject line:*

   a) *Standard designation (number);*

   b) *relevant clause, table, and/or figure number;*

   c) *wording of the proposed change; and*

   d) *rationale for the change.*

S16-14
Design of steel structures

1 Scope and application

1.1 General
This Standard provides rules and requirements for the design, fabrication, and erection of steel structures. The design is based on limit states. The term "steel structures" refers to structural members and frames that consist primarily of structural steel components, including the detail parts, welds, bolts, or other fasteners required in fabrication and erection. This Standard also applies to structural steel components in structures framed in other materials. The clauses related to fabrication and erection serve to show that design is inextricably a part of the design-fabrication-erection sequence and cannot be considered in isolation. For matters concerning standard practice pertinent to the fabrication and erection of structural steel not covered in this Standard, see Annex A.

1.2 Requirements
Requirements for steel structures such as bridges, antenna towers, offshore structures, and cold-formed steel structural members are given in other CSA Group Standards.

1.3 Application
This Standard applies unconditionally to steel structures, except that supplementary rules or requirements might be necessary for
a) unusual types of construction;
b) mixed systems of construction;
c) steel structures that
   i) have great height or spans;
   ii) are required to be movable or be readily dismantled;
   iii) are exposed to severe environmental conditions;
   iv) are exposed to severe loads such as those resulting from vehicle impact or explosion;
   v) are required to satisfy aesthetic, architectural, or other requirements of a non-structural nature;
   vi) employ materials or products not listed in Clause 5; or
   vii) have other special features that could affect the design, fabrication, or erection;
d) tanks, stacks, other platework structures, poles, and piling; and
e) crane-supporting structures.

1.4 Other standards
The use of other standards for the design, fabrication, erection, and/or inspection of members or parts of steel structures is neither warranted nor acceptable except where specifically directed in this Standard. The design formulas provided in this Standard may be supplemented by a rational design based on theory, analysis, and engineering practice acceptable to the regulatory authority, provided that nominal margins (or factors) of safety are at least equal to those intended in the provisions of this Standard. The substitution of other standards or criteria for fabrication, erection, and/or inspection is expressly prohibited unless specifically directed in this Standard.

1.5 Terminology

In this Standard, "shall" is used to express a requirement, i.e., a provision that the user is obliged to satisfy in order to comply with the standard; "should" is used to express a recommendation or that which is advised but not required; and "may" is used to express an option or that which is permissible within the limits of the Standard.

Notes accompanying clauses do not include requirements or alternative requirements; the purpose of a note accompanying a clause is to separate from the text explanatory or informative material.

Notes to tables and figures are considered part of the table or figure and may be written as requirements.

Annexes are designated normative (mandatory) or informative (non-mandatory) to define their application.

2 Reference publications

This Standard refers to the following publications, and where such reference is made, it shall be to the edition listed below, including all amendments published thereto.

CSA Group

A23.1/A23.2-14
Concrete materials and methods of concrete construction/Test methods and standard practices for concrete

A23.3-14
Design of concrete structures

A660-10
Certification of manufacturers of steel building systems

B95-1962 (withdrawn)
Surface Texture (Roughness, Waviness, and Lay)

G40.20-13/G40.21-13
General requirements for rolled or welded structural quality steel/Structural quality steel

CAN/CSA-G164-M92 (withdrawn)
Hot Dip Galvanizing of Irregularly Shaped Articles

G189-1966 (withdrawn)
Sprayed Metal Coatings for Atmospheric Corrosion Protection

S136-12
North American specification for the design of cold-formed steel structural members

S304-14
Design of masonry structures

S850-12
Design and assessment of buildings subject to blast loads

W47.1-09
Certification of companies for fusion welding of steel

W48-14
Filler metals and allied materials for metal arc welding

W55.3-08 (R2013)
Certification of companies for resistance welding of steel and aluminum

W59-13
Welded steel construction (metal arc welding)

W178.1-14
Certification of welding inspection organizations

W178.2-14
Certification of welding inspectors

ASTM International (American Society for Testing and Materials)

A27/A27M-10
Standard Specification for Steel Castings, Carbon, for General Application

A108-07
Standard Specification for Steel Bar, Carbon and Alloy, Cold-Finished

A148/A148M-08
Standard Specification for Steel Castings, High Strength, for Structural Purposes

A216/A216M-12
Standard Specification for Steel Castings, Carbon, Suitable for Fusion Welding, for High-Temperature Service

A307-12
Standard Specification for Carbon Steel Bolts, Studs, and Threaded Rod 60 000 PSI Tensile Strength

A325-10e1
Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength

A325M-13
Standard Specification for Structural Bolts, Steel, Heat Treated 830 MPa Minimum Tensile Strength (Metric)

A352/A352M-06(2012)
Standard Specification for Steel Castings, Ferritic and Martensitic, for Pressure-Containing Parts, Suitable for Low-Temperature Service

A490-12
Standard Specification for Structural Bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength

A490M-12
Standard Specification for High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints (Metric)

A500/A500M-10a
Standard Specification for Cold Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

A514/A514M-05(2009)
Standard Specification for High-Yield-Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding

A521/A521M-06(2011)
Standard Specification for Steel, Closed-Impression Die Forgings for General Industrial Use

A563-07a
Standard Specification for Carbon and Alloy Steel Nuts

A572/A572M-12a
Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel

A668/A668M-13
Standard Specification for Steel Forgings, Carbon and Alloy, for General Industrial Use

A913/A913M-11
Standard Specification for High Strength Low Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self Tempering Process

A958/A958M-10
Standard Specification for Steel Castings, Carbon and Alloy, with Tensile Requirements, Chemical Requirements Similar to Standard Wrought Grades

A992/A992M-11
Standard Specification for Structural Steel Shapes

A1011/A1011M-12b
Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, and Ultra-High Strength

A1085-13
Standard Specification for Cold-Formed Welded Carbon Steel Hollow Structural Sections (HSS)

F436-11
Standard Specification for Hardened Steel Washers

F959-13
Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners

F1554-07ae1
Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength

F1852-11
Standard Specification for "Twist Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength

F2280-12
Standard Specification for "Twist Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 150 ksi Minimum Tensile Strength

CISC (Canadian Institute of Steel Construction)
Code of Standard Practice for Structural Steel (2009)

Crane-Supporting Steel Structures: Design Guide, 2nd ed. (April 2013)

Handbook of Steel Construction, 11th ed. (2015)

Hollow Structural Section: Connections and Trusses — A Design Guide, 2nd ed. (June 1997)

Moment Connections for Seismic Applications, 2nd ed. (2014)

CISC/CPMA (Canadian Institute of Steel Construction/Canadian Paint Manufacturing Association)
1-73a (1975)
A Quick-Drying One-Coat Paint for Use on Structural Steel

2-75 (1975)
A Quick-Drying Primer for Use on Structural Steel

National Research Council Canada
National Building Code of Canada, 2015

User's Guide — NBC 2015: Structural Commentaries (Part 4)

RCSC (Research Council on Structural Connections)
Guide to Design Criteria for Bolted and Riveted Joints, 2nd ed., 2001

Specification for Structural Joints Using ASTM A325 or A490 Bolts, 2000

SSPC (Society for Protective Coatings)
SP 1 (2004)
Solvent Cleaning

SP 2 (2004)
Hand Tool Cleaning

SP 3 (2004)
Power Tool Cleaning

SP 5/NACE No. 1 (2007)
White Metal Blast Cleaning

SP 6/NACE No. 3 (2007)
Commercial Blast Cleaning

SP 7/NACE No. 4 (2007)
Brush-Off Blast Cleaning

SP 10/NACE No. 2 (2007)
Near-White Blast Cleaning

SP 11 (2004)
Power Tool Cleaning to Bare Metal

SP 12/NACE No. 5
Surface Preparation and Cleaning of Metals by Waterjetting Prior to Recoating

SP 14/NACE No. 8
Industrial Blast Cleaning

Structural Stability Research Council
Guide to Stability Design Criteria for Metal Structures, 6th ed., 2010

ULC (Underwriters Laboratories of Canada)
CAN/ULC-S101-07
Standard Methods of Fire Endurance Tests of Building Construction and Materials

Other publications
Frank, K. H. and Fisher, J. W. "Fatigue Strength of Welded Cruciform Joints", *Journal of the Structural Division*, ASCE. Vol.105, ST9, pp. 1727-1740, September 1979.

3 Definitions and symbols

3.1 Definitions

The following definitions apply in this Standard:

Approved — approved by the regulatory authority.

Brace point — the point on a member or element at which it is restrained (see Clause 9).

Camber — the deviation from straightness of a member or any portion of a member with respect to its major axis.

Note: *Frequently, camber is specified and produced in a member to compensate for deflections that occur in the member when loaded (see Clause 6.3.2). Unspecified camber is sometimes referred to as bow.*

Concrete — portland cement concrete in accordance with CSA A23.1.

Deck or decking — the structural floor or roof element spanning between adjacent joists and directly supported thereby.

Note: *The terms "deck" and "decking" include cast-in-place or precast concrete slabs, profiled metal deck, wood plank or plywood, and other relatively rigid elements suitable for floor or roof construction (see Clause 16).*

Designer — the engineer responsible for the design.

Erection tolerances — tolerances related to the plumbness, alignment, and level of the piece as a whole.

Note: *The deviations are determined by considering the location of the ends of the piece (see Clause 29).*

Fabrication tolerances — tolerances allowed from the nominal dimensions and geometry, such as cutting to length, finishing of ends, cutting of bevel angles, and out-of-straightness such as sweep and camber for fabricated members (see Clause 28).

Factors —

Importance factor, *I* — a factor applied severally to loads due to snow and rain, wind, or earthquake for both the ultimate and serviceability limit states.

Note: *It is based on the importance of the structure as defined by its use and occupancy (see Clause 6.2.2).*

Load factor, α — a factor, given in Clause 7.2, applied to a specified load for the limit states under consideration that takes into account the variability of the loads and load patterns and the analysis of their effects.

Resistance factor, φ — a factor, given in Clause 13.1, applied to a specified material property or the resistance of a member, connection, or structure that, for the limit state under consideration, takes into account the variability of material properties, dimensions, quality of work, type of failure, and uncertainty in prediction of member resistance.

Note: *To maintain the simplicity of the design formulas in this Standard, the type of failure and the uncertainty in prediction of member resistance have been incorporated in the expressions of member resistance (see Annex B for a more detailed discussion).*

Fatigue limit state — the limiting case of the slow propagation of a crack within a structural element that can result either from live load effects (load-induced fatigue effect) or as the consequence of local distortion within the structure (distortion-induced fatigue effects).

Firm contact — the condition that exists on a faying surface when plies are solidly seated against each other but not necessarily in continuous contact (see Clause 23.2).

Inspector — a qualified person who acts for and on behalf of the owner or designer on all inspection and quality matters within the scope of the contract documents.

Joist shoe — the connection assembly located at the junction of the top chord and the end diagonal that allows the joist to bear on its support (see Clause 16).

Limit states — those conditions of a structure under which the structure ceases to fulfill the function for which it was designed.

Fatigue limit states — conditions that concern safety and are related to crack propagation under cyclic loading.

Serviceability limit states — conditions that restrict the intended use and occupancy of the structure and include deflection, vibration, and permanent deformation.

Ultimate limit states — conditions that concern safety and include overturning, sliding, fracturing, and exceeding load-carrying capacity.

Loads —

Companion load — a specified variable load that accompanies the principal load in a given load combination.

Factored load — the product of a specified load and its load factor.

Gravity load (newtons) — a load equal to the mass of the object (kilograms) being supported multiplied by the acceleration due to gravity, g (9.81 m/s^2).

Notional lateral load — a fictitious lateral load, as given in Clause 8.4, that allows the stability of the frame, with failure modes involving in-plane bending, to be computed based on the actual length (K = 1) for beam-columns.

Principal load — the specified variable load or rare load that dominates in a given load combination.

Specified loads (D, E, H, L, L_{-c}, C, C_d, C_T, P, S, T, and W) — those loads prescribed by the regulatory authority (see Clause 6.2.1).

Mill tolerances — variations allowed from the nominal dimensions and geometry with respect to cross-sectional area, non-parallelism of flanges, and out-of-straightness such as sweep or camber in the product as manufactured and given in CSA G40.20.

Modulus of elasticity of concrete — the ratio of stress to strain in the elastic range of a stress-strain curve for concrete and, with density, γ_c, between 1500 and 2500 kg/m^3, is taken as follows:

$$E_c = \left(3300\sqrt{f'_c} + 6900\right)\left(\frac{\gamma_c}{2300}\right)^{1.5}$$

For normal density concrete with compressive strength, f'_c, between 20 and 40 MPa, the modulus of elasticity may be taken as follows:

$$E_c = 4500\sqrt{f'_c}$$

Pass through force — a load or force defined by the Structural Designer that must be accommodated in the design of the structural member(s) and the connections between those designated members in addition to those loads and forces normally associated in the member and connection design of each individual interconnecting member.

Protected zone — areas of members in a seismic force resisting system that undergo large inelastic strains and in which limitations apply to fabrication and attachments. See Clause 27.1.9.

Regulatory authority — a federal, provincial/territorial, or municipal ministry, department, board, agency, or commission that is responsible for regulating by statute the use of products, materials, or services.

Resistance —

Factored resistance, ϕR — the product of the nominal resistance and the appropriate resistance factor.

Nominal resistance, R — the nominal resistance of a member, connection, or structure as calculated in accordance with this Standard and based on the specified material properties and nominal dimensions.

Segmented member — a member with a constant cross-section when axial loads are applied between in-plane lateral supports or frame connections, and a member with cross-section changes between in-plane lateral supports or frame connections.

Seismic design storey drift — the storey drift obtained from the lateral deflections obtained from a linear elastic analysis multiplied by R_dR_o/I_e (see Clause 27).

Snug-tightness — the condition that exists when all of the plies in a connection have been pulled into firm contact by the bolts in the joint and all of those bolts remain tightened sufficiently to prevent the removal of the nuts without the use of a wrench (see Clause 23).

Span of an open-web steel joist — the centre-to-centre distance of joist bearings or shoes (see Clause 16).

Sweep — the deviation from straightness of a member or any portion of a member with respect to its minor axis.

Tie joists — joists that are designed to resist gravity loads only and, in accordance with Clause 16.5.12.2, have at least one end connected to a column to facilitate erection.

Truss — a triangulated framework loaded primarily in flexure (see Clause 15).

3.2 Symbols

The following symbols are used throughout this Standard. Deviations or additional nomenclature are noted where they appear.

A = area

A_{ar} = cross-sectional area of an anchor rod based on its nominal diameter

A_b = cross-sectional area of a bolt based on its nominal diameter; cross-sectional area of a plate wall beam

A_c = transverse area of concrete between longitudinal shear planes; cross-sectional area of concrete in composite columns; cross-sectional area of a plate wall column; effective area of concrete slab

A_{cv} = critical area of two longitudinal shear planes, one on each side of the area A_c, extending from the point of zero moment to the point of maximum moment

A_e = effective area of section in compression to account for elastic local buckling (see Clause 13.3.5)

A_f = flange area

A_g = gross area

A_{gv} = gross area in shear for block failure (see Clause 13.11)

A_m = area of fusion face

A_n = net area; the tensile area of a rod

A_{ne} = effective net area reduced for shear lag

A_p = concrete pull-out area

A_r = area of reinforcing steel

A_s = area of steel section, including cover plates; area of bottom (tension) chord of a steel joist; area of a stiffener or pair of stiffeners

A_{sc} = cross-sectional area of a steel shear connector; cross-sectional of the yielding segment of the steel core of a buckling restrained brace

A_{se} = effective steel area (see Clause 18.3.2)

A_{st} = area of steel section in tension

A_w = web area; shear area; effective throat area of a weld

a = centre-to-centre distance between transverse web stiffeners; depth of the concrete compression zone

a' = length of cover plate termination

a/h = aspect ratio; ratio of distance between stiffeners to web depth

B = bearing force in a member or component under specified load

B_f = bearing force in a member or component under factored load

B_r = factored bearing resistance of a member or component

b_l = longer leg of angle in Clause 13.3.3

b_s = shorter leg of angle in Clause 13.3.3

b = overall width of flange; design effective width of concrete or cover slab

b_{el} = width of stiffened or unstiffened compression elements

b_c = width of concrete at the neutral axis specified in Clause 18.2.3; width of column flange

b_e = effective flange width in Clause 18.3.2

b_f = width of flange

C_e = Euler buckling strength

= $\pi^2 EI/L^2$

C_{ec} = Euler buckling strength of a concrete-filled hollow structural section

C_f = compressive force in a member or component under factored load; factored axial load

C_{fs} = factored sustained axial load on a composite column

C_p = nominal compressive resistance of a composite column when $\lambda = 0$ (see Clause 18.3.2)

C_r = factored compressive resistance of a member or component; factored compressive resistance of steel acting at the centroid of that part of the steel area in compression

C_{rc} = factored compressive resistance of a composite column

C_{rcm} = factored compressive resistance that can coexist with M_{rc} when all of the cross-section is in compression

C_{rco} = factored compressive resistance with $\lambda = 0$

C'_r = compressive resistance of concrete acting at the centroid of the concrete area assumed to be in uniform compression; compressive resistance of a concrete component of a composite column

C_w = warping torsional constant, mm^6

C_y = axial compressive load at yield stress

c = cohesion stress for concrete (1.0 MPa) in accordance with Clause 11.5.2 c) of CSA A23.3

c_1 = coefficient used to determine slip resistance

D = outside diameter of circular sections; diameter of rocker or roller; stiffener factor; dead load

d = depth; overall depth of a section; diameter of a bolt or stud

d_b = depth of beam

d_c = depth of column

E = elastic modulus of steel (200 000 MPa assumed); earthquake load and effects (see Clause 6.2.1)

E_c = elastic modulus of concrete

E'_c = age adjusted effective modulus of electricity of concrete

E_{ct} = effective modulus of concrete in tension

e = end distance; lever arm between the compressive resistance, C_r, and the tensile resistance, T_r; length of link in eccentrically braced frames

e' = lever arm between the compressive resistance, C'_r, of concrete and tensile resistance, T_r, of steel

F = strength or stress

F_a = acceleration-based site coefficient (see Clause 27 and the *NBCC*)

F_{cr} = critical plate-buckling stress in compression, flexure, or shear

F_{cre} = elastic critical plate-buckling stress in shear

F_{cri} = inelastic critical plate-buckling stress in shear

F_e = Euler buckling stress; elastic buckling stress

F_s = ultimate shear stress

F_{sr} = allowable stress range in fatigue

F_{srt} = constant amplitude threshold stress range

F_{st} = factored axial force in the stiffener

F_u = specified minimum tensile strength

F_v = velocity-based site coefficient (see Clause 27 and the *NBCC*)

F_y = specified minimum yield stress, yield point, or yield strength

F'_y = yield level, including effect of cold-working

F_{ye} = effective yield stress of section in compression to account for elastic local buckling (see Clause 13.3.5)

F_{yr} = specified yield strength of reinforcing steel

f'_c = specified compressive strength of concrete at 28 days

f_{sr} = calculated stress range at detail due to passage of the fatigue load

G = shear modulus of steel (77 000 MPa assumed)

g = transverse spacing between fastener gauge lines (gauge distance)

H = weld leg size; permanent load due to lateral earth pressure (see Clause 6.2.1)

h = clear depth of web between flanges; height of stud; storey height

h_c = clear depth of column web

h_d = depth of steel deck

h_s = storey height

I = moment of inertia

I_b = moment of inertia of a beam

I_c = moment of inertia of a column

I_E = earthquake importance factor of the structure (see Clause 27 and the *NBCC*)

I_e = effective moment of inertia of a composite beam

I_g = moment of inertia of a cover-plated section

I_S = importance factor for snow load as defined in Table 4.1.6.2 of the *NBCC*

I_s = moment of inertia of OWSJ or truss

I_t = transformed moment of inertia of a composite beam

I_W = importance factor for wind load as defined in Table 4.1.7.1 of the *NBCC*

I_{yc} = moment of inertia of compression flange about the y-axis [see Clause 13.6 e)]

I_{yt} = moment of inertia of tension flange about the y-axis [see Clause 13.6 e)]

J = St. Venant torsional constant

K = effective length factor

K_z = effective length factor for torsional buckling

KL = effective length

k = distance from outer face of flange to web-toe of fillet of I-shaped sections; factor as specified in Clause 18.3.2

k_a = coefficient used in determining inelastic shear resistance

k_b = buckling coefficient; required stiffness of the bracing assembly

k_s = mean slip coefficient

k_v = shear buckling coefficient

L = length or span; length of longitudinal or flare bevel groove weld; live load; length of connection in direction of loading; centre-to-centre distance between columns in a plate wall; length of member between work points at truss chord centrelines in Clause 13.3.3

L_c = length of channel shear connector

L_{cr} = maximum unbraced length adjacent to a plastic hinge

L_u = longest unbraced length with which a beam will reach either $M_r = \phi M_p$ or $M_r = \phi M_y$, depending on the class of the cross-section [see Clause 13.6 e)]

L_{yr} = shortest unbraced length with which a singly symmetric beam will undergo elastic lateral-torsional buckling [see Clause 13.6 e)]

M = bending moment in a member or component under specified load

M_a = factored bending moment at one-quarter point of unbraced segment

M_b = factored bending moment at mid-point of unbraced segment

M_c = factored bending moment at three-quarter point of unbraced segment

M_f = bending moment in a member or component under factored load

M_{f1} = smaller factored end moment of a beam-column; factored bending moment at a point of concentrated load

M_{f2} = larger factored end moment of a beam-column

M_{fc} = bending moment in a girder, under factored load, at theoretical cut-off point

M_{max} = maximum factored bending moment magnitude in unbraced segment

M_p = plastic moment resistance = ZF_y

= ZF_y

M_{pb} = plastic moment of a beam

M_{pc} = plastic moment of a column

M_r = factored moment resistance of a member or component

M_{rc} = factored moment resistance of a composite beam; factored moment resistance of a column reduced for the presence of an axial load

M_u = critical elastic moment of a laterally unbraced beam

M_w = strength reduction factor for multi-orientation fillet welds to account for ductility incompatibility of the individual weld segments

M_y = yield moment resistance = SF_y

M_{yr} = yield moment resistance of a singly symmetric beam including the effects of residual stresses [see Clause 13.6 e)]

m = number of faying surfaces or shear planes in a bolted joint

= 1.0 for bolts in single shear

= 2.0 for bolts in double shear

N = length of bearing of an applied load; number of passages of moving load

N' = number of passages of moving load at which $F_{sr} = F_{srt}$

N_{fi} = number of cycles that would cause failure at stress range level i

n = number of bolts; number of shear connectors required between the point of maximum positive bending moment and the adjacent point of zero moment; parameter for compressive resistance; number of threads per inch; number of stress range cycles at a given detail for each passage of the moving load; modular ratio, E/E_c

n' = number of shear connectors required between any concentrated load and nearest point of zero moment in a region of positive bending moment

n_s = modular ratio, E/E_c'

n_t = modular ratio, E/E_{ct}

P = force to be developed in a cover plate; pitch of threads; permanent effects caused by prestress (see Clause 6.2.1)

P_b = force used to design the bracing system (when two or more points are braced, the forces P_b alternate in direction)

P_f = factored axial force

p = fraction of full shear connection

Q_r = sum of the factored resistances of all shear connectors between points of maximum and zero moment

q_r = factored resistance of a shear connector

q_{rr} = factored resistance of a shear connector in a ribbed slab

q_{rs} = factored resistance of a shear connector in a solid slab

R = end reaction or concentrated transverse load applied to a flexural member; nominal resistance of a member, connection, or structure; transition radius

R_d = ductility-related force modification factor that reflects the capability of a structure to dissipate energy through inelastic behaviour (see Clause 27 and the *NBCC*)

R_o = overstrength-related force modification factor that accounts for the dependable portion of reserve strength in a structure (see Clause 27 and the *NBCC*)

R_y = factor applied to F_y to estimate the probable yield stress

r = radius of gyration

r_t = radius of gyration of a compression flange plus one-third of web area in compression due to major axis bending [see Clause 13.6 e) i)]

r_x = radius of gyration of a single-angle member about its geometric axis parallel to the connected leg in Clause 13.3.3

r_y = radius of gyration of a member about its weak axis

r_y' = radius of gyration of a member about its minor principal axis

S = elastic section modulus of a steel section; variable load due to snow (see Clause 6.2.1)

$S_a(0.2)$ = 5% damped spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period of 0.2 s (see Clause 27 and the *NBCC*)

$S_a(1.0)$ = 5% damped spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period of 1 s (see Clause 27 and the *NBCC*)

S_e = effective section modulus as defined in Clause 13.5 c)

s = centre-to-centre longitudinal spacing (pitch) of any two successive fastener holes; longitudinal stud spacing; vertical spacing of tie bars (see Clause 18.3.1)

T = tensile force in a member or component under specified load; load effects due to contraction, expansion, or deflection (see Clause 6.2.1); period of a structure (see Clause 27 and the *NBCC*)

T_f = tensile force in a member or component under factored load

T_r = factored tensile resistance of a member or component; in composite construction, factored tensile resistance of the steel acting at the centroid of that part of the steel area in tension

T_y = axial tensile load at yield stress

t = thickness; thickness of flange; average flange thickness of channel shear connector

t_b = thickness of beam flange

t_c = concrete or cover slab thickness; thickness of column flange

t_p = thickness of plate

U_t = factor to account for efficiency of the tensile area (see Clause 13.11)

U_1 = factor to account for moment gradient and for second-order effects of axial force acting on the deformed member

U_2 = amplification factor to account for second-order effects of gravity loads acting on the laterally displaced storey

V = shear force in a member or component under specified load

V_f = shear force in a member or component under factored load

V_h = total horizontal shear to be resisted at the junction of the steel section or joist and the slab or steel deck; shear acting at plastic hinge locations when plastic hinging occurs

V_p = plastic shear resistance = $0.55wdF_y$

V_r = factored shear resistance of a member or component

V_{re} = probable shear resistance of a steel plate wall

V_s = slip resistance of a bolted joint

V_{st} = factored shear force in column web to be resisted by stiffener

W = wind load

w = web thickness; width of plate; infill plate thickness (see Clause 20)

w' = sum of thickness of column web plus doubler plates

w_c = column web thickness

w_d = average width of flute of steel deck

w_f = width of flare bevel groove weld face

w_n = net width (i.e., gross width less design allowance for holes within the width)

X_u = ultimate strength as rated by the electrode classification number

x = subscript relating to strong axis of a member; distance from flange face to centre of plastic hinge

$\bar{x}$ = eccentricity of the weld with respect to centroid of the element

x_o, y_o = principal coordinates of the shear centre with respect to the centroid of the cross-section

y = subscript relating to weak axis of a member; distance from centroid of cover plate to neutral axis of cover-plated section; distance from centroid of the effective area of concrete slab to elastic neutral axis

Z = plastic section modulus of a steel section

z = subscript related to Z-axis of a member

α = load factor; angle of inclination from vertical (see Clause 20)

α_f = angle between shear friction reinforcement and shear plane in concrete

α_1 = ratio of average stress in rectangular compression block to the specified concrete strength

β = value used to determine bracing stiffness; angle in radians as specified in Clause 18.2.3; coefficient for bending in beam-columns, as specified in Clause 13.8.2 or Clause 18.2.4

β_x = asymmetry parameter for singly symmetric beams as specified in Clause 13.6 e)

γ = fatigue life constant

γ' = fatigue life constant at which $F_{sr} = F_{srt}$

γ_c = density of concrete

Δ_b = displacement of the bracing system at the point of support under force C_f (may be taken as Δ_o)

Δ_f = relative first-order lateral (translational) displacement of the storey due to factored loads

Δ_o = initial misalignment of the member at a brace point (see Clause 9.2)

Δ_s = deflection due to shrinkage of concrete

εf = free shrinkage strain of concrete

κ = ratio of the smaller to the larger factored end moment, positive for double curvature and negative for single curvature (see Clauses 13.6 and 13.8)

λ = non-dimensional slenderness parameter in column formula; modification factor for concrete density

λ_p = non-dimensional slenderness parameter as specified in Clause 18.3.2

μ = coefficient of friction for concrete (1.4) in accordance with Clause 11.5.2 c) of CSA A23.3

ρ = density of concrete; slenderness ratio

ρ_e = equivalent slenderness ratio of a built-up member

ρ_i = maximum slenderness ratio of the component part of a built-up member between interconnectors

ρ_o = slenderness ratio of a built-up member acting as an integral unit

ρ_v = ratio of shear friction reinforcing steel in concrete extending from the point of zero moment to the point of maximum moment

ΣC_f = sum of factored axial compressive loads of all columns in the storey

ΣV_f = sum of factored lateral loads above the storey; total first-order storey shear

σ = effective normal stress for concrete in accordance with Clause 11.5.3 of CSA A23.3

σ_{ct} = tensile stress in concrete

ϕ = resistance factor as defined in Clause 2 and specified in Clause 13.1

ω_h = non-dimensional column flexibility parameter for plate walls

ω_L = non-dimensional boundary member flexibility parameter for extreme panels of plate walls

ω_1 = coefficient to determine equivalent uniform bending effect in beam-columns (see Clause 13.8)

ω_2 = coefficient to account for increased moment resistance of a laterally unsupported doubly symmetric beam segment when subject to a moment gradient [see Clause 13.6 a)]

ω_3 = coefficient to account for modified moment resistance of a laterally unsupported singly symmetric beam segment when subject to a moment gradient [see Clause 13.6 e)]

3.3 Units

Equations and expressions appearing in this Standard are compatible with the following SI (metric) units:

a) force: N (newtons);

b) length: mm (millimetres);

c) moment: N•mm; and
d) strength or stress: MPa (megapascals).

4 Structural documents

4.1 General

The term "structural documents" may include drawings, specifications, computer output, and electronic and other data.

4.2 Structural design documents

4.2.1

The structural design documents shall show a complete design of the structure with members suitably designated and located, including such dimensions and details as necessary to permit the preparation of fabrication and erection documents. Floor levels, column centres, and offsets shall be dimensioned. Structural design drawings shall be to a scale adequate to convey the required information.

4.2.2

In addition to the information required by the applicable building code, the structural design documents shall include, but not be limited to, the following information, as applicable:

a) the design standards used;
b) the material or product standards (see Clause 5);
c) the design criteria for snow, wind, seismic, and special loads;
d) the specified live, dead loads, and superimposed dead loads;
e) the type or types of construction (see Clause 8);
f) the structural system used for seismic design and the seismic design criteria (see Clause 27);
g) the requirements for roof and floor diaphragms;
h) the design criteria for open-web steel joists (see Clause 16);
i) the design criteria for crane-supporting structures (see Annex C);
j) all load-resisting elements essential to the integrity of the completed structure and the details necessary to ensure the effectiveness of the load-resisting system in the completed structure;
k) the camber of beams, girders, and trusses;
l) the governing combinations of shears, moments, axial forces, torsions including pass through forces to be resisted by the connections;
m) the bracing required to stabilize compression elements including the size and location of stiffeners and/or reinforcement;
n) the types of bolts, the pretensioning requirements, and the designation of joints as bearing or slip-critical (see Clause 22.2);
o) the type and configuration details of structural connections that are critical for ductile seismic response; and
p) the locations and dimensions of protected zones (see Clause 27.1.9).

4.2.3

Revisions to design documents shall be clearly indicated and dated.

4.2.4
Provided that all requirements for the structural steel are shown on the structural documents, architectural, electrical, and mechanical documents may be used as supplements to the structural documents to define the detail configurations and construction information.

4.3 Fabrication and erection documents

4.3.1 Connection design details
Connection design details shall be prepared before the preparation of shop details and submitted to the structural designer for confirmation that the intent of the design is met. Connection design details shall provide details of standard and non-standard connections and other data necessary for the preparation of shop details. Connection design details shall be referenced to the design documents, erection drawings, or both.

4.3.2 Shop details
Shop details shall
a) be prepared before fabrication and submitted to the structural designer for review;
b) provide complete information for the fabrication of various members and components of the structure, including the
   i) required material and product standards;
   ii) location, type, and size of all mechanical fasteners;
   iii) bolt installation requirements; and
   iv) welds; and
c) provide the locations and dimensions of the protected zones and a complete description of the fabrication operations that are prohibited in protected zones.

4.3.3 Erection diagrams
Erection diagrams shall be submitted to the designer for review. Erection diagrams are general arrangement drawings that should show the principal dimensions of the structure, piece marks, sizes of the members, all steel load-resisting elements essential to the integrity of the completed structure, size and type of bolts, field welds, bolt installation requirements, elevations of column bases, all necessary dimensions and details for setting anchor rods, and any other information necessary for the assembly of the structure. Erection diagrams shall provide the locations and dimensions of the protected zones and a complete description of the erection operations that are prohibited in protected zones.

4.3.4 Erection procedures
Erection procedures shall outline the construction methods, erection sequence, temporary bracing requirements, and other engineering details necessary for shipping, erecting, and maintaining the stability of the steel frame. Erection procedures shall be supplemented by drawings and sketches that identify the location of permanent and temporary load-resisting elements essential to the integrity of the partially completed structure. Erection procedures shall be submitted for review when so specified.

4.3.5 Fieldwork details
Fieldwork details shall be submitted to the designer for review. Fieldwork details shall provide complete information for modifying fabricated members in the shop or on the job site. All operations required to modify the member shall be shown on the fieldwork details. If extra materials are necessary to make modifications, shop details shall be required.

5 Material — Standards and identification

5.1 Standards

5.1.1 General

Acceptable material and product standards and specifications for use under this Standard are specified in Clauses 5.1.3 to 5.1.10. Materials and products other than those specified may be used if approved. Approval shall be based on published specifications that establish the properties, characteristics, and suitability of the material or product to the extent and in the manner of those covered in specified standards and specifications.

5.1.2 Strength levels

The yield strength, F_y, and the tensile strength, F_u, used as the basis for design shall be the specified minimum values as given in the material and product standards and specifications. The levels reported on mill test certificates shall not be used as the basis for design.

5.1.3 Structural steel

Structural steel shall meet the requirements of CSA G40.20/G40.21, ASTM A500/A500M, ASTM 1085, ASTM A572/A572M, ASTM A913/A913M, or ASTM A992/A992M. The design properties for ASTM A500/A500M products shall be determined from wall thickness equal to 90% of the nominal wall thickness.

5.1.4 Sheet steel

Sheet steel shall meet the requirements of ASTM A1011/A1011M.

Other standards for structural sheet are listed in Section A2 of CSA S136. Only structural-quality sheet standards that specify chemical composition and mechanical properties shall be acceptable for conformance with this Standard. Mill test certificates that list the chemical composition and the mechanical properties shall be available, upon request, in accordance with Clause 5.2.1 a).

5.1.5 Cast steel

Cast steel shall meet the design requirements for weldability, strength, ductility, toughness, and surface finish.

Note: *Reference standards include ASTM A27/A27M, ASTM A148/A148M, ASTM A216/A216M, ASTM A352/A352M, and ASTM A958/A959M.*

5.1.6 Forged steel

Forged steel shall meet the requirements of ASTM A521/A521M or ASTM A668/A668M.

Note: *Before specifying metric bolts, the designer should check on their availability in the quantities required.*

5.1.7 Bolts and bolt assemblies

Bolts and bolt assemblies shall meet the requirements of ASTM A307, ASTM A325, ASTM A325M, ASTM A490, ASTM A490M, ASTM F1852, or ASTM F2280.

5.1.8 Welding electrodes

Welding electrodes shall meet the requirements of CSA W48, as applicable.

5.1.9 Studs

Studs shall meet the requirements of ASTM A108.

5.1.10 Anchor rods

Anchor rods shall meet the requirements of CSA G40.20/G40.21 or ASTM F1554.

5.2 Identification

5.2.1 Methods

The specifications (including type or grade, if applicable) of the materials and products used shall be identified by the following means, except as specified in Clauses 5.2.2 and 5.2.3:

a) mill test certificates or producer's certificates satisfactorily correlated to the materials or products to which they pertain; and
b) legible markings on the material or product made by its producer in accordance with the applicable material or product standard.

5.2.2 Unidentified structural steel

Unidentified structural steel shall not be used unless approved by the building designer. If the use of unidentified steel is authorized, F_y shall be taken as 210 MPa and F_u shall be taken as 380 MPa.

5.2.3 Tests to establish identification

Unidentified structural steel may be tested to establish identification when permitted by the building designer. Testing shall be done by an approved testing agency in accordance with CSA G40.20. The test results, taking into account both mechanical properties and chemical composition, shall form the basis for classifying the steel as to specification. Once classified, the specified minimum values for steel of that specification grade shall be used as the basis for design (see Clause 5.1.2).

5.2.4 Affidavit

The fabricator, if requested, shall provide an affidavit stating that the materials and products that have been used in fabrication conform to the applicable material or product standards called for by the design drawings or specifications.

6 Design requirements

6.1 General

6.1.1 Limit states

Steel structures designed in accordance with this Standard shall be safe from collapse during construction and designed to be safe and serviceable during the useful life of the structure. Limit states define the various types of collapse and unserviceability that are to be avoided. Those concerning safety are called the ultimate limit states (strength, overturning, sliding, and fracture) or the fatigue limit state (crack propagation) and those concerning serviceability are called the serviceability limit states (deflections, vibration, and permanent deformation). The object of limit states design calculations is to keep the probability of reaching a limit state below a certain value previously established for the given type of structure. This is achieved in this Standard by the use of load factors applied to the specified loads (see Article 4.1.2.1 of the *National Building Code of Canada* [*NBCC*]) and resistance factors applied to the specified resistances (see Clause 13 and Annex B of this Standard).

The various limit states are specified in Clause 6. Some of these relate to the specified loads and others to the factored loads. Camber, provisions for expansion and contraction, and corrosion protection are further design requirements related to serviceability and durability. All limit states shall be considered in the design.

6.1.2 Structural integrity

The general arrangement of the structural system and the connection of its members shall be designed to provide resistance to disproportionate collapse as a consequence of local failure. The requirements of this Standard generally provide a satisfactory level of structural integrity for steel structures.

Note: *Further guidance can be found in the* User's Guide — NBC 2015: Structural Commentaries (Part 4).

6.2 Loads

6.2.1 Specified loads

Except as provided for in Clause 7.1, the loads and influences specified in Article 4.1.2.1 of the *NBCC* shall be considered in the design of structural steelwork, taking into consideration that the regulatory authority might specify other loads in some circumstances.

6.2.2 Importance factors based on use and occupancy

The specified snow, wind, and earthquake loads shall be multiplied by the importance factors for the different importance categories for buildings in accordance with Article 4.1.2.1 of the *NBCC*. For buildings having a Low Importance Category, the factor of 0.8 for the ultimate limit states may be applied to the live load, L.

6.3 Requirements under specified loads

6.3.1 Deflection

6.3.1.1

Steel members and frames shall be proportioned so that deflections are within acceptable limits for the nature of the materials to be supported and for the intended use and occupancy. Consideration shall be given to the differential deflections of adjacent parallel framing members in the same plane.

Note: *In the absence of a more detailed evaluation, see Annex D for recommended values for deflections.*

6.3.1.2

Roofs shall be designed to withstand any additional loads likely to occur as a result of ponding (see also Clause 6.2.1).

Note: *Further guidance can be found in the* User's Guide — NBC 2015: Structural Commentaries (Part 4).

6.3.2 Camber

6.3.2.1

Camber of beams, trusses, or girders, if necessary, shall be stipulated on the design drawings. Generally, trusses and crane girders with a span of 25 m or greater should be cambered for approximately the dead-plus-half-live-load deflection.

Note: *See Clause 16 for requirements for open-web joists, Clause 15 for requirements for trusses, and Clause 28.6 for fabrication tolerances.*

6.3.2.2
Any special camber requirements necessary to bring a loaded member into proper relation with the work of other trades shall be stipulated on the design drawings.

Note: *See also Clause 6.3.1.1. See Clause 16.12.2.5 for maximum deviation in elevation between adjacent joists.*

6.3.3 Dynamic effects

6.3.3.1
Suitable provision shall be made in the design for the effect of live loads that induce impact, vibration, or both. In severe cases, e.g., structural supports for heavy machinery that causes substantial impact or vibration when in operation, the possibility of harmonic resonance, fatigue, or unacceptable vibration shall be investigated.

6.3.3.2
Special consideration shall be given to floor systems susceptible to vibration, e.g., large open floor areas free of partitions, to ensure that such vibration is acceptable for the intended use and occupancy.

Note: *For further information, see Annex E.*

6.3.3.3
Unusually flexible structures (generally those whose ratio of height to effective resisting width exceeds 4:1) shall be investigated for lateral vibrations under dynamic wind load. Lateral accelerations of the structure shall be checked to ensure that such accelerations are acceptable for the intended use and occupancy.

Note: *Information on lateral accelerations under dynamic wind loads can be found in the* User's Guide — NBC 2015: Structural Commentaries (Part 4).

6.3.4 Resistance to fatigue
Structural steelwork shall be designed to resist the effects of fatigue under specified loads in accordance with Clause 26.

6.4 Requirements under factored loads

6.4.1 Strength
Structural steelwork shall be proportioned to resist moments and forces resulting from the application of the factored loads acting in the most critical combination, taking into account the resistance factors specified in Clause 13.1.

6.4.2 Overturning
The building or structure shall be designed to resist overturning resulting from the application of the factored loads acting in the most critical combination, taking into account the importance category of the building as specified in Clause 6.2.2 and the resistance factors specified in Clause 13.1.

6.5 Expansion and contraction
Suitable provision shall be made for expansion and contraction commensurate with the service and erection conditions of the structure.

6.6 Corrosion protection

6.6.1
Steelwork shall have sufficient corrosion protection to minimize any corrosion likely to occur in the service environment.

6.6.2
Interiors of buildings conditioned for human comfort may be generally assumed to be non-corrosive environments; however, the need for corrosion protection shall be assessed and protection shall be furnished in those buildings where it is deemed to be necessary.

6.6.3
Corrosion protection of the inside surfaces of enclosed spaces permanently sealed from any external source of oxygen shall not be necessary.

6.6.4
The minimum required thickness of steelwork situated in a non-corrosive environment and therefore not requiring corrosion protection shall be in accordance with Clause 11.

6.6.5
Corrosion protection shall be provided by means of suitable alloying elements in the steel, by protective coatings, or by other effective means, either singly or in combination.

6.6.6
Localized corrosion likely to occur from trapped water, excessive condensation, or other factors shall be minimized by suitable design and detail. Where necessary, positive means of drainage shall be provided.

6.6.7
If the corrosion protection specified for steelwork exposed to the weather, or to other environments in which progressive corrosion can occur, is likely to require maintenance or renewal during the service life of the structure, the steelwork so protected, exclusive of fill plates and shims, shall have a minimum thickness of 4.5 mm.

6.7 Requirements under fire conditions
The fire endurance of structural steelwork for buildings shall be determined using CAN/ULC-S101. When permitted by the regulatory authority, a performance-based fire protection analysis and design of structural steelwork shall be conducted using the methods specified in Annex K.

Note: *Annex K is an "acceptable solution" that can be evaluated to determine compliance with the* NBCC *(Division A, Compliance, Objectives and Functional Statements).*

6.8 Brittle fracture
The risk of brittle fracture in steel structures subjected to tensile stresses shall be assessed.

Note: *See Annex L for guidance on material selection and details to minimize the risk of brittle fracture.*

6.9 Requirements under impulse loading
Structural steelwork that has been determined by the authority having jurisdiction to be potentially subjected to impulse loads shall follow design concepts and details that will mitigate collapse.

Notes:

1) *Annex L provides recommendations to prevent brittle fracture.*

2) *CSA S850 provides guidelines to account for blast loads.*

7 Factored loads and safety criterion

7.1 Safety during erection and construction

Suitable provision shall be made for loads imposed on the steel structure during its erection. During subsequent construction, suitable provision shall be made to support the construction loads on the steel structure with an adequate margin of safety.

7.2 Safety criterion and effect of factored loads for the ultimate limit states

7.2.1

The structural steelwork shall be designed to have sufficient strength or stability, or both, such that factored resistance is greater than or equal to the effect of factored loads, as follows:

$$\phi R \geq \Sigma \alpha i S_i$$

where the factored resistance is determined in accordance with the applicable clauses of this Standard and the effect of factored loads for the ultimate limit states is determined in accordance with Division B, Article 4.1.3.2 of the *NBCC*.

7.2.2

The effect of factored loads in force units shall be determined from the structural effect due to the specified loads, including importance factors due to use and occupancy (see Clause 6.2), multiplied by the load factors, α, for load combination cases in accordance with Division B, Article 4.1.3.2 of the *NBCC*.

8 Analysis of structure

8.1 General

In proportioning the structure to meet the design requirements of Clause 6, the methods of analysis specified in Clause 8 shall be used. The distribution of internal forces and bending moments shall be determined both under the specified loads to satisfy the requirements of serviceability and fatigue specified in Clause 6 and under the factored loads to satisfy strength and overturning requirements specified in Clause 7.

8.2 Types of construction

8.2.1 General

Three basic types of construction and associated design assumptions, i.e., “rigidly connected”, “simple”, and “semi-rigid” (see Clauses 8.2.2 to 8.2.4) may be used for all or part of a structure under this Standard. The distribution of internal forces and bending moments throughout the structure shall depend on the type or types of construction chosen and the forces to be resisted.

8.2.2 Rigidly connected and continuous construction

In this construction, the beams, girders, and trusses are rigidly connected to other frame members or are continuous over supports. Connections shall be generally designed to resist the bending moments and internal forces calculated by assuming that the angles between intersecting members remain unchanged as the structure is loaded.

8.2.3 Simple construction

Simple construction assumes that the ends of beams, girders, and trusses are free to rotate under load in the plane of loading. Resistance to lateral loads, including stability effects, shall be ensured by a suitable system of bracing or plate walls or by the design of part of the structure as rigidly connected or semi-rigid construction.

8.2.4 Semi-rigid (partially restrained) construction

8.2.4.1

In this construction, the angles between connected members change under applied bending moments and redistribute the moments between members while maintaining sufficient capacity to resist lateral loads and to provide adequate stability of the framework in accordance with Clause 8.4.

8.2.4.2

The design and construction of semi-rigid frameworks shall meet the following requirements:

a) The positive and negative moment/rotation response of the connections up to their maximum capacity shall have been established by test and either published in the technical literature or be available from a reputable testing facility.

b) The design of the structure shall be based on either linear analysis employing the secant stiffness of connections at ultimate load or incremental analyses following the non-linear test response of the connections.

c) Consideration shall be given to the effects of repeated vertical and horizontal loading and load reversals, with particular regard to incremental strain in connections and low-cycle fatigue.

8.3 Analysis methods

8.3.1 Elastic analysis

Under a particular loading combination, the forces and moments throughout all or part of the structure may be determined by an analysis that assumes that individual members behave elastically.

8.3.2 Plastic analysis

Under a particular loading combination, the forces and moments throughout all or part of the structure may be determined by a plastic analysis, provided that

a) the steel used has $F_y \leq 0.85F_u$ and exhibits the stress-strain characteristics necessary to achieve moment redistribution;

b) the width-to-thickness ratios meet the requirements of Class 1 sections as specified in Clause 11.2;

c) the members are braced laterally in accordance with the requirements of Clause 13.7;

d) web stiffeners are supplied on a member at a point of load application where a plastic hinge would form;

e) splices in beams or columns are designed to transmit 1.1 times the maximum calculated moment under factored loads at the splice location or $0.25M_p$, whichever is greater;

f) members are not subject to repeated heavy impact or fatigue; and

g) the influence of inelastic deformation on the strength of the structure is taken into account (see Clause 8.4).

8.4 Stability effects

8.4.1

The translational load effects produced by notional lateral loads, applied at each storey, equal to 0.005 times the factored gravity loads contributed by that storey, shall be added to the lateral loads for each load combination. The notional lateral loads shall be applied in both orthogonal directions independently when the three-dimensional effects of loading are included in the analysis of the structure.

8.4.2

The analyses referred to in Clause 8.3 shall include the sway effects in each storey produced by the vertical loads acting on the structure in its displaced configuration. The second-order effects that are due to the relative translational displacement (sway) of the ends of a member shall be determined from a second-order analysis. Elastic second-order effects may be accounted for by amplifying translational load effects obtained from a first-order elastic analysis by the factor

$$U_2 = \frac{1}{1 - \left[\frac{\Sigma C_f \Delta_f}{\Sigma V_f h}\right]}$$

Note: *For combinations including seismic loads,* U_2*, see Clause 27.1.8.2.*

9 Stability of structures and members

9.1 Stability of structures

The structural system shall be adequate to

a) resist the forces caused by factored loads;

b) transfer the factored loads to the foundations;

c) transfer forces from walls, floors, or roofs acting as shear-resisting elements or diaphragms to adjacent lateral-load-resisting elements; and

d) resist torsional effects.

See also Clause 8.4.

9.2 Stability of members

9.2.1 Initial misalignment at brace point

The initial misalignment of the member at a brace point, Δ_O, shall be taken such that the offset of that brace point relative to the adjacent brace points from the alignment shown on the drawings corresponds to the out-of-alignment tolerance specified in Clause 29.3.

9.2.2 Displacement of bracing systems

The displacement of the bracing system at the brace point, Δ_b, is the sum of the brace deformation, the brace connection deformation, and the brace support displacement. This displacement is due to the brace force and any other forces acting on the brace and shall be calculated in the direction perpendicular to the braced member at the brace point.

9.2.3 Function of bracing

Bracing systems provide lateral support to columns, the compression flange of beams and girders, or the compression chords of joists or trusses.

Bracing systems, including bracing members and their connections and supports, shall be proportioned to resist the forces that develop at the brace points and limit the lateral displacement of the brace points.

Bracing for beams shall provide lateral restraint to the compression flange, except that at cantilevered ends of beams and beams subject to double curvature, the restraint shall be provided at both top and bottom flanges unless otherwise accounted for in the design.

9.2.4 Twisting and lateral displacements

Twisting and lateral displacements shall be prevented at the supports of a member or element unless accounted for in the design.

9.2.5 Simplified analysis

Bracing systems shall be proportioned to have a strength perpendicular to the longitudinal axis of the braced member in the plane of buckling equal to at least 0.02 times the factored compressive force at each brace point in the member or element being braced, unless a detailed analysis is carried out in accordance with Clause 9.2.6 to determine the appropriate strength and stiffness of the bracing system. Any other forces acting on the bracing member shall also be taken into account. The displacement Δ_b shall not exceed Δ_o.

9.2.6 Detailed analysis

9.2.6.1 Second-order method

Forces acting in the member bracing system and its deformations shall be determined by means of a second-order elastic analysis of the member and its bracing system. This analysis shall include the most critical initial deformed configuration of the member and shall consider forces due to external loads. In the analysis, hinges may be assumed at brace points in the member or element being braced.

The displacement Δ_b shall not exceed Δ_o unless a greater value can be justified by analysis.

9.2.6.2 Direct method

Unless a second-order analysis is carried out in accordance with Clause 9.2.6.1 or a simplified analysis is carried out in accordance with Clause 9.2.5, bracing systems shall be proportioned at each brace point to have a factored resistance in the direction perpendicular to the longitudinal axis of the braced member in the plane of buckling equal to at least

$$P_b = \frac{\beta\left(\Delta_o + \Delta_b\right)C_f}{L_b}$$

where

P_b = force used to design the bracing system (when two or more points are braced, the forces P_b alternate in direction)

β = 2, 3, 3.41, 3.63, or 4 for 1, 2, 3, 4, or more equally spaced braces, respectively, unless a lesser value can be justified by the analysis

Δ_o = initial misalignment

Δ_b = displacement of the bracing system, assumed to be equal to Δ_o for the initial calculation of P_b

C_f = maximum factored compression in the segments bound by the brace points on either side of the brace point under consideration

L_b = length between braces

For flexural members, P_b shall be increased, as appropriate, when loads are applied above the shear centre or for beams in double curvature.

After P_b and any other forces acting on the bracing member are applied, the calculated displacement of the bracing system, Δ_b, shall not exceed Δ_o unless justified by analysis.

9.2.7 Slabs or decks

When bracing of the compression flange is effected by a slab or deck, the slab or deck and the means by which the calculated bracing forces are transmitted between the flange or chord and the slab or deck shall be adequate to resist a force in the plane of the slab or deck. This force, which shall be taken as at least 0.05 times the maximum force in the flange or chord unless a lesser amount can be justified by analysis, shall be considered to be uniformly distributed along the length of the compression flange or chord.

9.2.8 Accumulation of forces

Consideration shall be given to the probable accumulation of forces, C_f, when the bracing system restrains more than one member. When members are erected with random out-of-straightness, the initial misalignment may be taken as

$$(0.2 + 0.8 / \sqrt{n})\Delta_o$$

where
n = number of members or elements being braced

This reduction shall not be applied when member initial misalignments are dependent on each other and are likely to be in the same direction and of the same magnitude.

9.2.9 Torsion

Bracing systems for beams, girders, and columns designed to resist loads causing torsion shall be proportioned in accordance with Clause 14.10. Special consideration shall be given to the connection of asymmetric sections such as channels, angles, and Z-sections.

10 Design lengths and slenderness ratios

10.1 Simple span flexural members

Beams, girders, and trusses may be designed on the basis of simple spans, whose length may be taken as the distance between the centres of gravity of supporting members. Alternatively, the span length of beams and girders may be taken as the actual length of such members measured between centres of end connections. The length of trusses designed as simple spans may be taken as the distance between the extreme working points of the system of triangulation employed. The design of columns or other supporting members shall provide for the effect of any significant moment or eccentricity arising from the manner in which a beam, girder, or truss is connected or supported.

10.2 Continuous span flexural members

Beams, girders, or trusses having full or partial end restraint due to continuity or cantilever action shall be proportioned to carry all moments, shears, and other forces at any section, assuming the span, in general, to be the distance between the centres of gravity of the supporting members. Supporting members shall be proportioned to carry all moments, shears, and other forces induced by the continuity of the supported beam, girder, or truss.

10.3 Members in compression

10.3.1 General

A member in compression shall be designed on the basis of its effective length, *KL* (the product of the effective length factor, *K*, and the unbraced length, *L*).

Unless otherwise specified in this Standard, the unbraced length, *L*, shall be taken as the length of the compression member between the centres of restraining members. The unbraced length may differ for different cross-sectional axes of a compression member. At the bottom storey of a multi-storey structure or for a single-storey structure, *L* shall be taken as the length from the top of the base plate to the centre of restraining members at the next higher level.

The effective length factor, *K*, depends on the potential failure modes, whether by bending in-plane or buckling, as specified in Clauses 10.3.2 and 10.3.3.

Note: *See also Clause 9 on the effectiveness of the brace or support point.*

10.3.2 Failure mode involving bending in-plane

The effective length shall be taken as the actual length ($K = 1.0$) for beam-columns that would fail by in-plane bending, provided that, when applicable, the sway effects, including notional load effects, are included in the analysis of the structure to determine the end moments and forces acting on the beam-columns.

10.3.3 Failure mode involving buckling

The effective length for axially loaded columns that would fail by buckling and for beam-columns that would fail by out-of-plane (lateral-torsional) buckling shall be based on the rotational and translational restraint afforded at the ends of the unbraced length (see Annexes F and G).

10.4 Slenderness ratios

10.4.1 General

The slenderness ratio of a member in compression shall be taken as the ratio of the effective length, *KL*, to the corresponding radius of gyration, *r*. The slenderness ratio of a member in tension shall be taken as the ratio of the unbraced length, *L*, to the corresponding radius of gyration.

10.4.2 Maximum slenderness ratio

10.4.2.1

The slenderness ratio of a member in compression shall not exceed 200.

10.4.2.2
Except as specified in Clauses 15.2.7 and 16.5.6.1, the slenderness ratio of a member in tension shall not exceed 300. This limit may be waived if other means are provided to control flexibility, sag, vibration, and slack in a manner commensurate with the service conditions of the structure or if it can be shown that such factors are not detrimental to the performance of the structure or of the assembly of which the member is a part.

11 Width (or diameter)-to-thickness — Elements in compression

11.1 Classification of sections

11.1.1
For the purposes of this Standard, structural sections shall be designated as Class 1, 2, 3, or 4, depending on the maximum width (or diameter)-to-thickness ratios of the elements subject to compression, and as otherwise specified in Clauses 11.1.2 and 11.1.3, as follows:
a) Class 1 sections permit attainment of the plastic moment and subsequent redistribution of the bending moment;
b) Class 2 sections permit attainment of the plastic moment but need not allow for subsequent moment redistribution;
c) Class 3 sections permit attainment of the yield moment; and
d) Class 4 sections generally have elastic local buckling of elements in compression as the limit state of structural resistance.

11.1.2
Class 1 sections, when subject to flexure, shall have an axis of symmetry in the plane of loading and, when subject to axial compression, shall be doubly symmetric.

11.1.3
Class 2 sections, when subject to flexure, shall have an axis of symmetry in the plane of loading unless the effects of asymmetry of the section are included in the analysis.

11.2 Maximum width (or diameter)-to-thickness ratios of elements subject to compression
The maximum width (or diameter)-to-thickness ratios of elements subject to axial compression shall be as specified in Table 1 and those of elements subject to flexural compression shall be as specified in Table 2, for the specified section classification.

Sections that exceed the limits presented in Table 1 or Table 2 shall be classified as Class 4 sections. The factored axial compressive resistance of Class 4 sections shall be calculated in accordance with Clause 13.3.5. The factored bending resistance of Class 4 sections shall be calculated in accordance with Clause 13.5.

11.3 Width and thickness

11.3.1

For elements supported along only one edge parallel to the direction of compressive force, the width, b_{el}, shall be taken as follows:

a) plates: shall be the distance from the free edge to the first row of fasteners or line of welds;
b) legs of angles, flanges of channels and Z's, and stems of T's: the full nominal dimension; and
c) flanges of beams and T's: one-half of the full nominal dimension.

11.3.2

For elements supported along two edges parallel to the direction of compressive force, the width shall be taken as follows:

a) flange or diaphragm plates in built-up sections: the width, b_{el}, shall be the distance between adjacent lines of fasteners or lines of welds;
b) flanges, b_{el}, and webs, h, of rectangular hollow sections (HSS) shall be the nominal outside dimension less four times the wall thickness;
c) webs of built-up sections: the width, h, shall be the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used; and
d) webs of hot-rolled sections: the width, h, shall be the clear distance between flanges.

11.3.3

The thickness of elements, t or w, shall be taken as the nominal thickness. For tapered flanges of rolled sections, the thickness shall be taken as the nominal thickness halfway between a free edge and the corresponding face of the web.

12 Gross and net areas

12.1 Application

Members in tension shall be proportioned on the basis of the areas associated with the potential failure modes. Members in compression shall be proportioned on the basis of the gross area associated with the potential failure mode.

Note: *For beams and girders, see Clause 14.*

12.2 Gross area

Gross area shall be calculated by summing the products of the thickness and the gross width of each element (flange, web, leg, plate), as measured normal to the axis of the member.

12.3 Net area

12.3.1 General

The net area, A_n, shall be determined by summing the critical net areas, A_n, of each segment along a potential path of minimum resistance calculated as follows:

a) for a segment normal to the force (i.e., in direct tension):
$A_n = w_n t$
b) for a segment inclined to the force between openings (e.g., bolt holes) but not parallel to the force:

$$A_n = w_n t + \frac{s^2 t}{4g}$$

12.3.2 Allowance for bolt holes

In calculating w_n, the width of bolt holes shall be taken as 2 mm larger than the specified hole dimension. If drilled holes are used, this allowance may be waived.

12.3.3 Effective net area — Shear lag

12.3.3.1

When fasteners transmit load to each of the cross-sectional elements of a member in tension in proportion to their respective areas, the effective net area shall be taken as the net area, i.e., $A_{ne} = A_n$.

12.3.3.2

When bolts transmit load to some but not all of the cross-sectional elements and when the critical net area includes the net area of unconnected elements, the effective net area shall be taken as follows:

a) for WWF, W, M, or S shapes with flange widths not less than two-thirds the depth, and for structural tees cut from these shapes, when only the flanges are connected with three or more transverse lines of fasteners:
   $A_{ne} = 0.90A_n$
b) for angles connected by only one leg with
   i) four or more transverse lines of fasteners:
      $A_{ne} = 0.80A_n$
   ii) fewer than four transverse lines of fasteners:
      $A_{ne} = 0.60A_n$
c) for all other structural shapes connected with
   i) three or more transverse lines of fasteners:
      $A_{ne} = 0.85A_n$
   ii) two transverse lines of fasteners:
      $A_{ne} = 0.75A_n$

12.3.3.3

When a tension load is transmitted by welds, the effective net area, A_{ne}, shall be computed as the sum of the effective net areas of the elements, A_{n1}, A_{n2}, and A_{n3}, as applicable, but shall not exceed A_g. The net areas of the connected plate elements shall be defined as follows:

a) for elements connected by transverse welds, A_{n1}:
   $A_{n1} = wt$
b) for elements connected by longitudinal welds along two parallel edges, A_{n2}:
   i) when $L \geq 2w$:
      $A_{n2} = 1.00wt$
   ii) when $2w > L \geq w$:
      $A_{n2} = 0.50wt + 0.25Lt$
   iii) when $w > L$:
      $A_{n2} = 0.75Lt$

   where
   L = average length of welds on the two edges
   w = plate width (distance between welds)

c) for elements connected by a single longitudinal weld, A_{n3}:
   i) when $L \geq w$:

$$A_{n3} = \left(1 - \frac{\bar{x}}{L}\right) wt$$

   ii) when $w > L$:
   $A_{n3} = 0.50Lt$

where
$\bar{x}$ = eccentricity of the weld with respect to centroid of the connected element
L = length of weld in the direction of the loading

The outstanding leg of an angle shall be considered connected by the (single) line of weld along the heel.

12.3.3.4
When round or rectangular HSS members are slotted and welded to a plate, the effective net area, A_{ne}, of the HSS member under concentric tension shall be taken as follows:

$$A_{ne} = A_n\left(1.1 - \frac{\bar{x}'}{L_w}\right) \geq 0.8A_n, \text{ when } \frac{\bar{x}'}{L_w} > 0.1$$

$$A_{ne} = A_n, \text{ when } \frac{\bar{x}'}{L_w} \leq 0.1$$

where
$\bar{x}'$ = the distance between the centre of gravity of half of the HSS cross section taken from the edge of the connection plate
L_w = the length of a single weld segment on the HSS (the usual case has the total weld length being $4L_w$)

12.3.3.5
Larger values of the effective net area may be used if justified by test or rational analysis, but shall not exceed A_g.

12.3.4 Angles
For angles, the gross width shall be the sum of the widths of the legs minus the thickness. The gauge for holes in opposite legs shall be the sum of the gauges from the heel of the angle minus the thickness.

12.3.5 Plug or slot welds
In calculating the net area of a member across plug or slot welds, the weld metal shall not be taken as adding to the net area.

12.4 Pin-connected members in tension

12.4.1 Effective net areas
Two effective net areas shall be computed as follows
a) The effective net area for tension, A_{net} shall be taken as $2tb_e$
b) The effective net area for shear rupture, A_{nes} shall be taken as $2t(a + d/2)$

where
a = shortest distance parallel to the tensile force from the edge of the pin hole to the end of the tension member pin plate

b_e = $2t$ + 16 mm but not to exceed the actual distance from the edge of the hole to the edge of the part normal to the tensile force

d = diameter of pin

12.4.2 Detail requirements

The hole of the pin shall be located on the longitudinal member axis as defined by the centroid of the member cross section. The diameter of a pin hole shall be not more than 1 mm larger than the diameter of the pin when relative movement between connected parts under full service loads is required. At the centre of the pin hole the width of the plate, measured normal to the direction of the force shall be not less than $2b_e + d$. The distance from the edge of the hole to the edge of the pin plate on either side of the axis of the member axis, measured at an angle of 45° or less to the axis of the member, shall be not less than a.

13 Member and connection resistance

13.1 Resistance factors

Unless otherwise specified, resistance factors, ϕ, applied to resistances specified in this Standard shall be taken as follows:

a) structural steel: $\phi = 0.90$ and $\phi_u = 0.75$;
b) reinforcing steel bars: $\phi_r = 0.85$;
c) bolts: $\phi_b = 0.80$;
d) shear connectors: $\phi_{sc} = 0.80$;
e) beam web bearing, interior: $\phi_{bi} = 0.80$ (see Clause 14.3.2);
f) beam web bearing, end: $\phi_{be} = 0.75$ (see Clause 14.3.2);
g) bearing of bolts on steel: $\phi_{br} = 0.80$;
h) weld metal: $\phi_w = 0.67$;
i) anchor rods: $\phi_{ar} = 0.67$; and
j) concrete: $\phi_c = 0.65$.

The factored resistances so determined, in order to meet the strength requirements of this Standard, shall be greater than or equal to the effect of factored loads determined in accordance with Clause 7.2.

13.2 Axial tension

The factored tensile resistance, T_r, developed by a member subjected to an axial tensile force shall be taken as follows:

a) the least of
   i) $T_r = \phi A_g F_y$;
   ii) T_r = resistance determined using Clause 13.11; and
   iii) $T_r = \phi_u A_{ne} F_u$; and
b) for pin connections, the least of
   i) $T_r = 0.75\phi A_n F_y$;
   ii) $T_r = \phi_u A_{net} F_u$; and
   iii) $T_r = 0.6\phi_u A_{nes} F_u$.

13.3 Axial compression

13.3.1 Flexural buckling of doubly symmetric shapes

The factored axial compressive resistance, C_r, of doubly symmetric shapes meeting the requirements of Table 1 shall be taken as

$$C_r = \frac{\phi A F_y}{(1 + \lambda^{2n})^{\frac{1}{n}}}$$

where

n = 1.34 for hot-rolled, fabricated structural sections and hollow structural sections manufactured in accordance with CSA G40.20, Class C (cold-formed non-stress-relieved), ASTM A500, or ASTM A1085

= 2.24 for doubly symmetric welded three-plate members with flange edges oxy-flame-cut and hollow structural sections manufactured in accordance with CSA G40.20, Class H (hot-formed or cold-formed stress-relieved) and ASTM A1085 with Supplement S1

$$\lambda = \sqrt{\frac{F_y}{F_e}}$$

where

$$F_e = \frac{\pi^2 E}{(\frac{KL}{r})^2}$$

Doubly symmetric shapes that can be governed by torsional buckling shall also meet the requirements of Clause 13.3.2.

13.3.2 Flexural, torsional, or flexural-torsional buckling

The factored compressive resistance, C_r, of asymmetric, singly symmetric, and cruciform or other doubly symmetric sections not covered under Clause 13.3.1 shall be computed using the expressions given in Clause 13.3.1 with a value of n = 1.34 and the value of F_e taken as follows:

a) for doubly symmetric sections (e.g., cruciform) and point symmetric sections (e.g., Z-sections), the least of F_{ex}, F_{ey}, and F_{ez};

b) for singly symmetric sections (e.g., double angles, channels, and T-sections), with the y-axis taken as the axis of symmetry, the lesser of F_{ex} and F_{eyz}
where

$$F_{eyz} = \frac{F_{ey} + F_{ez}}{2\Omega}\left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}\Omega}{\left(F_{ey} + F_{ez}\right)^2}}\right]$$

c) for asymmetric sections (e.g., bulb angles), the smallest root of

$$\left(F_e - F_{ex}\right)\left(F_e - F_{ey}\right)\left(F_e - F_{ez}\right) - F_e^2\left(F_e - F_{ey}\right)\left(\frac{x_o}{\bar{r}_o}\right)^2 - F_e^2(F_e - F_{ex})\left(\frac{y_o}{r_o}\right)^2 = 0$$

where

F_{ex}, F_{ey}, and F_{ez} are calculated with respect to the principal axes:

$$F_{ex} = \frac{\pi^2 E}{\left(\frac{K_x L_x}{r_x}\right)^2}$$

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{K_y L_y}{r_y}\right)^2}$$

$$F_{ez} = \left(\frac{\pi^2 E C_w}{(K_z L_z)^2} + GJ\right)\frac{1}{A\bar{r}_o^2}$$

where

K_z = effective length factor for torsional buckling, conservatively taken as 1.0

$\bar{r}_o^2 = x_o^2 + y_o^2 + r_x^2 + r_y^2$

$\Omega = 1 - \left[\frac{x_o^2 + y_o^2}{\bar{r}_o^2}\right]$

where

x_o, y_o = principal coordinates of the shear centre with respect to the centroid of the cross-section

Note: *For equal-leg double angles connected back-to-back to a common gusset plate, flexural-torsional buckling is not a controlling limit state.*

13.3.3 Single-angle members in compression

13.3.3.1 General

The factored compressive resistance, C_r, of single-angle members may be calculated neglecting the effects of eccentricity if the appropriate slenderness as specified in Clause 13.3.3.2 or 13.3.3.3 is used, provided that

a) members are loaded at the ends in compression through the same one leg;
b) members are attached by welding or by minimum two-bolt connections; and
c) there are no intermediate transverse loads.

The factored compressive resistance, C_r, of single-angle members meeting the requirements of Table 1 shall be taken as

$$C_r = \frac{\phi A F_y}{(1 + \lambda^{2n})^{\frac{1}{n}}}$$

where

n = 1.34

$\Lambda = \sqrt{\frac{F_y}{F_e}}$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

13.3.3.2 Individual members and planar trusses

For equal-leg angles or unequal-leg angles with leg length ratios (b_l/b_s) less than 1.7 and connected through the longer leg that are individual members or are members of planar trusses with adjacent web members attached to the same side of the gusset plate or chord:

a) $0 \le \frac{L}{r_x} \le 80: \ \frac{KL}{r} = 72 + 0.75\frac{L}{r_x}$

b) $\frac{L}{r_x} > 80: \ \frac{KL}{r} = 32 + 1.25\frac{L}{r_x} \le 200$

For unequal-leg angles with leg length ratios (b_l/b_s) less than 1.7 and connected through the shorter leg, KL/r shall be increased by adding $4[(b_l/bs)^2 - 1]$. KL/r shall be not less than $0.95L/r'_y$

where

L = length of member between work points at truss chord centrelines

b_l = longer leg of angle

b_s = shorter leg of angle

r_x = radius of gyration of single-angle member about geometric axis parallel to connected leg

r'_y = radius of gyration of single-angle member about minor principal axis

13.3.3.3 Box and space trusses

For equal-leg angles or unequal-leg angles with leg length ratios (b_l/b_s) less than 1.7 and connected through the longer leg that are members of box or space trusses with adjacent web members attached to the same side of the gusset plate or chord:

a)

$$0 \le \frac{L}{r_x} \le 75: \ \frac{KL}{r} = 60 + 0.8\frac{L}{r_x}$$

b)

$$\frac{L}{r_x} > 75: \ \frac{KL}{r} = 45 + \frac{L}{r_x} \le 200$$

For unequal-leg angles with leg length ratios (b_l/b_s) less than 1.7 and connected through the shorter leg, KL/r shall be increased by adding $6\,[(b_l/bs)^2 - 1]$. KL/r shall be not less than $0.82L/r'_y$.

13.3.3.4 Other members

Single-angle members with different end conditions from those described in Clause 13.3.3.1, leg length ratios (b_l/b_s) greater than 1.7, adjacent web members attached to opposite sides of the gusset plate or chord, or transverse loading shall be designed for compressive resistance, C_r, with Clause 13.3.2, accounting for the effects of eccentricity.

13.3.4 Segmented members in compression

The factored compressive resistance of segmented columns shall be determined using a rational method. Notional loads need not be applied between in-plane lateral supports.

13.3.5 Members in compression subjected to elastic local buckling

The factored compressive resistance, C_r, for sections that exceed the width (or diameter)-to-thickness ratios specified in Table 1 shall be determined as either

a)

$$C_r = \frac{\phi A_e F_y}{(1+\lambda^{2n})^{\frac{1}{n}}}$$

where

$$\lambda = \sqrt{\frac{F_y}{F_e}}$$

with an effective area, A_e, calculated using reduced element widths meeting the maximum width-to-thickness ratio specified in Table 1; or

b)

$$C_r = \frac{\phi A F_{ye}}{\left(1+\lambda_{ye}{}^{2n}\right)^{\frac{1}{n}}}$$

where

$$\lambda_{ye} = \sqrt{\frac{F_{ye}}{F_e}}$$

with an effective yield stress, F_{ye}, determined from the maximum width (or diameter)-to-thickness ratio meeting the limit specified in Table 1.

The elastic buckling stress, F_e, shall be calculated using Clause 13.3.1, 13.3.2, or 13.3.3, as applicable, and using gross section properties.

13.4 Shear

13.4.1 Webs of flexural members with two flanges

13.4.1.1 Elastic analysis

The factored shear resistance, V_r, developed by the web of a flexural member shall be taken as

$V_r = \phi A_w F_s$

where

A_w = shear area (dw for rolled shapes and hw for girders, $2ht$ for rectangular HSS)

F_s = as follows:

a) for unstiffened webs:

i) when $\frac{h}{w} \leq \frac{1014}{\sqrt{F_y}}$:

$F_s = 0.66F_y$

ii) when $\frac{1014}{\sqrt{F_y}} < \frac{h}{w} \leq \frac{1435}{\sqrt{F_y}}$:

$$F_s = \frac{670\sqrt{F_y}}{(h/w)}$$

iii) when $\frac{h}{w} > \frac{1435}{\sqrt{F_y}}$:

$$F_s = \frac{961\ 200}{(h/w)^2}$$

b) for stiffened webs:

i) when $\frac{h}{w} \leq 439\sqrt{\frac{k_v}{F_y}}$:

$F_s = 0.66F_y$

ii) when $439\sqrt{\frac{k_v}{F_y}} < \frac{h}{w} \leq 502\sqrt{\frac{k_v}{F_y}}$:

$F_s = F_{cri}$

iii) when $502\sqrt{\frac{k_v}{F_y}} < \frac{h}{w} \leq 621\sqrt{\frac{k_v}{F_y}}$:

$F_s = F_{cri} + k_a(0.50F_y - 0.866F_{cri})$

iv) when $621\sqrt{\frac{k_v}{F_y}} < \frac{h}{w}$:

$F_s = F_{cre} + k_a(0.50F_y - 0.866\ F_{cre})$

where

k_v = shear buckling coefficient, as follows:

1) when $a / h < 1$

$$k_v = 4 + \frac{5.34}{(a/h)^2}$$

2) when $a / h \geq 1$

$$k_v = 5.34 + \frac{4}{(a/h)^2}$$

where

a / h = aspect ratio = ratio of the distance between stiffeners to web depth

where

$$F_{cri} = 290\frac{\sqrt{F_y k_v}}{(h/w)}$$

K_a = aspect coefficient

$$= \frac{1}{\sqrt{1+(a/h)^2}}$$

$$F_{cre} = \frac{180\,000\ k_v}{(h/w)^2}$$

13.4.1.2 Combined shear and moment in stiffened web beams

Transversely stiffened web members depending on tension field action to carry shear shall be proportioned to satisfy the requirements of Clause 14.6 for combined shear and moment.

13.4.1.3 Tubular members and concrete-filled tubular members

The shear resistance, V_r, of Class 1 and 2 tubular members and concrete-filled tubular members where local wall buckling is prevented shall be taken as

$V_r = 0.66\phi\,(A/2)F_y$

where
A = cross-sectional area of the tubular member portion of the concrete-filled member

13.4.2 Plastic analysis

In structures designed on the basis of a plastic analysis as defined in Clause 8.3.2, the factored shear resistance, V_r, developed by the web of a flexural member subjected to shear shall be taken as

$V_r = 0.8\phi A_w F_s$

where F_s is determined in accordance with Clause 13.4.1.1.

13.4.3 Webs of flexural members not having two flanges

The factored shear resistance for cross-sections not having two flanges (e.g., solid rectangles, rounds, and T_s) shall be determined by rational analysis. The factored shear stress at any location in the cross-section shall be taken as not greater than $0.66\phi\,F_y$ and shall be reduced where shear buckling is a consideration.

13.4.4 Pins

The total factored shear, V_r, resistance of the nominal area of pins shall be taken as

$V_r = 0.66\phi A F_y$

13.4.5 Gusset plates and coped beams

The shear resistance of gusset plates and the shear resistance at the ends of coped beams shall be computed in accordance with Clause 13.11.

13.5 Bending — Laterally supported members

The factored moment resistance, M_r, developed by a member subjected to uniaxial bending moments about a principal axis where effectively continuous lateral support is provided to the compression flange, or where the member has no tendency to buckle laterally, shall be taken as follows:

a) for Class 1 and Class 2 sections (except that singly symmetric I-sections and T-sections shall not yield under service loads):
$M_r = \phi Z F_y$
$= \phi M_p$

b) for Class 3 sections:
$M_r = \phi S F_y$
$= \phi M_y$

c) for Class 4 sections:
   i) when both the web and compression flange slenderness exceed the limits for Class 3 sections, the value of M_r shall be determined in accordance with CSA S136. The calculated value, F_y', applicable to cold-formed members, shall be determined using only the values for F_y and F_u that are specified in the relevant structural steel material standard;
   ii) when the flanges meet the requirements of Class 3 but the web slenderness exceeds the limit for Class 3, the requirements of Clause 14 shall apply; and
   iii) when the web meets the requirements of Class 3 but the flange slenderness exceeds the limit for Class 3, M_r shall be calculated as follows:
   $M_r = \phi\,S_e\,F_y$
   where

S_e = effective section modulus determined using an effective flange width of $670t/\sqrt{F_y}$ for flanges supported along two edges parallel to the direction of stress and an effective width of $200t/\sqrt{F_y}$ for flanges supported along one edge parallel to the direction of stress. For flanges supported along one edge, b_{el}/t shall not exceed 60.

Alternatively, the moment resistance may be calculated using an effective yield stress determined from the flange width-to-thickness ratio meeting the Class 3 limit.

13.6 Bending — Laterally unsupported members

Where continuous lateral support is not provided to the compression flange of a member subjected to uniaxial strong axis bending, the factored moment resistance, M_r, of a segment between effective brace points shall be determined as follows:

a) For doubly symmetric Class 1 and 2 sections, except closed square and circular sections:

i) when $M_u > 0.67M_p$:

$$M_r = 1.15\phi M_p\left[1 - \frac{0.28M_p}{M_u}\right] \le \phi M_p$$

ii) when $M_u \le 0.67M_p$:

$M_r = \phi M_u$

where the critical elastic moment of the unbraced segment, M_u, is given by

$$M_u = \frac{\omega_2 \pi}{L}\sqrt{EI_yGJ + \left(\frac{\pi E}{L}\right)^2 I_y C_w}$$

where

$$\omega_2 = \frac{4M_{max}}{\sqrt{M_{max}^2 + 4M_a^2 + 7M_b^2 + 4M_c^2}} \le 2.5$$

where

C_w = warping torsional constant, taken as 0 for rectangular hollow structural sections

J = St. Venant torsional constant

L = length of unbraced segment of beam

M_{max} = maximum factored bending moment magnitude in unbraced segment

M_a = factored bending moment at one-quarter point of unbraced segment

M_b = factored bending moment at midpoint of unbraced segment

M_c = factored bending moment at three-quarter point of unbraced segment

ω_2 = coefficient to account for increased moment resistance of a laterally unsupported doubly symmetric beam segment when subject to a moment gradient

Where the bending moment distribution within the unbraced segment is effectively linear, the equivalent moment factor, ω_2, may be taken as

$1.75 + 1.05\kappa + 0.3\kappa^2 \le 2.5$

where

κ = ratio of the smaller factored moment to the larger factored moment at opposite ends of the unbraced length (positive for double curvature and negative for single curvature)

For unbraced beam segments loaded above the shear centre between brace points, where the method of load delivery to the member provides neither lateral nor rotational restraint to the member, the associated destabilizing effect shall be taken into account using a rational method. For loads applied at the level of the top flange, in lieu of a more accurate analysis, M_u may be determined using ω_2 = 1.0 and using an effective length, for pinned-ended beams, equal to 1.2 L and, for all other cases, 1.4 L.

b) For doubly symmetric Class 3 and 4 sections, except closed square and circular sections, and for channels:
   i) when $M_u > 0.67M_y$:

$$M_r = 1.15\phi M_y\left[1 - \frac{0.28M_y}{M_u}\right]$$

   but not greater than ϕM_y for Class 3 sections and the value specified in Clause 13.5 c) iii) for Class 4 sections; and
   ii) when $M_u \le 0.67M_y$:
   $M_r = \phi M_u$
   where M_u and ω_2 are as specified in Item a) ii).

c) For closed square and circular sections, M_r shall be determined in accordance with Clause 13.5.

d) For cantilever beams, a rational method of analysis taking into account the lateral and torsional restraint conditions at the supports and tip of the cantilever, as well as the loading conditions and the flexibility of the backspan, shall be used.

e) For singly symmetric (monosymmetric) Class 1, 2, or 3 I-sections and T-sections, lateral-torsional buckling strength shall be checked separately for each flange that experiences compression under factored loads at any point along its unbraced length, as follows (except that these sections shall not yield under service loads):
   i) when $M_u > M_{yr}$:

$$M_r = \phi\left[M_p - \left(M_p - M_{yr}\right)\left(\frac{L - L_u}{L_{yr} - L_u}\right)\right] \le \phi M_p$$

   except for Class 3 sections, as well as Class 1 and 2 T-sections where at any point within the unbraced segment the stem tip is in compression, where M_p is replaced with M_y

   where

   $M_{yr} = 0.7S_xF_y$ with S_x taken as the smaller of the two potential values

   L_{yr} = length L obtained by setting $M_u = M_{yr}$

$$L_u = 1.1r_t\sqrt{E / F_y} = \frac{490r_t}{\sqrt{F_y}}$$

   where

$$r_t = \frac{b_c}{\sqrt{12\left(1 + \frac{h_c w}{3b_c t_c}\right)}}$$

   where

   h_c = depth of the web in compression
   b_c = width of compression flange
   t_c = thickness of compression flange

   ii) when $M_u \le M_{yr}$:
   $M_r = \phi M_u$
   where the critical elastic moment of the unbraced segment, M_u, is given by

$$M_u = \frac{\omega_3 \pi^2 E I_y}{2L^2}\left[\beta_x + \sqrt{\beta_x^2 + 4\left(\frac{GJL^2}{\pi^2 E I_y} + \frac{C_w}{I_y}\right)}\right]$$

and where in lieu of more accurate values the section properties βx and C_w may be evaluated as

$$\beta_x = 0.9(d-t)\left(\frac{2I_{yc}}{I_y} - 1\right)\left(1 - \left(\frac{I_y}{I_x}\right)^2\right)$$

$$C_w = \frac{I_{yc} I_{yt} (d-t)^2}{I_y}$$

where
β_x = asymmetry parameter for singly symmetric beams
I_{yc} = moment of inertia of the compression flange about the y-axis
I_{yt} = moment of inertia of the tension flange about the y-axis

and when singly symmetric beams are in single curvature
ω_3 = ω_2 for beams with two flanges
= 1.0 for T-sections

in all other cases
ω_3 = ω_2 $(0.5 + 2\,(I_{yc}/I_y)^2)$ but ≤ 1.0 for T-sections

For unbraced beam segments loaded above the section mid-height and between brace points, where the method of load delivery to the member provides neither lateral nor rotational restraint to the member, the associated destabilizing effect shall be taken into account using a rational method.
For other singly symmetric shapes, a rational method of analysis shall be used.

f) For biaxial bending, the member shall meet the following requirement:

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \leq 1.0$$

13.7 Lateral bracing for members in structures analyzed plastically

Members in structures or portions of structures in which the distributions of moments and forces have been determined by a plastic analysis shall be braced to resist lateral and torsional displacement at all hinge locations. However, bracing shall not be required at the location of the last hinge to form in the failure mechanism assumed as the basis for proportioning the structure. The laterally unsupported distance, L_{cr}, from braced hinge locations to the nearest adjacent point on the frame similarly braced shall not exceed the following:

a) for static plastic analysis and for seismic design in accordance with Clauses 27.3 and 27.7.9.3:

$$\frac{L_{cr}}{r_y} = \frac{25\,000 + 15\,000\,\kappa}{F_y}$$

b) for seismic design in accordance with Clauses 27.2 and 27.9:

$$\frac{L_{cr}}{r_y} = \frac{17\,250 + 15\,500\,\kappa}{F_y}$$

where κ is as specified in Clause 13.6 a).

Except as specified in Items a) and b), the maximum unsupported length of members in structures analyzed plastically need not be less than that permitted for the same members in structures analyzed elastically.

13.8 Axial compression and bending

13.8.1 General

In Clause 13.8, a distinction is made between braced and unbraced frames. A frame without bracing is classified as unbraced. A frame with bracing is classified as braced if its sway stiffness is at least five times that of the frame with only the existing moment connections and without the bracing; otherwise, it is classified as unbraced. For members not contributing through bending to the lateral strength and stability of the structure, the conditions applicable to braced frames may be used.

Note: *For segmented members, the in-plane compressive resistance may be determined assuming pinned end connections. See Clause 13.3.4.*

13.8.2 Member strength and stability — Class 1 and Class 2 sections of I-shaped members

Members required to resist both bending moments and an axial compressive force shall be proportioned so that

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \le 1.0$$

where

C_f and M_f = the maximum load effects, including stability effects as specified in Clause 8.4

β = $0.6 + 0.4\lambda_y \le 0.85$

The capacity of the member shall be examined for

a) cross-sectional strength (members in braced frames only) with $\beta = 0.6$, in which case
   i) C_r shall be as specified in Clause 13.3, with the value $\lambda = 0$;
   ii) M_r shall be as specified in Clause 13.5 (for the appropriate class of section); and
   iii) U_{1x} and U_{1y} shall be as specified in Clause 13.8.4, but not less than 1.0;
b) overall member strength, in which case
   i) C_r shall be as specified in Clause 13.3, with the value $K = 1$, except that for uniaxial bending, C_r shall be based on the axis of bending (see also Clause 10.3.2);
   ii) M_r shall be as specified in Clause 13.5 (for the appropriate class of section);
   iii) U_{1x} and U_{1y} shall be taken as 1.0 for members in unbraced frames; and
   iv) U_{1x} and U_{1y} shall be as specified in Clause 13.8.4 for members in braced frames; and
c) lateral torsional buckling strength, when applicable, in which case
   i) C_r shall be as specified in Clause 13.3 and based on weak-axis or torsional-flexural buckling (see also Clause 10.3.3);
   ii) M_{rx} shall be as specified in Clause 13.6 (for the appropriate class of section);
   iii) M_{ry} shall be as specified in Clause 13.5 (for the appropriate class of section);
   iv) U_{1x} and U_{1y} shall be taken as 1.0 for members in unbraced frames;
   v) U_{1x} shall be as specified in Clause 13.8.4, but not less than 1.0, for members in braced frames; and
   vi) U_{1y} shall be as specified in Clause 13.8.4 for members in braced frames.
   In addition, the member shall meet the following requirement:

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \le 1.0$$

where M_{rx} and M_{ry} are as specified in Clause 13.5 or 13.6, as appropriate.

13.8.3 Member strength and stability — All classes of sections except Class 1 and Class 2 sections of I-shaped members

Members required to resist both bending moments and an axial compressive force shall be proportioned so that

$$\frac{C_f}{C_r} + \frac{U_{1x}M_{fx}}{M_{rx}} + \frac{U_{1y}M_{fy}}{M_{ry}} \le 1.0$$

where all terms are as specified in Clause 13.8.2.

The capacity of the member shall be examined for the following cases in the manner specified in Clause 13.8.2:

a) cross-sectional strength (members in braced frames and tapered members only);
b) overall member strength; and
c) lateral-torsional buckling strength.

In addition, for braced frames, the member shall meet the following requirement:

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \le 1.0$$

where M_{rx} and M_{ry} are as specified in Clause 13.5 or 13.6, as appropriate.

13.8.4 Value of U_1

In lieu of a more detailed analysis, the value of U_1 for the axis under consideration, accounting for the second-order effects due to the deformation of a member between its ends, shall be taken as

$$U_1 = \left[\frac{\omega_1}{1 - \frac{C_f}{C_e}}\right]$$

where ω_1 is as specified in Clause 13.8.5 and

$$C_e = \frac{\pi^2 EI}{L^2}$$

13.8.5 Values of ω_1

Unless otherwise determined by analysis, the following values shall be used for ω_1:

a) for members not subjected to transverse loads between supports:
   $\omega_1 = 0.6 - 0.4\kappa \ge 0.4$
   where
   κ = ratio of the smaller factored moment to the larger factored moment at opposite ends of the member length (positive for double curvature and negative for single curvature)

b) for members subjected to distributed loads or a series of point loads between supports:
   $\omega_1 = 1.0$

c) for members subjected to a concentrated load or moment between supports:
$\omega_1 = 0.85$
For the purpose of design, members subjected to a concentrated load or moment between supports (e.g., segmented columns) may be considered to be divided into segments at the points of load (or moment) application. Each segment shall then be treated as a member that depends on its own flexural stiffness to prevent sidesway in the plane of bending considered and ω_1 shall be taken as 0.85. In calculating the slenderness ratio for use in Clause 13.8, the total length of the compression member shall be used.
Note: *For references to more exact methods often justified for crane-supporting columns and similar applications, see Annex C.*

13.9 Axial tension and bending

13.9.1

Members required to resist both bending moments and an axial tensile force shall be proportioned so that

$$\frac{T_f}{T_r} + \frac{M_f}{M_r} \leq 1.0$$

where M_r is as specified in Clause 13.5.

13.9.2

Additionally, the following shall apply to laterally unsupported members:

a)

$$\frac{M_f}{M_r} - \frac{T_f Z}{M_r A} \leq 1.0 \text{ for Class 1 and Class 2 sections}$$

b)

$$\frac{M_f}{M_r} - \frac{T_f S}{M_r A} \leq 1.0 \text{ for Class 3 and Class 4 sections}$$

where M_r is as specified in Clause 13.6.

13.10 Load bearing

The factored bearing resistance in newtons, B_r, developed by a member or portion of a member subjected to bearing shall be taken as follows:

a) on the contact area of accurately cut or fitted parts:
$B_r = 1.50\phi F_y A$

b) on expansion rollers or rockers:

$$B_r = 0.00026\phi\left(\frac{R_1}{1-\frac{R_1}{R_2}}\right)LF_y^2$$

where

F_y = specified minimum yield point of the weaker part in contact

R_1 and L = radius and length, respectively, of the roller or rocker

R_2 = radius of the groove of the supporting plate

13.11 Block shear — Tension member, beam, and plate connections

The factored resistance for a potential failure involving the simultaneous development of tensile and shear component areas shall be taken as follows:

$$T_r = \phi_u \left[U_t A_n F_u + 0.6 A_{gv} \frac{(F_y + F_u)}{2} \right]$$

where

a) U_t is an efficiency factor and U_t = 1.0 is used for symmetrical blocks or failure patterns and concentric loading or is taken from the following for specific applications:

Connection type	U_t
Flange-connected T_s	1.0
Angles connected by one leg and stem-connected T_s	0.6
Coped beams	
One bolt line	0.9
Two bolt lines	0.3

b) A_n is the net area in tension, as specified in Clause 12; and

c) A_{gv} is the gross area in shear.
For steel grades with F_y > 460 MPa, $(F_y + F_u)/2$ shall be replaced with F_y in the determination of T_r.

The second term of the expression in this Clause may be used to calculate the potential plate tear-out resistance of one or more bolts along parallel planes tangent to the bolt hole(s) and directed towards the edge of the plate.

13.12 Bolts and local connection resistance

13.12.1 Bolts in bearing-type connections

13.12.1.1 General

For bolts subject to shear or tension, ϕ_b, shall be taken as 0.80.

13.12.1.2 Bolts in bearing and shear

The factored resistance developed at the bolts in a bolted joint subjected to bearing and shear shall be taken as the lesser of

a) the factored bearing resistance at bolt holes B_r (except for long slotted holes loaded perpendicular to the slot), B_r, as follows:
$B_r = 3\phi_{br} ntdF_u$

b) the factored bearing resistance perpendicular to long slotted holes, B_r, as follows:
$B_r = 2.4\phi_{br} ntdF_u$
where
ϕ_{br} = 0.8
F_u = tensile strength of the connected material

The reduced bearing resistance of holes close to the edge in the direction of the loading shall be accounted for by appropriate consideration of the resistance requirements of Clause 13.11; or

Note: *See also Clauses 13.2 and 13.11 for resistances of bolted parts and Clause 22.3 for limiting end and edge distances.*

c) the factored shear resistance of the bolts, V_r, as follows:
$V_r = 0.60\phi_b nmA_bF_u$
For lap splices with $L \geq 760$ mm:
$V_r = 0.50\phi_b nmA_bF_u$
When the bolt threads are intercepted by a shear plane, the factored shear resistance shall be taken as $0.70V_r$.

Note: *The specified minimum tensile strength, F_u, for bolts is given in the relevant ASTM Standard, e.g., for*
a) *ASTM A325M, F_u is* 830 MPa*;*
b) *ASTM A490M, F_u is* 1040 MPa*;*
c) *ASTM A325 or ASTM F1852 bolts 1 inch or less in diameter, F_u is* 825 MPa*;*
d) *ASTM A325 or ASTM F 1852 bolts greater than 1 inch in diameter, F_u is* 725 MPa*;*
e) *ASTM A490 or ASTM F2280 bolts, F_u is* 1035 MPa*; and*
f) *ASTM A307 Grade A bolts with heavy hex nuts as appropriate, per ASTM A563, F_u =* 410 MPa.

13.12.1.3 Bolts in tension

The factored tensile resistance, T_r, that can be developed by a bolt in a joint subjected to a factored tensile force, T_f, shall be taken as

$$T_r = 0.75\phi_b A_b F_u$$

The calculated factored tensile force, T_f, is independent of the pretension and shall be taken as the sum of the external load plus any tension caused by prying action.

Note: *See also Clause 26.5 for bolts in tension subjected to load combinations involving fatigue.*

13.12.1.4 Bolts in combined shear and tension

A bolt in a joint that is required to develop resistance to both tension and shear shall be proportioned so that

$$\left(\frac{V_f}{V_r}\right)^2 + \left(\frac{T_f}{T_r}\right)^2 \leq 1$$

where V_r is as specified in Clause 13.12.1.2 and T_r is as specified in Clause 13.12.1.3.

13.12.2 Bolts in slip-critical connections

13.12.2.1 General

For a slip-critical connection under the forces and moments produced by specified loads, slip of the assembly shall not occur. In addition, the effects of factored loads shall not exceed the resistances of the connection as specified in Clause 13.12.1.

13.12.2.2 Shear connections

The slip resistance, V_s, of a bolted joint, subjected to shear, V, shall be taken as

$$V_s = 0.53c_s k_s mnA_bF_u$$

where
c_s = the resistance factor for slip resistance of bolted joints

k_s = the mean slip coefficient as determined by tests carried out in accordance with "Testing method to Determine the Slip Coefficient for Coatings Used in Bolted Joints", Annex A, of RCSC *Specification for Structural Joints Using High-Strength Bolts*

See Table 3 for values of k_s and c_s.

When long slotted holes are used in slip-critical connections, slip resistance shall be taken as 0.75 V_s.

13.12.2.3 Connections in combined shear and tension

A bolt in a joint that is required to develop resistance to both tension and shear shall be proportioned so that the following relationship is satisfied for the specified loads:

$$\frac{V}{V_s} + 1.9\frac{T}{nA_bF_u} \leq 1.0$$

where V_s is the slip resistance specified in Clause 13.12.2.2.

13.13 Welds

13.13.1 General

The resistance factor, ϕ_w, for welded connections shall be taken as 0.67.

Note: *See Table 4 for matching electrode classifications for CSA G40.21 steels.*

13.13.2 Shear

13.13.2.1 Complete and partial joint penetration groove welds, and plug and slot welds

The factored shear resistance shall be taken as the lesser of

a) for the base metal:
$V_r = 0.67\phi_w A_m F_u$

b) for the weld metal:
$V_r = 0.67\phi_w A_w X_u$

where

A_m = shear area of effective fusion face

A_w = area of effective weld throat, plug, or slot

13.13.2.2 Fillet welds

The factored resistance for direct shear and tension- or compression-induced shear shall be taken as

$$V_r = 0.67\phi_w A_w X_u(1.00 + 0.50 \sin^{1.5}\theta)M_w$$

where

θ = angle, in degrees, of axis of weld segment with respect to the line of action of applied force (e.g., 0° for a longitudinal weld and 90° for a transverse weld)

M_w = strength reduction factor for multi-orientation fillet welds. For joints with a single weld orientation, M_w = 1.0; for joints with multiple weld orientations, for each segment

$$M_w = \frac{0.85 + \theta_1 / 600}{0.85 + \theta_2 / 600}$$

where

θ_1 = orientation of the weld segment under consideration

θ_2 = orientation of the weld segment in the joint that is nearest to 90°

Weld returns that are not accounted for in the joint capacity need not be considered a weld segment in the context of this Clause.

Where over-matched electrodes are used, the base metal capacity at the fusion face shall also be checked and may be considered to have the following strength:

$V_r = 0.67\phi_w A_m F_u$

13.13.2.3 Flare bevel groove welds for open-web steel joists

The factored resistance for direct shear and tension- or compression-induced shear shall be taken as

$V_r = 0.67\phi_w A_w F_u$

where
A_w = $0.50 w_f L$ (or as established by procedure qualification tests)

where
w_f = width of flare bevel groove weld face

F_u = least ultimate tensile strength of the components in the joint

13.13.3 Tension normal to axis of weld

13.13.3.1 Complete joint penetration groove weld made with matching electrodes

The factored tensile resistance shall be taken as that of the base metal.

13.13.3.2 Partial joint penetration groove weld made with matching electrodes

The factored tensile resistance shall be taken as

$T_r = \phi_w A_n F_u \le \phi A_g F_y$

where
A_n = nominal area of fusion face normal to the tensile force

When overall ductile behaviour is desired (member yielding before weld fracture), the following shall apply:

$A_n F_u > A_g F_y$

13.13.3.3 Partial joint penetration groove weld combined with a fillet weld, made with matching electrodes

The factored tensile resistance shall be taken as

$T_r = \phi_w \sqrt{(A_n F_u)^2 + (A_w X_u)^2} \le \phi A_g F_y$

where
A_g = gross area of the components of the tension member connected by the welds

13.13.4 Compression normal to axis of weld

13.13.4.1 Complete and partial joint penetration groove welds made with matching electrodes

The compressive resistance shall be taken as that of the effective area of base metal in the joint. For partial joint penetration groove welds, the effective area in compression shall be taken as the nominal area of the fusion face normal to the compression plus the area of the base metal fitted in contact bearing (see Clause 28.5).

13.13.4.2 Cross-sectional properties of continuous longitudinal welds

Continuous longitudinal welds made with matching electrodes may be considered as contributing to the cross-sectional properties A, S, Z, and I of the cross-section.

13.13.4.3 Welds for hollow structural sections

The provisions of Annex L of CSA W59 may be used for hollow structural sections.

13.14 Welds and high-strength bolts in combination

The factored shear resistance of a joint that combines welds and bolts in the same plane, $V_{r,joint}$, shall be taken as the largest of

a) $V_{friction} + V_{r,trans} + 0.85V_{r,long}$;

b) $V_{friction} + V_{r,long} + 0.5V_{r,bolt}$; and

c) $V_{r,bolt}$.

where

$V_{friction}$ = plate friction resistance component

= $0.25V_s$ when the bolts are pretensioned in accordance with Clause 23.7

= 0 when the bolts are not pretensioned

$V_{r,trans}$ = transverse weld resistance component

= V_r determined from Clause 13.13.2.2 for $\theta = 90°$

$V_{r,long}$ = longitudinal weld resistance component

= V_r determined from Clause 13.13.2.2 for $\theta = 0°$

$V_{r,bolt}$ = bolt shear resistance component

= V_r determined from Clause 13.12.1.2

14 Beams and girders

14.1 Proportioning

Beams and girders consisting of rolled shapes (with or without cover plates), hollow structural sections, or fabricated sections shall be proportioned on the basis of the properties of the gross section or the modified gross section. No deduction need be made for fastener holes in webs or flanges unless the reduction of flange area by such holes exceeds 15% of the gross flange area, in which case the excess shall be deducted. The effect of openings other than holes for fasteners shall be considered in accordance with Clause 14.3.3.

14.2 Flanges

14.2.1

Flanges of welded girders should consist of a single plate or a series of plates joined end-to-end by complete penetration groove welds.

14.2.2

Flanges of bolted girders shall be proportioned so that the total cross-sectional area of cover plates does not exceed 70% of the total flange area.

14.2.3

Fasteners or welds connecting flanges to webs shall be proportioned to resist horizontal shear forces due to bending combined with any loads that are transmitted from the flange to the web other than by direct bearing. Spacing of fasteners or intermittent welds in general shall be in proportion to the intensity of the shear force and shall not exceed the maximum for compression or tension members, as applicable, in accordance with Clause 19.

14.2.4

Partial-length flange cover plates shall be extended beyond the theoretical cut-off point and the extended portion shall be connected with sufficient fasteners or welds to develop a force in the cover plate at the theoretical cut-off point not less than

$$P = \frac{AM_{fc}y}{I_g}$$

where

P = required force to be developed in cover plate

A = area of cover plate

M_{fc} = moment due to factored loads at theoretical cut-off point

y = distance from centroid of cover plate to neutral axis of cover-plated section

I_g = moment of inertia of cover-plated section

Additionally, for welded cover plates, the longitudinal welds connecting the cover-plate termination to the beam or girder shall be designed to develop the force, P, within a length, a', measured from the actual end of the cover plate, determined as follows:

a) when there is a continuous weld equal to or larger than three-fourths of the cover-plate thickness across the end of the plate and along both edges of the cover plate, a' shall be taken as the width of the cover plate;

b) when there is a continuous weld smaller than three-fourths of the cover-plate thickness across the end of the plate and along both edges, a' shall be taken as 1.5 times the width of the cover plate; and

c) when there is no weld across the end of the plate but there are continuous welds along both edges, a' shall be taken as 2 times the width of the cover plate.

14.3 Webs

14.3.1 Maximum slenderness

The slenderness ratio, h/w, of a web shall not exceed $83\,000/F_y$

where
F_y = specified minimum yield point of the compression flange steel

This limit may be waived if analysis indicates that buckling of the compression flange into the web will not occur at factored load levels.

14.3.2 Web crippling and yielding

The factored bearing resistance of the web shall be taken as follows:

a) for interior loads (concentrated load applied at a distance from the member end greater than the member depth), the smaller of
   i) $B_r = \phi_{bi} w \left(N + 10t\right) F_y$
   ii) $B_r = 1.45 \phi_{bi} w^2 \sqrt{F_y E}$
b) for end reactions, the smaller of
   i) $B_r = \phi_{be} w (N + 4t) F_y$
   ii) $B_r = 0.60 \phi_{be} w^2 \sqrt{F_y E}$

where
ϕ_{bi} = 0.80
ϕ_{be} = 0.75
N = length of bearing

Where the bearing resistance of the web is exceeded, bearing stiffeners shall be used (see Clause 14.4).

14.3.3 Openings

14.3.3.1

Except as specified in Clause 14.1, the effect of all openings in beams and girders shall be considered in the design. At all points where the factored shear or moments at the net section would exceed the capacity of the member, adequate reinforcement shall be added to the member at these points to provide the required strength and stability.

14.3.3.2

Unreinforced circular openings may be located in the web of unstiffened prismatic Class 1 and Class 2 beams or girders without considering net section properties, provided that

a) the load is uniformly distributed;
b) the section has an axis of symmetry in the plane of bending;
c) the openings are located within the middle third of the depth and the middle half of the span of the member;
d) the spacing between the centres of any two adjacent openings, measured parallel to the longitudinal axis of the member, is a minimum of 2.5 times the diameter of the larger opening; and
e) the factored maximum shear at the support does not exceed 50% of the factored shear resistance of the section.

14.3.3.3

If the forces at openings are determined by an elastic analysis, the procedure shall be in accordance with published, recognized principles.

14.3.3.4

The strength and stability of the member in the vicinity of openings may be determined on the basis of assumed locations of plastic hinges, such that the resulting force distributions satisfy the requirements of equilibrium, provided that the analysis is carried out in accordance with Items a), b), and f) of Clause 8.3.2.However, for I-type members, the width-to-thickness ratio of the flanges may meet only the requirements of Class 1 or 2 sections, provided that the webs meet the width-to-thickness limit of Class 1 sections.

14.3.4 Effect of thin webs on moment resistance

When the web slenderness ratio, h/w, exceeds $1900/\sqrt{M_f/\phi S}$, the flange shall meet the width-to-thickness ratios of Class 3 sections in accordance with Clause 11, and the factored moment resistance of the beam or girder, M_r', shall be determined as follows:

$$M_r' = M_r\left[1 - 0.0005\frac{A_w}{A_f}\left(\frac{h}{w} - \frac{1900}{\sqrt{M_f/\phi S}}\right)\right]$$

where
M_r = factored moment resistance determined in accordance with Clause 13.5 or 13.6, but not to exceed ϕM_y

When an axial compressive force acts on the girder in addition to the moment, the constant 1900 in the expression for M_r' shall be reduced by the factor $(1 - 0.65C_f/\phi C_y)$ (see also Clause 11.2).

14.4 Bearing stiffeners

14.4.1

Pairs of bearing stiffeners on the webs of single-web beams and girders shall be required at points of concentrated loads and reactions wherever the bearing resistance of the web is exceeded (see Clause 14.3.2).Bearing stiffeners shall also be required at unframed ends of single-web girders having web depth-to-thickness ratios greater than $1100\sqrt{F_y}$. Box girders may employ diaphragms designed to act as bearing stiffeners.

14.4.2

Bearing stiffeners shall bear against the flange or flanges through which they receive their loads and shall extend approximately to the edge of the flange plates or flange angles. They shall be designed as columns in accordance with Clause 13.3, assuming that the column section consists of the pair of stiffeners and a centrally located strip of the web equal to not more than 25 times its thickness at the interior stiffeners or a strip equal to not more than 12 times its thickness when the stiffeners are located at the end of the web. The effective column length, KL, shall be taken as not less than three-fourths of the length of the stiffeners in calculating the ratio KL/r. Only that portion of the stiffeners outside of the angle fillet or the flange-to-web welds shall be considered effective in bearing. Angle bearing stiffeners shall not be crimped. Bearing stiffeners shall be connected to the web to develop the full force required to be carried by the stiffener into the web or vice versa. The stiffeners shall conform to Clause 11.2 (see Table 1) and have a width to thickness ratio that satisfies $\frac{b}{t} \le \frac{200}{\sqrt{F_y}}$.

14.5 Intermediate transverse stiffeners

14.5.1

Intermediate transverse stiffeners, when used, shall be spaced to suit the shear resistance determined in accordance with Clause 13.4, except that at girder end panels or at panels adjacent to large openings, the tension-field component shall be taken as zero unless means are provided to anchor the tension field.

14.5.2

Except as specified in Clause 14.5.1, the maximum distance between stiffeners, when required, shall not exceed the values shown in Table 5.

14.5.3

Intermediate transverse stiffeners may be furnished singly or in pairs. Width-to-thickness ratios shall meet the requirements of Clause 11. The moment of inertia of the stiffener, or pair of stiffeners if so furnished, shall be not less than $(h/50)^4$ taken about an axis in the plane of the web. The gross area, A_s, of intermediate stiffeners, or pairs of stiffeners if so furnished, shall be as follows:

$$A_s = \frac{aw}{2}\left[1 - \frac{a/h}{\sqrt{1+(a/h)^2}}\right]CYD$$

where

a = centre-to-centre distance of adjacent stiffeners (i.e., panel length)

w = web thickness

h = web depth

C = $\left[\frac{1 - 310\,000\,k_v}{F_y(h/w)^2}\right]$ but not less than 0.10

where

k_v = shear buckling coefficient (see Clause 13.4.1.1)

F_y = specified minimum yield point of web steel

Y = ratio of specified minimum yield point of web steel to specified minimum yield point of stiffener steel

D = stiffener factor

= 1.0 for stiffeners furnished in pairs

= 1.8 for single-angle stiffeners

= 2.4 for single-plate stiffeners

When the greatest shear, V_f, in an adjacent panel is less than that permitted by Clause 13.4.1.1, this gross area requirement may be reduced by multiplying by the ratio V_f/V_r.

14.5.4

Intermediate transverse stiffeners shall be connected to the web for a shear transfer per pair of stiffeners (or per single stiffener when so furnished), in newtons per millimetre of web depth, h, not less than

$1 \times 10^{-4} h F_y^{1.5}$, except that when the largest calculated shear, V_f, in the adjacent panels is less than V_r, this shear transfer may be reduced in the same proportion. However, the total shear transfer shall not be less than the value of any concentrated load or reaction required to be transmitted to the web through the stiffener. Fasteners connecting intermediate transverse stiffeners to the web shall be spaced not more than 300 mm from centre-to-centre. If intermittent fillet welds are used, the clear distance between welds shall not exceed 16 times the web thickness or four times the weld length.

14.5.5

When intermediate stiffeners are used on only one side of the web, the stiffeners shall be attached to the compression flange. Intermediate stiffeners used in pairs shall have at least a snug fit against the compression flange. When stiffeners are cut short of the tension flange, the distance cut short shall be equal to or greater than four times but not greater than six times the girder web thickness. Stiffeners should be clipped to clear girder flange-to-web welds.

14.6 Combined shear and moment

Transversely stiffened girders depending on tension-field action to carry shear shall be proportioned such that

a) $0.727\dfrac{M_f}{M_r} + 0.455\dfrac{V_f}{V_r} \le 1.0$

b) $\dfrac{M_f}{M_r} \le 1.0$

c) $\dfrac{V_f}{V_r} \le 1.0$

where

M_r = value determined in accordance with Clause 13.5 or 13.6, as applicable

V_r = value determined in accordance with Clause 13.4

14.7 Rotational restraint at points of support

Beams and girders shall be restrained against rotation about their longitudinal axes at points of support.

14.8 Copes

14.8.1

The effect of copes on flexural yielding, local web buckling, and lateral torsional buckling resistance of a beam or girder shall be taken into account.

14.8.2

The effect of copes in reducing the net area of the web available to resist transverse shear and the effective net area of potential paths of minimum resistance shall be taken into account (see Clause 13.11).

14.9 Lateral forces

The flanges of beams and girders supporting cranes or other moving loads shall be proportioned to resist any lateral forces produced by such loads.

14.10 Torsion

14.10.1

Beams and girders subjected to torsion shall have sufficient strength and rigidity to resist the torsional moment and forces in addition to other moments or forces. The connections and bracing of such members shall be adequate to transfer the reactions to the supports.

14.10.2

The factored resistance of I-shaped members subject to combined flexure and torsion may be determined from moment-torque interaction diagrams that take into account the normal stress distribution due to flexure and warping torsion and the St. Venant torsion. Assumed normal stress distributions shall be consistent with the class of section.

14.10.3

Members subject to torsional deformations required to maintain compatibility of the structure need not be designed to resist the associated torsional moments, provided that the structure satisfies the requirements of equilibrium.

14.10.4

For all members subject to loads causing torsion, the torsional deformations under specified loads shall be limited in accordance with Clause 6.3.1.1. For members subject to torsion or to combined flexure and torsion, the maximum combined normal stress, as determined by an elastic analysis, arising from warping torsion and bending due to the specified loads shall not exceed F_y.

15 Trusses

15.1 Analysis

15.1.1 Simplified method

The simplified method assumes that all members are pin-connected and loads are only applied at the panel points, except that bending effects due to transverse loads applied between panel points are assessed by taking into account any continuity of the members. This method may be used when compression members are at least Class 3.

15.1.2 Detailed method

The detailed method accounts for the actual loading and joint fixity. The detailed method shall be used for trusses

a) with panels adjacent to abrupt changes in the slope of a chord;
b) with Vierendeel panels;
c) with panels at abrupt changes in transverse shear; or
d) designed for fatigue.

15.2 General requirements

15.2.1 Effective lengths of compression members

The effective length for buckling in the plane of the truss shall be taken as the distance between the lines of intersection of the working points of the web members and the chord. The effective length for buckling perpendicular to the plane of the truss shall be equal to the distance between the points of lateral support. For built-up members, see Clause 19.

Note: *For the effective lengths of compression members in trusses comprising hollow structural sections, see CISC's* Hollow Structural Section: Connections and Trusses — A Design Guide.

15.2.2 Joint eccentricities

Bending moments due to joint eccentricities shall be taken into account. The eccentricity of work points at a joint or at a support shall be taken into account.

15.2.3 Stability

Trusses shall be braced to ensure their lateral stability. Brace members that support compression chords at discrete points shall meet the requirements of Clause 9.2. Ends of compression chords that are not attached to a supporting member shall be braced laterally, unless it can be demonstrated that the support is not necessary.

15.2.4 Chord members

Splices may occur at any point in chord members.

15.2.5 Web members

The factored resistances of the first compression web member subject to transverse shear, and its connections, shall be determined with their respective resistance factors, ϕ, multiplied by 0.85.

The bending moments due to truss geometric distortions of end compression web members of bottom bearing trusses shall be included in the design. The simplified method may be used.

Splices may occur at any point in web members.

15.2.6 Compression chord supports

Truss web members that provide support to a compression chord in the plane of the truss shall be designed for an additional force equal to 0.02 of the chord force, unless the brace force has been determined by rigorous analysis.

15.2.7 Maximum slenderness ratio of tension chords

The maximum slenderness ratio shall be limited to 240, except when other means are provided to control flexibility, sag, vibration, and slack in a manner commensurate with the service conditions of the structure.

15.2.8 Deflection and camber

Except for the deflection due to flexural deformation of Vierendeel panels, deflections may be determined from the axial deformations of the truss members. For camber, see Clause 6.3.2.

15.3 Composite trusses

Trusses designed to act compositely with the slab or cover slab shall also meet the requirements of Clause 17.

16 Open-web steel joists

16.1 Scope

Clause 16 specifies requirements for the design, manufacture, transportation, and erection of open-web steel joists used in the construction of buildings. Joists intended to act compositely with the deck slab shall also meet the requirements of Clause 17. Clause 16 shall be used only for the design of joists having an axis of symmetry in the plane of the joist.

16.2 General

Open-web steel joists are steel trusses of relatively low mass with parallel or slightly pitched chords and triangulated web systems proportioned to span between walls or structural supporting members, or both, and to provide direct support for floor or roof deck. In general, joists are manufactured on a production line that employs jigs, with certain details of the members being standardized by the individual manufacturer. When specified, joists can be designed to provide lateral support to compression elements of beams or columns, to participate in lateral-load-resisting systems, or as continuous joists, cantilevered joists, or joists having special support conditions.

16.3 Materials

Steel used for joists shall be a weldable structural grade meeting the requirements of Clause 5.1. Structural members cold-formed to shape may use the effect of cold-forming in accordance with Clause A7 of CSA S136. The calculated value of F_y' shall be determined using only the values for F_y and F_u that are specified in the relevant structural steel material standard. Yield levels reported on mill test certificates or determined in accordance with Clause F3 of CSA S136 shall not be used as the basis for design.

16.4 Design documents

16.4.1 Building structural design documents

The building structural design documents shall include, as a minimum, the following:

a) all the loads carried by the joists, such as the uniformly distributed specified live and total dead loads, unbalanced loading conditions, any concentrated loads, and any special loading conditions, e.g., non-uniform snow loads, ponding loads, horizontal loads, end moments, net uplift, downward wind load, bracing forces to provide lateral support to compression elements of beams or columns, and allowances for mechanical equipment;
b) joist spacing, deflection limits and camber (see Clause 6.3.2), joist depth, and shoe depth;
c) where joists are not supported on steel members, maximum bearing pressures or sizes of bearing plates;
d) anchorage requirements in excess of the requirements of Clause 16.5.12;
e) bracing required by Clause 16.5.6.2 (if any);
f) method for and spacing of attachments of steel deck to the top chord (the documents shall indicate the special cases where the deck is incapable of supplying lateral support to the top chord [see Clause 16.8.1]);

g) minimum moment of inertia to provide satisfactory design criteria for floor vibrations, if applicable (see Clause 6.3.3.2);
h) any other information necessary for designing and supplying the joists; and
i) a note that no drilling, cutting, or welding is to be done unless approved by the building designer.

Note: *The building drawings should include a note warning that attachments for mechanical, electrical, and other services should be made using approved clamping devices or U-bolt-type connectors.*

16.4.2 Joist design documents

Joist design documents prepared by the joist manufacturer shall show, as a minimum, the
a) specified loading;
b) factored member loads;
c) material specification;
d) member sizes;
e) dimensions;
f) spacers;
g) welds;
h) shoes;
i) anchorages;
j) bracing;
k) bearings;
l) field splices;
m) bridging locations;
n) camber; and
o) coating type.

16.5 Design

16.5.1 Loading for open-web steel joists

The factored moment and shear resistances of open-web steel joists at every section shall be not less than the moment and shear due to the loading conditions specified by the building designer in the documents described in Clause 16.4.1 a) or to the factored dead load plus the following factored live load conditions, considered separately:
a) for floor joists, an unbalanced live load applied on any continuous portion of the joist to produce the most critical effect on any component;
b) for roof joists, an unbalanced loading condition with 100% of the snow load plus other live loads applied on any continuous portion of the joist and 50% of the snow load on the remainder of the joist to produce the most critical effect on any component;
c) for roof joists, wind uplift;
d) for roof joists, 100% of the snow load plus 40% of the downward wind load (companion load) (1.5S + 0.4W); and
e) the appropriate factored concentrated load (from the *NBCC*) applied at any one panel point to produce the most critical effect on any component.

16.5.2 Design assumptions

Open-web steel joists shall be designed for loads acting in the plane of the joist applied to the top chord assumed to be prevented from lateral buckling by the deck. For the purpose of determining axial forces in all members, members may be assumed to be pin-connected and the loads may be replaced by statically equivalent loads applied at the panel points.

The resistance of the deck connections as well as the resistance of the deck shall be verified by the joist designer to ensure that adequate lateral support is provided to the top chord of a joist as determined in accordance with Clause 9.2.7. When additional stability elements are necessary, they shall be designed in accordance with Clause 9.2.6.2.

16.5.3 Member and connection resistance

Member and connection resistance shall be calculated in accordance with Clause 13, except as otherwise specified in Clause 16.

16.5.4 Width-to-thickness ratios

Note: *Clause 16.5.4 is applicable for members made of more than one shape.*

16.5.4.1

The width-to-thickness ratios of compressive elements of hot-formed sections and cold-formed HSS shall be governed by Clause 11. The width-to-thickness ratios of compressive elements of cold-formed sections shall be governed by CSA S136.

16.5.4.2

For the purpose of determining the appropriate width-to-thickness ratio of compressive elements supported along one edge, any stiffening effect of the deck or the joist web shall be neglected.

16.5.5 Bottom chord

16.5.5.1

The bottom chord shall be continuous and, when in tension, may be designed as an axially loaded tension member unless subject to eccentricities in excess of those permitted under Clause 16.5.10.4 or subject to applied load between panel points. The governing radius of gyration of the tension chord or any component thereof shall be not less than 1/240 of the corresponding unsupported length. For joists with the web in the y-plane, the unsupported length of chord for computing L_x/r_x shall be taken as the panel length centre-to-centre of panel points and the unsupported length of chord for calculating L_y/r_y shall be taken as the distance between bridging lines connected to the tension chord. Joist shoes, when anchored, may be assumed to be equivalent to bridging lines. A bottom chord subjected to concentrated loads between panel points shall be designed, when the chord is in tension, in accordance with Clause 13.9 and, when the chord is in compression, in accordance with Clause 16.5.7.3, respectively.

16.5.5.2

The bottom chord shall be designed in accordance with Clause 16.5.7.3 for the resulting compressive forces when

a) net uplift is specified;
b) joists are made continuous or cantilevered;
c) end moments are specified; or
d) it provides lateral support to compression elements of beams or columns.

Bracing, when required, shall be provided in accordance with Clause 9.2. For joists with net uplift, a single line of bottom-chord bridging shall be provided at each end of the joists near the first bottom chord panel points unless the ends of the bottom chord are otherwise restrained. [See also Clause 16.7.9 a)].

16.5.6 Top chord

16.5.6.1

The top chord shall be continuous and may be designed for axial compressive force alone when

a) the panel length does not exceed 610 mm;
b) concentrated loads are not applied between the panel points; and
c) not subject to eccentricities in excess of those permitted under Clause 16.5.10.4.

When the panel length exceeds 610 mm, the top chord shall be designed as a continuous member subject to combined axial and bending forces.

16.5.6.2

The slenderness ratio, KL/r, of the top chord or of its components shall not exceed 90 for interior panels or 120 for end panels. The governing KL/r shall be the maximum value determined by the following:

a) for the x-x (horizontal) axis, L_x shall be the centre-to-centre distance between panel points and K shall be taken as 0.9;
b) for the y-y (vertical) axis, L_y shall be the centre-to-centre distance between the attachments of the deck. The spacing of attachments shall be not more than the design slenderness ratio of the top chord times the radius of gyration of the top chord about its vertical axis and not more than 1000 mm.K shall be taken as 1.0; and
c) for the z-z (skew) axis of individual components, L_z shall be the centre-to-centre distance between panel points or spacers, or both, and K shall be taken as 0.9. Decking shall not be considered to fulfill the function of batten plates or spacers for top chords consisting of two separated components, where r = the appropriate radius of gyration.

16.5.6.3

Compression chords shall be proportioned such that

$$\frac{C_f}{C_r} + \frac{M_f}{M_r} \le 1.0$$

where

M_r = value specified in Clause 13.5

C_r = value specified in Clause 13.3

At the panel point, C_r may be taken as $\phi A F_y$ and Clause 13.5 a) may be used to determine M_r, provided that the chord meets the requirements of a Class 2 section and $M_f/M_p < 0.25$.

For top chords with panel lengths not exceeding 610 mm, M_f resulting from any uniformly distributed loading may be neglected.

The chord shall be assumed to be pinned at the joist supports.

16.5.6.4

Top chords in tension whose panel lengths exceed 610 mm shall be designed in accordance with Clause 13.9.

16.5.6.5
When welding is used to attach steel deck to the chord of a joist, the flat width of any chord component in contact with the deck shall be at least 5 mm larger than the nominal design dimensions of the deck welds, measured transverse to the longitudinal axis of the chord.

16.5.6.6
When mechanical fasteners are used to attach steel deck to the chord of a joist, the minimum chord thickness shall be specified by the designer.

16.5.7 Webs

16.5.7.1
Webs shall be designed in accordance with Clause 13 to resist the shear at any point due to the factored loads specified in Clause 16.5.1. Particular attention shall be paid to possible reversals of force in each web member.

16.5.7.2
The length of a web member shall be taken as the distance between the intersections of the neutral axes of the web member and the chords. For buckling in the plane of the web, the effective length factor shall be taken as 0.9 if the web consists of individual members. For all other cases, the effective length factor shall be taken as 1.0.

16.5.7.3
The factored resistances of the first compression web member subject to transverse shear, and its connections, shall be determined with their respective resistance factors, ϕ, multiplied by 0.85.

16.5.7.4
The vertical web members of a joist with a modified Warren geometry shall be designed to resist an axial force equal to the calculated sum of the compressive force in the web member plus 0.02 times the force in the compression chord at that location.

16.5.7.5
The slenderness ratio of a web member in tension need not be limited.

16.5.7.6
The slenderness ratio of a web member in compression shall not exceed 200.

16.5.8 Spacers and battens
Compression members consisting of two or more sections shall be interconnected so that the slenderness ratio of each section calculated using its least radius of gyration is less than or equal to the design slenderness ratio of the built-up member. Spacers or battens shall be an integral part of the joist.

16.5.9 Connections and splices

16.5.9.1
Component members of joists shall be connected by welding, bolting, or other approved means.

16.5.9.2
Connections and splices shall develop the factored loads without exceeding the factored member resistances specified in Clause 16. Butt-joint splices shall develop the factored tensile resistance, T_r, of the member.

16.5.9.3
Splices may occur at any point in chord or web members.

16.5.9.4
Members connected at a joint should have their centroidal axes meet at a point. Where this is impractical and eccentricities are introduced, such eccentricities may be neglected if they do not exceed the following:
a) for continuous web members, the greater of the two distances measured from the neutral axis of the chord member to the extreme fibres of the chord member; and
b) for non-continuous web members, the distance measured from the neutral axis to the back (outside face) of the chord member.

When the eccentricity exceeds these limits, provision shall be made for the effects of the total eccentricity. Eccentricities assumed in design shall be taken as the maximum fabrication tolerances and shall be included with the shop details.

16.5.10 Bearings

16.5.10.1
Bearings of joists shall be proportioned so that the factored bearing resistance of the supporting material is not exceeded.

16.5.10.2
Where a joist bears, with or without a bearing plate, on solid masonry or concrete support, the bearing shall meet the requirements of CSA S304.1 for masonry and CSA A23.3 for concrete.

16.5.10.3
Where a joist bears on a structural steel member, the end of the shoe shall extend at least 65 mm beyond the edge of the support, except that when the available bearing area is restricted, this distance may be reduced, provided that the shoe is adequately proportioned and anchored to the support.

16.5.10.4
The joist shoe and the end panel of the joist shall be proportioned to include the effect of the eccentricity between the centre of the bearing and the intersection of the centroidal axes of the chord and the end diagonal.

16.5.10.5
Bottom bearing joists shall have their top and bottom chords held adequately in position at the supports.

16.5.11 Anchorage

16.5.11.1

Joists shall be properly anchored to withstand the effects of the combined factored loads, including net uplift. As a minimum, the following shall be provided:

a) when anchored to masonry or concrete:
   i) for floor joists, a 10 mm diameter rod at least 300 mm long embedded horizontally; and
   ii) for roof joists, a 20 mm diameter anchor rod 300 mm long embedded vertically with a 50 mm, 90° hook or a 20 mm diameter headed anchor rod; and

b) when supported on steel, one 20 mm diameter bolt, or a pair of fillet welds satisfying the minimum size and length requirements of CSA W59; the connection shall be capable of withstanding a horizontal load equal to 10% of the reaction of the joist.

16.5.11.2

Tie joists may have their top and bottom chords connected to a column. Unless otherwise specified by the building designer, tie joists shall have top and bottom chord connections that are each at least equivalent to those required by Clause 16.5.12.1. Either the top or bottom connection shall utilize a bolted connection.

16.5.11.3

Where joists are used as a part of a frame, the joist-to-column connections shall be designed to carry the moments and forces due to the factored loads.

16.5.12 Deflection

16.5.12.1

Steel joists shall be proportioned so that deflection due to specified loads is within acceptable limits for the nature of the materials to be supported and the intended use and occupancy. Such deflection limits shall be as specified in Clause 6.3.1 unless otherwise specified by the building designer.

16.5.12.2

The deflection shall be calculated based on truss action, taking into account the axial deformation of all of the components of the joists.

16.5.13 Camber

Unless otherwise specified by the building designer, the nominal camber shall be 0.002 of the span. Negative cambers to satisfy roof drainage requirements shall be designed for appropriate rainwater ponding loads.

Note: *For manufacturing tolerances, see Clause 16.10.9. For maximum deviation between adjacent joists, or joists and adjacent beams or walls, see Clause 16.12.2.5. For special camber requirements, see Clause 6.3.2.2.*

16.5.14 Vibration

The building designer shall give special consideration to floor systems where unacceptable vibration can occur. When requested, the joist manufacturer shall supply joist properties and details to the building designer (see Annex E).

16.5.15 Welding

Welding shall meet the requirements of Clause 24. Specific welding procedures for joist fabrication shall be developed and meet the requirements of CSA W47.1.

16.6 Stability during construction

Means shall be provided to support joist chords against lateral movement and to hold the joist in the vertical or specified plane during construction.

16.7 Bridging

16.7.1 General

Bridging transverse to the span of joists may be used to meet the requirements of Clause 16.6 and also to meet the slenderness ratio requirements for chords. Bridging shall not be considered “bracing” as described in Clause 9.2.

16.7.2 Installation

All bridging and bridging anchors shall be completely installed before any construction loads, except for the weight of the workers necessary to install the bridging, are placed on the joists.

16.7.3 Types

Unless otherwise specified or approved by the building designer, the joist manufacturer shall supply bridging that may be of the diagonal or horizontal type.

16.7.4 Diagonal bridging

Diagonal bridging consisting of crossed members running from the top chord to the bottom chord of adjacent joists shall have a slenderness ratio, L/r, of not more than 200, where L is the length of the diagonal bridging member or one-half of this length when crossed members are connected at their point of intersection and r is the least radius of gyration. All diagonal bridging shall be connected adequately to the joists by bolts or welds.

16.7.5 Horizontal bridging

A line of horizontal bridging shall consist of a continuous member perpendicular to the joist span attached to either the top chord or the bottom chord of each joist. Horizontal bridging members shall have a slenderness ratio of not more than 300.

16.7.6 Attachment of bridging

Attachment of diagonal and horizontal bridging to joist chords shall be by welding or mechanical means capable of resisting an axial load of at least 3 kN in the attached bridging member. Welds shall meet the minimum length requirements specified in CSA W59.

16.7.7 Anchorage of bridging

Each line of bridging shall be adequately anchored at each end to sturdy walls or to main components of the structural frame, if practicable. Otherwise, diagonal and horizontal bridging shall be provided in combination between adjacent joists near the ends of bridging lines.

16.7.8 Bridging systems

Bridging systems, including sizes of bridging members and all necessary details, shall be shown on the erection diagrams. If a specific bridging system is required by the design, the design drawings shall show all information necessary for the preparation of shop details and erection diagrams.

16.7.9 Spacing of bridging

Diagonal and horizontal bridging shall be spaced so that the unsupported length of the chord between bridging lines or between laterally-supported ends of the joist and adjacent bridging lines does not exceed

a) $170r$ for chords in compression; and
b) $240r$ for chords always in tension

where
r = applicable chord radius of gyration about its axis in the plane of the web.

Ends of joists anchored to supports may be assumed to be equivalent to bridging lines. If ends of joists are not so anchored before the deck is installed, the distance from the face of the support to the nearest bridging member in the plane of the bottom chord shall not exceed $120r$. There shall not be less than one line of horizontal or diagonal bridging attached to each joist spanning 4 m or more. If only a single line of bridging is required, it shall be placed at the centre of the joist span. If bridging is not used on joists less than 4 m in span, the ends of such joists shall be anchored to the supports to prevent overturning of the joist during placement of the deck.

16.8 Decking

16.8.1 Decking to provide lateral support

Decking shall bear directly on the top chord of the joist. If not sufficiently rigid to provide lateral support to the compression chord of the joist, the compression chord of the joist shall be braced laterally in accordance with Clause 9.2.

16.8.2 Deck attachments

Attachments considered to provide lateral support to top chords shall meet the requirements of Clause 9.2.3. The spacing of attachments shall not exceed

a) the design slenderness ratio of the top chord times the radius of gyration of the top chord about its vertical axis; and
b) 1 m.

16.8.3 Diaphragm action

Where decking is used in combination with joists to form a diaphragm for the purpose of transferring lateral applied loads to vertical bracing systems, special attachment requirements shall be fully specified on the building design drawings.

16.8.4 Cast-in-place slabs

Cast-in-place slabs used as decking shall have a minimum thickness of 65 mm. Forms for cast-in-place slabs shall not cause lateral displacement of the top chords of joists during installation of the forms or the placing of the concrete. Non-removable forms shall be positively attached to top chords by means of welding, clips, ties, wedges, fasteners, or other suitable means at intervals not exceeding 1 m; however, there shall be at least two attachments in the width of each form at each joist. Forms and their method of attachment shall be such that the cast-in-place slab, after hardening, is capable of furnishing lateral support to the joist chords.

16.8.5 Installation of steel deck

16.8.5.1

To facilitate attachment of the steel deck, the location of the top chord of the joist shall be confirmed by marking the deck at suitable intervals or by other means.

16.8.5.2

The installer of the steel deck to be fastened to joists by arc spot welding shall be a company that is certified by the Canadian Welding Bureau to the requirements of CSA W47.1.

The welding procedures shall meet the requirements of CSA W47.1.

The welders shall meet the requirements of CSA W47.1 for arc spot welding.

16.9 Shop coating

Joists shall have a shop coating meeting the requirements of Clause 28.7.3.3, unless otherwise specified by the building designer.

16.10 Manufacturing tolerances

16.10.1

The tolerance on the specified depth of the manufactured joist shall be ± 7 mm.

16.10.2

The deviation of a panel point from the design location, measured along the length of a chord, shall not exceed 13 mm. The centroidal axes of the bottom chord and the end diagonals carrying transverse shear should meet at the first bottom panel point even when the end diagonal is an upturned bottom chord (see Clause 16.5.10.4).

16.10.3

The deviation of a panel point from the design location, measured perpendicular to the longitudinal axis of the chord and in the plane of the joist, shall not exceed 7 mm.

16.10.4

The connections of web members to chords shall not deviate laterally more than 3 mm from that assumed in the design.

16.10.5

The sweep of a joist or any portion of the length of the joist, upon completion of manufacture, shall not exceed 1/500 of the length on which the sweep is measured.

16.10.6

The tilt of bearing shoes shall not exceed 1 in 50 measured from a plane perpendicular to the plane of the web and parallel to the longitudinal axis of the joist.

16.10.7
The tolerance on the specified shoe depth shall be ± 3 mm.

16.10.8
The tolerance on the specified length of the joist shall be ± 7 mm. The connection holes in a joist shall not vary from the detailed location by more than 2 mm for joists 10 m or less in length or by more than 3 mm for joists more than 10 m in length.

16.10.9
The tolerance in millimetres on the nominal or specified camber shall be $\pm\left(6+\frac{L}{4000}\right)$.

The minimum camber in a joist shall be 4 mm. The range in camber for joists of the same span shall be 20 mm.

16.11 Inspection and quality control

16.11.1 Inspection
Material and quality of work shall be accessible for inspection at all times by qualified inspectors representing the building designer. Random in-process inspection shall be carried out by the manufacturer and all joists shall be thoroughly inspected by the manufacturer before shipping. Third-party welding inspection shall be in accordance with Clause 30.5.

16.11.2 Identification and control of steel
Steel used in the manufacture of joists shall be identified in the manufacturer's plant as to its specification (and grade, where applicable) by suitable markings, recognized colour-coding, or a system devised by the manufacturer that will ensure to the satisfaction of the building designer that the correct material is being used.

16.11.3 Quality control
Upon request by the building designer, the manufacturer shall provide evidence of having suitable quality control measures to ensure that the joists meet all specified requirements. When testing is part of the manufacturer's normal quality control program, the loading criteria shall be 1.0/0.9 times the factored loads for the specific joist design.

16.12 Handling and erection

16.12.1 General
Care shall be exercised to avoid damage during strapping, transport, unloading, site storage, stacking, and erection. Dropping of joists shall be avoided. Special precautions shall be taken when erecting long, slender joists, and hoisting cables should not be released until the member is stayed laterally by at least one line of bridging. Joists shall have all bridging attached and permanently fastened in place before the application of any loads. Construction loads shall be adequately distributed so as not to exceed the capacity of any joist. Field welding shall not cause damage to joists, bridging, deck, and supporting steel members.

16.12.2 Erection tolerances

16.12.2.1
The maximum sweep of a joist or a portion of the length of a joist upon completion of erection shall not exceed the limit specified in Clause 16.10.5 and shall be in accordance with the requirements of Clause 29.

16.12.2.2
All members shall be free from twists, sharp kinks, and bends.

16.12.2.3
The deviation of joists as erected from the location in the plan shown on the erection diagrams shall not exceed 15 mm.

16.12.2.4
The deviation of the bottom chord with respect to the top chord, normal to the specified plane of the web of a joist, shall not exceed 1/50 of the depth of the joist.

16.12.2.5
The maximum deviation in elevation between the tops of any three adjacent joists shall not be greater than 0.01 times the joist spacing and not greater than 25 mm. The deviation is the vertical offset from the top of the centre joist to the line joining the tops of the centres of the adjacent joists. The maximum shall also apply to joists adjacent to beams or walls.

17 Composite beams, trusses, and joists

17.1 Application
Clause 17 shall apply to composite beams consisting of steel sections, trusses, or joists interconnected with either a reinforced concrete slab or a steel deck with a concrete cover slab. Trusses and joists designed to act compositely with the slab or cover slab shall also meet the requirements of Clauses 15 and 16, respectively. The minimum slab or cover slab thickness shall be 65 mm unless the adequacy of a lesser thickness has been established by appropriate tests.

17.2 Definitions
The following definitions apply in Clause 17:

Cover slab — the concrete above the flutes of the steel deck. All flutes are filled with concrete so as to form a ribbed slab.

Effective cover slab thickness — the minimum thickness of concrete measured from the top of the slab to the top of the steel deck.

Effective slab thickness — the overall slab thickness, provided that the slab is cast
a) with a flat underside;
b) on corrugated steel forms having a height of corrugation not greater than 0.25 times the overall slab thickness; or

c) on fluted steel forms whose profile has the following characteristics:
   i) the minimum concrete rib width is 125 mm;
   ii) the maximum rib height is 40 mm but not more than 0.4 times the overall slab thickness; and
   iii) the average width between ribs does not exceed 0.25 times the overall slab thickness nor 0.2 times the minimum width of concrete ribs.

In all other cases, “effective slab thickness” means the overall slab thickness minus the height of the corrugation or the flute .

Flute — the portion of the steel deck that forms a valley.

Rib — the portion of the concrete slab that is formed by the flute.

Slab — a reinforced cast-in-place concrete slab at least 65 mm in effective thickness. The area equal to the effective width times the effective slab thickness should be free of voids or hollows except for those specifically permitted in the definition of effective slab thickness.

Steel deck — a load-carrying steel deck consisting of a
a) single fluted element (non-cellular deck); or
b) two-element section consisting of a fluted element in conjunction with a flat sheet (cellular deck).

Steel joist — an open-web steel joist suitable for composite design (see Clause 16).

Steel section — a steel structural section with a solid web or webs suitable for composite design. Web openings may be used only if their effects are fully investigated and accounted for in the design.

Steel truss — a steel truss suitable for composite design (see Clause 15).

17.3 General

17.3.1 Deflections

Calculation of deflections shall take into account the effects of creep of concrete, shrinkage of concrete, and increased flexibility resulting from partial shear connection and from interfacial slip. These effects shall be established by test or analysis, where practicable. Consideration shall also be given to the effects of full or partial continuity in the steel beams and concrete slabs in reducing calculated deflections.

In lieu of tests or analysis, the effects of partial shear connection and interfacial slip, creep, and shrinkage may be assessed as follows:

a) for increased flexibility resulting from partial shear connection and interfacial slip, the deflections shall be calculated using an effective moment of inertia given by

   $I_e = I_s + 0.85p^{0.25}(I_t - I_s)$

   where

   I_s = moment of inertia of a steel beam, or of a steel joist or truss adjusted to include the effect of shear deformations, which may be taken into account by decreasing the moment of inertia based on the cross-sectional areas of the top and bottom chords by 15% or by a more detailed analysis

   p = fraction of full shear connection
   = 1.00 for full shear connection

   I_t = transformed moment of inertia of composite beam based on the modular ratio $n = E/E_c$

b) for creep, elastic deflections caused by dead loads and long-term live loads, as calculated in Item a), need to be increased by 15%; and

c) for shrinkage of concrete, using a selected free shrinkage strain, strain compatibility between the steel and concrete, and an age-adjusted effective modulus of elasticity of concrete as it shrinks and creeps, the deflection of a simply supported composite beam, joist, or truss shall be calculated as follows:

$$\Delta_s = \frac{L^2}{8}\psi = \frac{L^2}{8}c\frac{\varepsilon_f A_c y}{n_s I_{es}}$$

where

L = span of the beam, joist, or truss

ψ = curvature along length of the beam, joist, or truss due to shrinkage of concrete

c = empirical coefficient used to match theory with test results (accounting for cracking of concrete in tension, the non-linear stress-strain relationship of concrete, and other factors)

ε_f = free shrinkage strain of concrete

A_c = effective area of concrete slab

y = distance from centroid of effective area of concrete slab to centroidal axis of the composite beam, joist, or truss

n_s = modular ratio, E/E'_c

where

E'_c = $E_c/(1+\chi\phi)$

= age-adjusted effective modulus of elasticity of concrete

where

χ = aging coefficient of concrete

ϕ = creep coefficient of concrete

I_{es} = $I_s + 0.85p^{0.25}(I_{ts} - I_s)$

= effective moment of inertia of composite beam, truss, or joist based on the modular ratio n_s

where

I_{ts} = transformed moment of inertia based on the modular ratio n_s

Note: *For typical values of c, ε_f, χ, and ϕ, see Annex H.*

17.3.2 Vertical shear

The web area of steel sections or the web system of steel trusses and joists shall be proportioned to carry the total vertical shear, V_f.

17.3.3 End connections

End connections of steel sections, trusses, and joists shall be proportioned to transmit the total end reaction of the composite beam.

17.3.4 Steel deck

The maximum depth of the deck shall be 80 mm and the average width of the minimum flute shall be 50 mm. A steel deck may be of a type intended to act compositely with the cover slab in supporting applied load.

17.4 Design effective width of concrete

17.4.1

Slabs or cover slabs extending on both sides of the steel section or joist shall be deemed to have a design effective width, *b*, equal to the lesser of

a) 0.25 times the composite beam span; or
b) the average distance from the centre of the steel section, truss, or joist to the centres of adjacent parallel supports.

17.4.2

Slabs or cover slabs extending on one side only of the supporting section or joist shall be deemed to have a design effective width, *b*, not greater than the width of the top flange of the steel section or top chord of the steel joist or truss plus the lesser of

a) 0.1 times the composite beam span; or
b) 0.5 times the clear distance between the steel section, truss, or joist and the adjacent parallel support.

17.5 Slab reinforcement

17.5.1 General

Slabs shall be adequately reinforced to support all loads and to control both cracking transverse to the composite beam span and longitudinal cracking over the steel section or joist. Reinforcement shall not be less than that required by the specified fire-resistance design of the assembly.

17.5.2 Parallel reinforcement

Reinforcement parallel to the span of the beam in regions of negative bending moment of the composite beam shall be anchored by embedment in concrete that is in compression. The reinforcement of slabs that are to be continuous over the end support of steel sections or joists fitted with flexible end connections shall be given special attention. Reinforcement at the ends of beams supporting ribbed slabs perpendicular to the beam shall be not less than two 15M bars or equivalent.

17.5.3 Transverse reinforcement — Concrete slab on metal deck

Unless it is known from experience that longitudinal cracking caused by composite action directly over the steel section or joist is unlikely, additional transverse reinforcement or other effective means shall be provided. Such additional reinforcement shall be placed in the lower part of the slab and anchored so as to develop the yield strength of the reinforcement. The area of such reinforcement shall be not less than 0.002 times the concrete area being reinforced and shall be uniformly distributed.

17.5.4 Transverse reinforcement — Ribbed slabs

17.5.4.1

Where the ribs are parallel to the beam span, the area of transverse reinforcement shall be not less than 0.002 times the concrete cover slab area being reinforced and shall be uniformly distributed.

17.5.4.2

Where the ribs are perpendicular to the beam span, the area of transverse reinforcement shall be not less than 0.001 times the concrete cover slab area being reinforced and shall be uniformly distributed.

17.6 Interconnection

17.6.1

Except as permitted by Clauses 17.6.2 and 17.6.4, interconnection between steel sections, trusses, or joists and slabs or steel decks with cover slabs shall be attained by the use of shear connectors as specified in Clause 17.7.

17.6.2

Uncoated steel sections, trusses, or joists that support slabs and are totally encased in concrete shall not require interconnection by means of shear connectors, provided that

a) a minimum of 50 mm of concrete covers all portions of the steel section, truss, or joist except as specified in Item c);
b) the cover in Item a) is reinforced to prevent spalling; and
c) the top of the steel section, truss, or joist is at least 40 mm below the top and 50 mm above the bottom of the slab.

17.6.3

Studs may be welded through a maximum of two steel sheets in contact, each not more than 1.71 mm in overall thickness, including coatings (1.52 mm in nominal base steel thickness plus zinc coating not greater than nominal 275 g/m^2). Otherwise, holes for placing studs shall be made through the sheets as necessary. Welded studs shall meet the requirements of CSA W59.

17.6.4

Methods of interconnection other than those specified in Clause 17.7 that have been adequately demonstrated by test and verified by analysis may be used to effect the transfer of forces between the steel section, truss, or joist and the slab or steel deck with cover slab. In such cases, the design of the composite member shall conform, to the extent practicable, to the design of a similar member employing shear connectors.

17.6.5

The diameter of a welded stud shall not exceed 2.5 times the thickness of the part to which it is welded unless test data satisfactory to the designer are provided to establish the capacity of the stud as a shear connector.

17.7 Shear connectors

17.7.1 General

The resistance factor, ϕ_{sc}, to be used with the shear resistances specified in Clause 17.7 shall be taken as 0.80. The factored shear resistance, q_r, of other shear connectors shall be established by tests acceptable to the designer.

17.7.2 End-welded studs

17.7.2.1

End-welded studs shall be headed or hooked with $h/d \geq 4$. The projection of a stud in a ribbed slab, based on its length prior to welding, shall be at least two stud diameters above the top surface of the steel deck. The factored resistance of end-welded studs shall be as specified in Clauses 17.7.2.2 and 17.7.2.4.

17.7.2.2

In solid slabs,

$$q_{rs} = 0.50\phi_{sc} A_{sc} \sqrt{f'_c E_c} \le \phi_{sc} A_{sc} F_u$$

where

q_{rs} = factored shear resistance

F_u = 450 MPa for commonly available studs (CSA W59 Type B studs)

17.7.2.3

In ribbed slabs with ribs parallel to the beam,

a) when $3.0 > w_d/h_d \ge 1.50$:

$$q_{rr} = q_{rs}\left[0.75 + 0.167\left(\frac{w_d}{h_d} - 1.5\right)\right] \le q_{rs}$$

b) when $w_d/h_d < 1.50$:

$$q_{rr} = \phi_{sc}\left[0.92\frac{w_d}{h_d}dh(f'_c)^{0.8} + 1.1sd(f'_c)^{0.2}\right] \le 0.75q_{rs}$$

where

s = longitudinal stud spacing

17.7.2.4

In ribbed slabs with ribs perpendicular to the beam

a) when h_d = 75 mm:

$$q_{rr} = 0.35\phi_{sc}\rho A_p \sqrt{f'_c} \le q_{rs}$$

b) when h_d = 38 mm:

$$q_{rr} = 0.61\phi_{sc}\rho A_p \sqrt{f'_c} \le q_{rs}$$

where

A_p = concrete pullout area, taking the deck profile and stud burnoff into account. For a single stud, the apex of the pyramidal pullout area, with four sides sloping at 45°, shall be taken as the centre of the top surface of the head of the stud. For a pair of studs, the pullout area has a ridge extending from stud to stud

ρ = 1.0 for normal-density concrete (2150 to 2500 kg/m^3)

= 0.85 for semi-low-density concrete (1850 to 2150 kg/m^3)

17.7.2.5

The longitudinal spacing of stud connectors in solid slabs and in ribbed slabs when ribs of formed steel deck are parallel to the beam shall be not less than six stud diameters. The spacing of studs shall not exceed 1000 mm (see also Clause 17.8).

The transverse spacing of stud connectors shall be not less than four stud diameters.

17.7.3 Channel connectors

In solid slabs of normal-density concrete with $f'_c \ge 20$ MPa and a density of at least 2300 kg/m^3, the following shall apply:

$$q_{rs} = 45\phi_{sc}\ (t + 0.5w)L_c\sqrt{f'_c}$$

The spacing of the shear connectors shall be in accordance with Clause 17.9.8.

17.8 Ties

Mechanical ties shall be provided between the steel section, truss, or joist and the slab or steel deck to prevent separation. Shear connectors may serve as mechanical ties if suitably proportioned. The maximum spacing of ties shall not exceed 1000 mm. The average spacing in a span shall not exceed 600 mm or be greater than that required to achieve any specified fire-resistance rating of the composite assembly.

17.9 Design of composite beams with shear connectors

17.9.1

The composite beam shall consist of steel section, truss or joist, shear connectors, ties, and slab or steel deck with cover slab.

The flat width of the top chord or that of a component member of the top chord shall be not less than $1.4d + 20$ mm

where
d = diameter of the stud connector

17.9.2

The properties of the composite section shall be based on the maximum effective area (equal to effective width times effective thickness), neglecting any concrete area in tension. If a steel truss or joist is used, the area of its top chord shall be neglected in determining the properties of the composite section and only Clause 17.9.3 a) shall apply.

17.9.3

The factored moment resistance, M_{rc}, of the composite section with the slab or cover slab in compression shall be calculated as follows, where $\phi = 0.90$, the resistance factor for concrete, $\phi_c = 0.65$, and $\alpha_1 = 0.85 - 0.0015 f'_c$ (but not less than 0.67):

a) Case 1 — full shear connection and plastic neutral axis in the slab, i.e., $Q_r \geq \phi A_s F_y$ and $\phi A_s F_y \leq \alpha_1 \phi_c b t f'_c$

where
Q_r = sum of the factored resistances of all shear connectors between points of maximum and zero moment

$M_{rc} = T_r = \phi A_s F_y e'$

where
e' = the lever arm and is calculated from the equation
$a = \dfrac{\phi A_s F_y}{\alpha_1 \phi_c b f'_c}$

b) Case 2 — full shear connection and plastic neutral axis in the steel section, i.e., $Q_r \geq \alpha_1 \phi_c b t f'_c$ and $\alpha_1 \phi_c b t f'_c < \phi A_s F_y$

$M_{rc} = C_r e + C'_r e'$

where
$C_r = \dfrac{\phi A_s F_y - C'_r}{2}$

$C'_r = \alpha_1 \phi_c b t f'_c$

c) Case 3 — partial shear connection, i.e., $Q_r < \alpha_1 \phi_c b t f'_c$ and $\phi A_s F_y$

$M_{rc} = C_r e + C'_r e'$

where

$$C_r = \frac{\phi A_s F_y - C_r'}{2}$$

$C_r' = Q_r$

where

e' = the lever arm and is calculated from the equation

$$a = \frac{C_r'}{\alpha_1 \phi_c b f_c'}$$

17.9.4

No composite action shall be assumed in calculating

a) flexural strength when Q_r is less than 0.4 times the lesser of $\alpha_1 \phi_c b t f_c'$ and $\phi A_s F_y$; and

b) deflections when Q_r is less than 0.25 times the lesser of $\alpha_1 \phi_c b t f_c'$ and $\phi A_s F_y$.

17.9.5

For full shear connection, the sum of the factored resistances of all shear connectors distributed between the point of maximum bending moment and each adjacent point of zero moment, Q_r, shall equal or exceed the total horizontal shear, V_h, at the junction of the steel section, truss, or joist and the concrete slab or steel deck, calculated as $V_h = \phi A_s F_y$ or $V_h = \alpha_1 \phi_c b t f_c'$ for Cases 1 and 2, as specified in Items a) and b), respectively, of Clause 17.9.3.

17.9.6

For partial shear connection, the total horizontal shear, V_h, as specified in Clause 17.9.3 c), shall be calculated as $V_h = Q_r$.

17.9.7

Composite beams employing steel sections and concrete slabs may be designed as continuous members. The factored moment resistance of the composite section, with the concrete slab in the tension area of the composite section, shall be the factored moment resistance of the steel section alone, except that when sufficient shear connectors are placed in the negative moment region, suitably anchored concrete slab reinforcement parallel to the steel sections and within the design effective width of the concrete slab may be included in calculating the properties of the composite section. The total horizontal shear, V_h, to be resisted by shear connectors between the point of maximum negative bending moment and each adjacent point of zero moment shall be taken as $\phi_r A_r F_{yr}$.

17.9.8

The number of shear connectors to be located on each side of the point of maximum bending moment (positive or negative, as applicable), distributed between that point and the adjacent point of zero moment, shall be not less than

$$n = \frac{V_h}{q_r}$$

Shear connectors may be spaced uniformly, except that in a region of positive bending the number of shear connectors, n', required between any concentrated load applied in that region and the nearest point of zero moment shall be not less than

$$n' = n\left(\frac{M_{f1} - M_r}{M_f - M_r}\right)$$

where
M_{f1} = positive bending moment under factored load at concentrated load point
M_r = factored moment resistance of the steel section alone
M_f = maximum positive bending moment under factored load

17.9.9

In the end panels of composite joists and trusses, the top chord shall be designed to resist all factored forces, ignoring any composite action unless adequate shear connectors are placed over the seat or along a top chord extension to carry horizontal shear. Studs shall not be placed closer than their height to the end of the concrete slab.

17.9.10

The shear that is to be developed on the longitudinal shear surfaces, A_{cv}, of composite beams with solid slabs or with cover slabs and steel deck parallel to the beam shall be taken as

$$V_u = \Sigma q_r - \alpha_1 \phi_c f'_c A_c - \phi_r A_r F_{yr}$$

where
A_r = area of longitudinal reinforcement within the concrete area, A_c

For normal-weight concrete, the factored shear resistance along any potential longitudinal shear surfaces in the concrete slab shall be taken as

$$V_r = \left(0.80\phi_r A_r F_{yr} + 2.76\phi_c A_{cv}\right) \leq 0.50\phi_c f'_c A_{cv}$$

where
A_r = area of transverse reinforcement crossing shear planes, A_{cv}

17.10 Design of composite beams without shear connectors

17.10.1

Uncoated steel sections or joists supporting concrete slabs and encased in concrete in accordance with Clause 17.6.2 may be proportioned based on the assumption that the composite section supports the total load.

17.10.2

The properties of the composite section for determination of load-carrying capacity shall be calculated using ultimate strength methods, neglecting any area of concrete in tension.

17.10.3

As an alternative method of design, encased simple-span steel sections or joists may be proportioned based on the assumption that the steel section, truss, or joist alone supports 0.90 times the total load.

17.11 Unshored beams

For composite beams that are unshored during construction, the stresses in the tension flange of the steel section, truss, or joist due to the loads applied before the concrete strength reaches $0.75f'_c$ plus the stresses at the same location due to the remaining specified loads considered to act on the composite section shall not exceed F_y.

17.12 Beams during construction

The steel section, truss, or joist alone shall be proportioned to support all factored loads applied prior to hardening of the concrete without exceeding its calculated capacity under the conditions of lateral support or shoring, or both, to be furnished during construction.

18 Composite columns

18.1 Resistance prior to composite action

The factored resistance of the steel member prior to the attainment of composite action shall be determined in accordance with Clause 13.

18.2 Concrete-filled hollow structural sections

18.2.1 General

18.2.1.1 Scope

Clause 18.2 applies to composite members consisting of steel hollow structural sections completely filled with concrete, provided that

a) the width-to-thickness ratio of the walls of rectangular hollow structural sections does not exceed $\frac{1350}{\sqrt{F_y}}$;

b) the outside diameter-to-thickness ratio of circular hollow structural sections does not exceed 28 000/F_y; and

c) the concrete strength is between 20 and 80 MPa for axially loaded columns and between 20 and 40 MPa for columns subjected to axial compression and bending.

18.2.1.2 Axial load on concrete

The axial load assumed to be carried by the concrete at the top level of a column shall be only that portion applied by direct bearing on concrete. At the bottom of a column, a base plate or other means shall be provided for load transfer. At intermediate floor levels, direct bearing on the concrete shall not be considered necessary.

18.2.1.3 Composite action in bending

Full composite resistance as specified in Clause 18.2.3 may be developed at the ends of concrete-filled hollow structural members in bending or combined axial-bending, e.g., at column bases, only if the connection is able to transfer the forces from both the steel and concrete elements to the adjacent structural elements.

18.2.2 Compressive resistance

The factored compressive resistance of a composite concrete-filled hollow structural section shall be taken as

$$C_{rc} = (\tau \phi A_s F_y + \tau' \alpha_1 \phi_c A_c f'_c)(1 + \lambda^{2n})^{1/n}$$

where

τ = τ'

= 1.0, except for circular hollow structural sections with a height-to-diameter ratio (L/D) of less than 25 for which

$$\tau = \frac{1}{\sqrt{1+\rho+\rho^2}}$$

and

$$\tau' = 1 + \left(\frac{25\rho^2\tau}{D/t}\right)\left(\frac{F_y}{\alpha_1 f'_c}\right)$$

where

ρ = 0.02 (25 – L/D)

α_1 = 0.85 – 0.0015f'_c (but not less than 0.73)

$$\lambda = \sqrt{\frac{C_p}{C_{ec}}}$$

where

$C_p = C_{rc}$, computed with $\phi = \phi_c = 1.0$ and $\lambda = 0$

$$C_{ec} = \frac{\pi^2 EI_e}{(KL)^2}$$

where

$$EI_e = EI_s + \frac{0.6E_cI_c}{1 + C_{fs}/C_f}$$

where

I_s and I_c = moment of inertia of the steel and concrete areas, respectively, as computed with respect to the centre of gravity of the cross-section

E_c = modulus of elasticity of concrete as defined in Clause 3

C_{fs} = sustained axial load on the column

C_f = total axial load on the column

n = 1.80

18.2.3 Bending resistance

The factored bending resistance of a composite concrete-filled hollow structural section shall be taken as

$M_{rc} = C_r e + C'_r e'$

where

a) for a rectangular hollow structure section:

$$C_r = \frac{\phi A_s F_y - C'_r}{2}$$

$C'_r = 1.18\,\alpha_1\phi_c a\,(b - 2t)\,f'_c$

$C_r + C'_r = T_r$

$= \phi A_{st} F_y$

Note: *The concrete in compression is taken to have a rectangular stress block of intensity f'_c over a depth of a.*

b) for a circular hollow structural section:

$$C_r = \phi F_y \beta \frac{Dt}{2}$$

$$C'_r = 1.18\alpha_1 \phi_c f'_c \left[\frac{\beta D^2}{8} - \frac{b_c}{2}\left(\frac{D}{2} - a\right)\right]$$

$$e = b_c\left[\frac{1}{(2\pi - \beta)} + \frac{1}{\beta}\right]$$

$$e' = b_c\left[\frac{1}{(2\pi - \beta)} + \frac{b_c^2}{1.5\beta D^2 - 6b_c(0.5D - a)}\right]$$

where

β = value in radians found from the recursive equation

$$\beta = \frac{\phi A_s F_y + 0.295\alpha_1 \phi_c D^2 f'_c\,[\sin(\beta/2) - \sin^2(\beta/2)\tan(\beta/4)]}{\left(0.148\alpha_1 \phi_c D^2 f'_c + \phi D t F_y\right)}$$

$$b_c = D\sin\left(\frac{\beta}{2}\right)$$

$$a = \frac{b_c}{2}\tan\left(\frac{\beta}{4}\right)$$

Conservatively, M_{rc} may be taken as

$$M_{rc} = (Z - 2th_n^2)\phi F_y + \left[\frac{2}{3}(0.5D - t)^3 - (0.5D - t)h_n^2\right]1.18\alpha_1 \phi_c f'_c$$

where

Z = the plastic modulus of the steel section alone

$$h_n = \frac{1.18\alpha_1 \phi_c A_c f'_c}{2.36D\alpha_1 \phi_c f'_c + 4t(2\phi F_y - 1.18\alpha_1 \phi_c f'_c)}$$

α_1 = value as defined in Clause 18.2.2

18.2.4 Axial compression and bending

Composite concrete-filled hollow structural sections required to resist both bending moments and axial compression shall be proportioned analogously to members conforming to Clause 13.8.2 so that

$$\frac{C_f}{C_{rc}} + \frac{\beta \omega_1 M_f}{M_{rc}\left(1 - \frac{C_f}{C_{ec}}\right)} \le 1.0 \text{ and}$$

$$\frac{M_f}{M_{rc}} \le 1.0$$

where

$$\beta = \frac{C_{rco} - C_{rcm}}{C_{rco}}$$

where
C_{rco} = factored compressive resistance with $\lambda = 0$

C_{rcm} = $1.18\,\alpha_1 \phi_c A_c f'_c$

where
α_1 = value as defined in Clause 18.2.2

M_{rc} = value as defined in Clause 18.2.3

18.3 Partially encased composite columns

Note: *The Canam Group Inc. holds patents on the partially encased composite columns described in this Clause. Canam Group Inc. will make available any patent rights to interested applicants, wherever located, either as a free licence or on reasonable terms and conditions.*

18.3.1 General

Clause 18.3 applies to doubly symmetrical composite members consisting of three-plate built-up steel H-sections, with plain tie bars welded between the flange tips at regular intervals, in which the cells between the column flanges and the web are completely filled with concrete in the field during construction, provided that

a) concrete is of normal density and has a compressive strength, f'_c, between 20 and 70 MPa;
b) $A_s + A_r \leq 0.20$ of the gross cross-sectional area;
c) the full width of flange, b_f, is between 0.9 and 1.1 times the section depth, d;
d) the flanges and the web are of equal thickness, t;
e) the flange width-to-thickness ratio is not greater than 32;
f) a pair of continuous fillet welds, sufficient to develop the shear yield capacity of the web, connects the web to each flange;
g) the vertical spacing of tie bars, s, does not exceed the lesser of 500 mm or two-thirds of the least dimension of the cross-section. The area of a tie bar shall be taken as the greatest of
   i) 63 mm^2;
   ii) $0.01 b_f t$; and
   iii) 0.5 mm^2 per mm of tie bar spacing;
h) the tie bars are welded to the flanges to develop the yield strength of the tie bars and the cover of the tie bars is at least 30 mm;
i) out-of-straightness of the flanges, as measured between any two adjacent ties along the column edges, does not exceed 0.005 times the tie spacing;
j) the specified yield strength of structural steel, F_y, does not exceed 350 MPa;
k) the specified yield strength of reinforcement, F_{yr}, does not exceed 400 MPa; and
l) the clear height-to-width ratio of the column does not exceed 14.

18.3.2 Compressive resistance

The factored compressive resistance of a partially encased three-plate built-up composite column shall be taken as

$$C_{rc} = (\phi A_{se} F_y + 0.95 \alpha_1 \phi_c A_c f'_c + \phi_r A_r F_{yr})(1 + \lambda^{2n})^{1/n}$$

where
A_{se} = effective steel area of the steel section
$= (d - 2t + 2b_e)t$

where

$$b_e = \frac{b_f}{\left(1 + \lambda_p^3\right)^{1/1.5}} \le b_f$$

where

$$\lambda_p = \frac{b_f}{t}\sqrt{\frac{F_y}{720\,000k}}$$

where

$$k = \frac{0.9}{(s / b_f)^2} + 0.2(s / b_f)^2 + 0.75$$

α_1 = value specified in Clause 18.2.2

A_r = area of longitudinal reinforcement

$$\lambda = \sqrt{\frac{C_p}{C_{ec}}}$$

where

C_p = C_{rc} computed with ϕ, ϕ_c, and ϕ_r = 1.0 and λ = 0

C_{ec} = value specified in Clause 18.2.2

n = 1.34

18.3.3 Bending resistance

The factored bending resistance of a partially encased three-plate built-up composite column shall be taken as

$$M_{rc} = C_r e + C'_r e'$$

where

$$C_r = \frac{\phi A_s F_y - C'_r}{2}$$

$C_r + C'_r$ = T_r

= $\phi A_{st} F_y$

C'_r = $1.18\alpha_1\phi_c a\,(b - t)\,f'_c$ for strong axis bending

C'_r = $1.18\alpha_1\phi_c a\,(b - 2t)\,f'_c$ for weak axis bending

Note: *The concrete in compression is taken to have a rectangular stress block of intensity f'_c over a depth of a.*

18.3.4 Axial compression and bending

Partially encased three-plate built-up composite columns required to resist both bending moments and axial compression shall be proportioned so that

$$\frac{C_f}{C_{rc}} + \frac{M_{fx}}{M_{rcx}} + \frac{M_{fy}}{M_{rcy}} \le 1$$

18.3.5 Special reinforcement for seismic zones

18.3.5.1
Columns larger than 500 mm in depth in buildings where the specified one-second spectral acceleration ratio ($I_E F_a S_a(1.0)$) is greater than 0.30 shall be reinforced with longitudinal and transverse bars.

18.3.5.2
The longitudinal bars specified in Clause 18.3.5.1 shall
a) have an area not less than 0.005 times the total gross cross-sectional area;
b) be at least two in number in each cell; and
c) be positioned against the tie bars and at a spacing not greater than the tie spacing, s.

18.3.5.3
The transverse bars specified in Clause 18.3.5.1 shall
a) be U-shaped 15M bars arranged to provide corner support to at least every alternate longitudinal bar in such a way that no unsupported longitudinal bar is farther than 150 mm clear from a laterally supported bar;
b) have ends welded to the web of the steel shape, in line with the ends of the transverse bars located in the opposite cell, or ends anchored within the concrete core located on the opposite side of the web; and
c) have a vertical spacing not greater than the tie spacing, s, or 16 times the diameter of the smallest longitudinal bar.

18.4 Encased composite columns

18.4.1 General
Clause 18.4 applies to doubly symmetrical steel columns encased in concrete, provided that
a) the steel shape is a Class 1, 2, or 3 section;
b) $A_s \geq 0.04$ of the gross cross-sectional area;
c) $A_s + A_r \leq 0.20$ of the gross cross-sectional area;
d) the concrete is of normal density and has a compressive strength, f'_c, between 20 and 55 MPa;
e) the specified yield strength of structural steel, F_y, does not exceed 350 MPa; and
f) the specified yield strength of reinforcement, F_{yr}, does not exceed 400 MPa.

18.4.2 Compressive resistance
The factored compressive resistance of a steel concrete-encased composite column shall be taken as

$$C_{rc} = (\phi A_s F_y + \alpha_1 \phi_c A_c f'_c + \phi_r A_r F_{yr})(1 + \lambda^{2n})^{1/n}$$

where
α_1 = value specified in Clause 18.2.2
A_r = value specified in Clause 18.3.2
λ = value specified in Clause 18.3.2
n = value specified in Clause 18.3.2

18.4.3 Reinforcement

18.4.3.1
The concrete encasement shall be reinforced with longitudinal bars and lateral ties extending completely around the structural steel core. The clear cover shall be not less than 40 mm.

The longitudinal bars shall
a) be continuous at framed levels when considered to carry load;
b) have an area not less than 0.01 times the total gross cross-sectional area;
c) be located at each corner; and
d) spaced on all sides not further apart than the lesser of $525t/\sqrt{F_y}$ and one-half the least dimension of the composite section.

18.4.3.2
The lateral ties shall
a) be 15M bars, except that 10M bars may be used when no side dimension of the composite section exceeds 500 mm; and
b) have a vertical spacing not exceeding the least of the following:
   i) two-thirds of the least side dimension of the cross-section;
   ii) 16 longitudinal bar diameters; or
   iii) 500 mm.

18.4.4 Columns with multiple steel shapes
Where the composite cross-section includes two or more steel shapes, the steel shapes shall be considered built-up members subject to the requirements of Clause 19 until the concrete strength reaches $0.75f'_c$.

18.4.5 Load transfer
The portion of the total axial load resisted by the concrete shall be developed by direct bearing at connections. The bearing strength of concrete may be taken as $1.95\phi_c\alpha_1 f'_c A_L$, where A_L is the loaded area, provided that the concrete is restrained against lateral expansion.

18.4.6 Bending resistance
The bending resistance of encased composite columns may be determined according to the Structural Stability Research Council's *Guide to Stability Design Criteria for Metal Structures*.

19 Built-up members

19.1 General
Components of built-up members shall be joined for the applied forces and other minimum connection requirements specified in this Clause.

Note: *The use of fillet welds or partial penetration welds, instead of complete joint penetration welds, is encouraged. If undermatching is permitted per CSA W59, this also needs to be considered. This will provide better ductility, improve fracture resistance, minimize lamellar tearing, and minimize distortion of the overall built-up section.*

19.2 Members in compression

19.2.1
All components of built-up compression members and the transverse spacing of their lines of connecting bolts or welds shall meet the requirements of Clauses 10 and 11.

19.2.2
Component parts that are in contact with one another at the ends of built-up compression members shall be connected by
a) bolts spaced longitudinally not more than four diameters apart for a distance equal to 1.5 times the width of the member; or
b) continuous welds having a length of not less than the width of the member.

19.2.3
Unless closer spacing is required for transfer of load or sealing inaccessible surfaces, the longitudinal spacing in-line between intermediate bolts or the clear longitudinal spacing between intermittent welds for the outside plate component of built-up compression members shall not exceed the following, where t is the thickness of the outside plate:
a) when the bolts or intermittent welds are staggered on adjacent lines: $525t / \sqrt{F_y}$, but not more than 450 mm; and
b) when the bolts on all gauge lines or intermittent welds along the component edges are not staggered: $330t / \sqrt{F_y}$, but not more than 300 mm.

19.2.4
Compression members composed of two or more shapes in contact or separated from one another shall be interconnected in such a way that the slenderness ratio of any component, based on its least radius of gyration and the distance between interconnections, shall not exceed that of the built-up member. The compressive resistance of the built-up member shall be based on
a) the slenderness ratio of the built-up member with respect to the appropriate axis, when the buckling mode does not involve relative deformation that produces shear forces in the interconnectors; or
b) an equivalent slenderness ratio, with respect to the axis orthogonal to that in Item a), when the buckling mode involves relative deformation that produces shear forces in the interconnectors, taken as follows:

$$\rho_e = \sqrt{\rho_o^2 + \rho_i^2}$$

where
ρ_e = equivalent slenderness ratio of the built-up member
ρ_o = slenderness ratio of the built-up member acting as an integral unit
ρ_i = maximum slenderness ratio of component part of the built-up member between interconnectors

For built-up members composed of two interconnected shapes, e.g., back-to-back angles or channels, in contact or separated only by filler plates, the maximum slenderness ratio of component parts between fasteners or welds shall be based on an effective length factor of 1.0 when the fasteners are snug-tight bolts and 0.65 when welds or pretensioned bolts are used.

For built-up members composed of two interconnected shapes separated by lacing or batten plates, the maximum slenderness ratio of component parts between fasteners or welds shall be based on an effective length factor of 1.0 for both snug-tight and pretensioned bolts and for welds.

For compound compression members, connections at the ends and interconnectors should be capable of transferring the shears and moments through a rigid connection up to the factored load levels.

19.2.5

For starred angle compression members interconnected at least at the one-third points, Clause 19.2.4 need not apply.

19.2.6

The fasteners and interconnecting parts, if any, of members identified in Clause 19.2.4 shall be proportioned to resist a force equal to 0.01 times the total force in the built-up member.

19.2.7

Spacing requirements of Clauses 19.2.3, 19.3.3, and 19.3.4 might not always provide a continuous tight fit between components in contact. When the environment is such that corrosion could be a serious problem, it is possible that the spacing of bolts or welds will need to be less than the specified maximum.

19.2.8

Open sides of compression members built up from plates or shapes shall be connected to each other by lacing, batten plates, or perforated cover plates.

19.2.9

Lacing shall provide a complete triangulated shear system and may consist of bars, rods, or shapes. Lacing shall be proportioned to resist a shear normal to the longitudinal axis of the member of not less than 0.025 times the total axial load on the member plus the shear from transverse loads, if any.

19.2.10

The slenderness ratio of lacing members shall not exceed 140. The effective length for single lacing shall be the distance between connections to the main components; for double lacing connected at the intersections, the effective length shall be half of that distance.

19.2.11

Lacing members shall be inclined preferably to the longitudinal axis of the built-up member at an angle of not less than 45°.

19.2.12

Lacing systems shall have diaphragms in the plane of the lacing and as near to the ends as practicable, as well as at intermediate points where lacing is interrupted. Such diaphragms may be plates (tie plates) or shapes.

19.2.13

End tie plates used as diaphragms shall have a length not less than the distance between the lines of bolts or welds connecting them to the main components of the member. Intermediate tie plates shall be at least one-half the specified length of end tie plates. The thickness of tie plates shall be at least 1/60 of the width between lines of bolts or welds connecting them to the main components, and the longitudinal spacing of the bolts or clear longitudinal spacing between welds shall not exceed 150 mm. At least three bolts shall connect the tie plate to each main component or a total length of weld not less than one-third the length of tie plate shall be used.

19.2.14

Shapes used as diaphragms shall be proportioned and connected to transmit a longitudinal shear equal to 0.05 times the axial compression in the member from one main component to the other.

19.2.15

Perforated cover plates may be used in lieu of lacing and tie plates on open sides of built-up compressive members. The net width of such plates at access holes may be assumed to resist axial load, provided that

a) the width-to-thickness ratio is as specified in Clause 11;
b) the length of the access hole does not exceed twice its width;
c) the clear distance between access holes in the direction of load is not less than the transverse distance between lines of bolts or welds connecting the perforated plate to the main components of the built-up member; and
d) the periphery of the access hole has a minimum radius of 40 mm at all points.

19.2.16

Battens consisting of plates or shapes may be used on open sides of built-up compression members that do not carry primary bending in addition to axial load. Battens shall be provided at the ends of the member, at locations where the member is laterally supported along its length, and elsewhere as required by Clause 19.2.4.

19.2.17

Battens shall have a length of not less than the distance between lines of bolts or welds connecting them to the main components of the member and shall have a thickness of not less than 1/60 of this distance if the batten consists of a flat plate. Battens and their connections shall be proportioned to resist the following simultaneously:

a) a longitudinal shear force $V_f = \dfrac{0.025\, C_f d}{na}$; and
b) a moment $M_f = 0.025\, C_f d/2n$

where

d = longitudinal centre-to-centre distance between battens

n = number of parallel planes of battens

a = distance between lines of bolts or welds connecting the batten to each main component

19.3 Members in tension

19.3.1

Members in tension composed of two or more shapes, plates, or bars separated from one another by intermittent fillers shall have the components interconnected at fillers spaced so that the slenderness ratio of any component between points of interconnection shall not exceed 300.

19.3.2

Members in tension composed of two plate components in contact or a shape and a plate component in contact shall have the components interconnected so that the spacing between connecting bolts or clear spacing between welds does not exceed the lesser of 36 times the thickness of the thinner plate or 450 mm (see Clause 19.2.3).

19.3.3

Members in tension composed of two or more shapes in contact shall have the components interconnected so that the spacing between connecting bolts or the clear spacing between welds does not exceed 600 mm, except where it can be determined that a greater spacing would not affect the satisfactory performance of the member (see Clause 19.2.3).

19.3.4

Members in tension composed of two separated main components may have perforated cover plates or tie plates on the open sides of the built-up member. Tie plates, including end tie plates, shall have a length of not less than two-thirds of the transverse distance between bolts or welds connecting them to the main components of the member and shall be spaced so that the slenderness ratio of any component between the tie plates does not exceed 300. The thickness of tie plates shall be at least 1/60 of the transverse distance between the bolts or welds connecting them to the main components, and the longitudinal spacing of the bolts or welds shall not exceed 150 mm. Perforated cover plates shall meet the requirements of Items b), c), and d) of Clause 19.2.15.

19.4 Open box-type beams and grillages

Two or more rolled beams or channels used side by side to form a flexural member shall be connected at intervals of not more than 1500 mm. Through-bolts and separators may be used, provided that, in beams having a depth of 300 mm or more, not fewer than two bolts are used at each separator location. When concentrated loads are carried from one beam to the other or distributed between the beams, diaphragms having sufficient stiffness to distribute the load shall be bolted or welded between the beams. The design of members shall provide for torsion resulting from any unequal distribution of loads. Where beams are exposed, they shall be sealed against corrosion of interior surfaces or spaced sufficiently far apart to permit cleaning and coating.

20 Plate walls

20.1 General

20.1.1 Definition

A plate wall is a lateral-force-resisting structural system consisting of a framework of columns and beams, with relatively thin infill plates in the plane of the frame connected all around to the surrounding members. Frame connections between the beams and columns may be moment-resisting or simple shear connections.

20.1.2 Lateral resistance

Lateral storey shears are considered to be carried by a combination of frame action, if applicable, and post-buckling tension fields that develop in the infill plates parallel to the direction of the principal tensile stresses. Axial forces and moments develop in the beams and columns of plate walls as a result of the

a) response of the wall to the overall bending and shear; and

b) tension field action in the adjacent infill plates.

20.2 Seismic applications

Under seismic loading, plate walls shall meet the additional requirements of Clause 27.9 or 27.10, as appropriate.

20.3 Analysis

Forces and moments in the members and connections, including those resulting from tension field action, may be determined from a plane frame analysis, with the infill plates represented by a series of inclined pin-ended strips.

20.4 Angle of inclination

20.4.1

When the aspect ratio of the panel lies within the limits $0.6 \le L/h \le 2.5$, the angle of inclination from the vertical, α, of the inclined pin-ended strips may be taken as 40°. Otherwise, it shall be determined as follows and shall be between 38° and 45°:

$$\tan^4\alpha = \frac{1 + \frac{wL}{2A_c}}{1 + wh\left(\frac{1}{A_b} + \frac{h^3}{360 I_c L}\right)}$$

where

w = infill plate thickness

L = centre-to-centre distance between columns

A_c = cross-sectional area of column

h = storey height

A_b = cross-sectional area of beam

I_c = moment of inertia of column

20.4.2

A single angle of inclination taken as the average for all the panels may be used to analyze the entire plate wall.

20.5 Limits on column and beam flexibilities

20.5.1

The column flexibility parameter at each panel, ω_h, shall be determined as follows and shall not exceed 2.5:

$$\omega_h = 0.7h\left(\frac{w}{2LI_c}\right)^{0.25}$$

This requirement is met by providing columns with moments of inertia, I_c, greater than or equal to $0.0031wh^4/L$.

20.5.2

The boundary member flexibility parameter for the extreme panels, ω_L, shall be determined as follows:

a) not exceed 2.5 at the top panel of the plate wall;

b) not exceed 2.0 at the bottom panel of the plate wall; and

c) be greater than $0.84\omega_h$:

$$\omega_L = 0.7\left(\left(\frac{h^4}{I_c} + \frac{L^4}{I_b}\right)\frac{w}{4L}\right)^{0.25}$$

These requirements are met by providing a beam with a moment of inertia, I_b, greater than or equal to $\frac{wL^4}{650L - (wh^4 / I_c)}$ for the top beam and $\frac{wL^4}{267L - (wh^4 / I_c)}$ for the bottom beam, if present. See also Clause 20.9.2.

20.6 Infill plates

The factored tensile resistance of the inclined infill plate strips shall be calculated in accordance with Clause 13.2.

20.7 Beams

Beams shall be proportioned to resist bending moments and axial compressive forces in accordance with Clause 13.8. Infill plates shall not be deemed to provide lateral support to adjacent beams. Either Class 1 or 2 sections may be used, except as required by Clause 27.9.3.1.

20.8 Columns

Columns shall be Class 1 sections and proportioned to resist bending moments and axial forces in accordance with Clause 13.8 or 13.9, as appropriate. Infill plates shall not be deemed to provide lateral support to adjacent columns.

20.9 Anchorage of infill plates

20.9.1

At the top panel, the vertical component of the infill plate tension field shall be anchored to a beam meeting the requirements of Clause 20.5.2.

20.9.2

At the bottom panel, the vertical component of the infill plate tension field shall be anchored by connecting the infill plate directly to the substructure or to a beam that meets the requirements of Clause 20.5.2.

20.9.3

At the bottom panel, the horizontal component of the infill plate tension field shall be transferred to the substructure.

20.10 Infill plate connections

Infill plates shall be connected to the surrounding beams and columns. These connections and, if required, any infill plate splices shall be in accordance with Clause 13.12 or 13.13. The factored ultimate tensile strength of the infill plate strips shall be developed by the connections.

21 Connections

21.1 Alignment of members

Axially-loaded members that meet at a joint shall have their centroidal axes intersect at a common point if practicable. Bending resulting from joint eccentricity shall be taken into account.

21.2 Unrestrained members

Except as otherwise indicated in the structural design documents, all connections of beams, girders, and trusses shall be designed and detailed as flexible and ordinarily may be proportioned for the reaction shears only. Flexible beam connections shall accommodate end rotations of unrestrained (simple) beams. To accomplish this, inelastic action at the specified load levels in the connection is permitted.

21.3 Restrained members

When beams, girders, or trusses are subject to both reaction shear and end moment due to full or partial end restraint or to continuous or cantilever construction, their connections shall be designed for the combined effect of shear, bending, and axial load.

When beams are rigidly framed to the flange of an I-shaped column and the distance from the end of the column to the top flange of the beam is greater than the depth of the column, stiffeners shall be provided on the column web if the following bearing and tensile resistances of the column are exceeded:

a) opposite the compression flange of the beam:

$$B_r = \phi_{bi} w_c (t_b + 10t_c) F_{yc} < \frac{M_f}{d_b}$$

except when the column has a Class 3 or 4 web, in which case the following shall apply:

$$B_r = \frac{640\,000 \phi_{bi} w_c (t_b + 10t_c)}{(h_c / w_c)^2}$$

b) opposite the tension flange of the beam when the connected element is

i) welded to the column:

$$T_r = 7\phi t_c^2 F_{yc} < \frac{M_f}{d_b}$$

ii) bolted to the column with two rows of bolts centered about the web of column and the tension flange of the beam:

$$T_r = \phi 2 t_c^2 F_{yc}\left[\sqrt{\frac{b_c}{g}} + \frac{e + c_b}{g}\right] < \frac{M_f}{d_b}$$

where

w_c = thickness of column web

t_b = thickness of beam flange

t_c = thickness of column flange

b_c = width of column flange, but not to be taken as greater than $0.25(9g - 5w_c)$

g = bolt gauge, spacing of the tension bolts transverse to long axis of the column

c_b = bolt spacing between two bolt rows in tension, taken parallel to the long axis of the column, but not to be taken as greater than $2\sqrt{b_c \times g}$

e = distance from the free end of the unstiffened column to the nearest bolt row in tension, but e is not to be taken as greater than $\sqrt{b_c \times g}$

F_{yc} = specified yield point of column
d_b = depth of beam
h_c = clear depth of column web

The stiffener or pair of stiffeners opposite either beam flange shall develop a force, F_{st}, equal to

$(M_f/d_b) - B_r$.

Stiffeners shall also be provided on the web of columns, beams, or girders if V_r calculated from Clause 13.4.2 is exceeded, in which case the stiffener or stiffeners shall transfer a shear force, $V_{st,}$ equal to

$V_f - 0.8\phi A_w F_s$

The stiffeners shall be connected so that the force in the stiffener is transferred through the stiffener connection. When beams frame to one side of the column only, the stiffeners need not be longer than one-half of the depth of the column. When an axial tension or compression force is acting on the beam, its effects (additive only) shall be considered in the design of the stiffeners.

When beams are rigidly framed to the flange of an I-shaped column and the distance from the end of the column to the top flange of the beam is less than or equal to the depth of the column, the requirement of stiffeners shall be evaluated by rational analysis. In lieu of rational analysis, stiffeners shall be provided.

21.4 Connections of tension or compression members

The connections at ends of compression members not finished to bear or of tension members shall be designed for the full factored load effect.

21.5 Bearing joints in compression members

Where columns or other compression members bear on bearing plates or are finished to bear at splices, there shall be sufficient fasteners or welds to hold all parts securely in place to provide a satisfactory level of structural integrity (see Clauses 6.1.2, 28.5, and 29.3.9). The flanges of single web members shall be connected.

21.6 Lamellar tearing

Corner or T-joint details of rolled structural members or plates involving transfer of tensile forces in the through-thickness direction resulting from shrinkage due to welding executed under conditions of restraint shall be avoided where possible. If this type of connection cannot be avoided, measures shall be taken to address the possibility of lamellar tearing.

21.7 Placement of fasteners and welds

Except in members subject to fatigue (see Clause 26) and in braces subject to seismic loads (see Clause 27.5.4.1),disposition of fillet welds to balance the forces about the neutral axis or axes for end connections of single-angle, double-angle, or similar types of axially loaded members shall not be required. Eccentricity between the centroidal axes of such members and the gauge lines of bolted end connections may also be neglected. In axially loaded members subject to fatigue, the fasteners or welds in end connections shall have their centroid on the centroidal axis of the member unless provision is made for the effect of the resulting eccentricity.

21.8 Fillers

21.8.1 Fillers in bolted connections

21.8.1.1

When load-carrying fasteners pass through fillers with a total thickness greater than 19 mm, the fillers shall be extended beyond the splice material and the filler extension shall be secured by sufficient fasteners to distribute the total force in the connected element uniformly over the combined cross-section of the connected element and the filler. , If the filler extension is not provided and/or the filler is not secured by sufficient fasteners, an equivalent number of fasteners shall be included in the connection.

21.8.1.2

When load-carrying fasteners pass through fillers with a total thickness between 6.4 and 19 mm, the shear capacity of the fasteners shall be reduced to account for bending in the fasteners by R_v, as follows:

$$R_v = 1.1 - 0.0158t$$

where
t = thickness of the fillers

Alternatively, the fillers shall be extended beyond the splice material and the filler extension shall be secured by sufficient fasteners to distribute the total force in the connected element uniformly over the combined cross-section of the connected element and the filler or an equivalent number of fasteners shall be included in the connection.

21.8.1.3

When load-carrying fasteners pass through fillers with a total thickness less than or equal to 6.4 mm, the shear capacity of the fasteners need not be reduced.

21.8.2 Fillers in welded connections

In welded construction, any filler with a total thickness greater than 6 mm shall extend beyond the edges of the splice plate and shall be welded to the part on which it is fitted with sufficient weld to transmit the splice plate load, applied at the surface of the filler, as an eccentric load. Welds that connect the splice plate to the filler shall be sufficient to transmit the splice plate load and shall be long enough to avoid overloading the filler along the toe of the weld. Any filler that is 6 mm or less in thickness shall have its edges made flush with the edges of the splice plate and the required weld size shall be equal to the thickness of the filler plate plus the size necessary to transmit the splice plate load.

21.9 Welds in combination

If two or more of the general types of weld (groove, fillet, plug, or slot) are combined in a single connection, the effective capacity of each shall be calculated separately with reference to the axis of the group to determine the factored resistance of the combination.

21.10 Fasteners and welds in combination

21.10.1 New connections

The strength of a joint that combines welds and bolts in the same plane shall be proportioned in accordance with Clause 13.14.

21.10.2 Existing connections

The strength of a joint that combines welds and bolts in the same plane shall be proportioned in accordance with Clause 13.14.

The loads that are being carried by the existing welds and/or bolts at the time that the new fasteners are installed shall be considered when determining the strength of the joint.

21.11 High-strength bolts (in slip-critical joints) and rivets in combination

In making alterations, rivets and high-strength bolts in slip-critical joints may be considered as sharing forces caused by specified dead and live loads.

21.12 Connected elements under combined tension and shear stresses

Except as noted elsewhere in this Standard, welded connection plates under combined normal stress, σ_n, and shear stress, τ, shall be proportioned such that $\tau \leq 0.66\phi F_y$ and $\sigma_n \leq \sigma_{nR}$, where

when $\tau \leq 0.5\phi F_y$, $\sigma_{nR} = \phi F_y$

when $\tau > 0.5\phi F_y$, $\sigma_{nR} = 25/4\ (0.66\phi F_y - \tau)$

22 Design and detailing of bolted connections

22.1 General

Clause 22 deals primarily with ASTM A325, ASTM A325M, ASTM A490, ASTM A490M, ASTM F1852, and ASTM F2280 bolt assemblies and equivalent fasteners. The bolts may be required to be installed to a specified minimum tension, depending on the type of connection.

22.2 Design of bolted connections

22.2.1 Use of snug-tightened high-strength bolts

Snug-tightened high-strength bolts may be used in connections other than those specified in Clause 22.2.2 (see Clause 23.6).

22.2.2 Use of pretensioned high-strength bolts

Pretensioned high-strength bolts (ASTM A325, ASTM A325M, ASTM A490, ASTM A490M, ASTM F1852, and ASTM F2280) shall be used in

a) slip-critical connections where slippage cannot be tolerated (e.g., connections subject to fatigue or frequent load reversal, or connections in structures that have rigorous deflection or stiffness limit states);
b) shear connections, when required by Clause 27.1;
c) all elements resisting crane loads;
d) connections subject to impact or cyclic loading;
e) connections where the bolts are subject to tensile loading (see Clause 13.12.1.3); and
f) connections using oversize or long slotted holes (unless specifically designed to accommodate movement).

22.2.3 Joints subject to fatigue loading

Joints subject to fatigue loading shall be proportioned in accordance with Clause 26.

22.2.4 Effective bearing area

The effective bearing area of bolts shall be the nominal diameter multiplied by the length in bearing. For countersunk bolts, half of the depth of the countersink shall be deducted from the bearing length.

22.2.5 Fastener components

22.2.5.1 Structural bolt assemblies

Except as specified in Clause 22.2.5.3, bolts, nuts, and washers for structural bolt assemblies shall meet the requirements of ASTM A325, ASTM A325M, ASTM A490, ASTM A490M, ASTM F1852, or ASTM F2280.

22.2.5.2 Galvanized bolt assemblies

Galvanized ASTM A325 and ASTM A325M bolt assemblies shall meet the galvanizing requirements of ASTM A325 and ASTM A325M.

22.2.5.3 Zinc/aluminum coated bolt assemblies

Zinc/aluminum coated ASTM A325, ASTM A325M, ASTM A490, and ASTM A490M bolt assemblies shall meet the coating requirements of ASTM F1136.

22.2.5.4 Alternatives to ASTM A325, ASTM A325M, ASTM A490, and ASTM A490M bolt assemblies

Other fasteners may be used if they meet the chemical and mechanical requirements of ASTM A325, ASTM A325M, ASTM A490, or ASTM A490M and have body diameters and bearing areas under the head and nut specified in those Standards. Such fasteners may differ in other dimensions and their use shall be subject to the approval of the designer.

22.3 Detailing of bolted connections

22.3.1 Minimum pitch

The minimum distance between centres of bolt holes shall be 2.7 times the bolt diameter.

22.3.2 Minimum edge distance

The minimum distance from the centre of a bolt hole to an edge shall be as specified in Table 6.

22.3.3 Maximum edge distance

The maximum distance from the centre of any bolt to the nearest edge of parts in contact shall be 12 times the thickness of the outside connected part, but not greater than 150 mm.

22.3.4 Minimum end distance

In the connection of tension members having more than two bolts in a line parallel to the direction of load, the minimum end distance (from the centre of the end fastener to the nearest end of the connected part) shall be governed by the edge distance values specified in Table 6. In members having one or two bolts in the line of load, the end distance shall be not less than 1.5 bolt diameters.

22.3.5 Bolt holes

22.3.5.1

Holes may be punched, sub-punched, sub-drilled and reamed, or drilled, as permitted by Clause 28.4. The nominal diameter of a hole shall be not more than 2 mm greater than the nominal bolt size. This requirement may be waived to permit the use of the following bolt diameters and hole combinations in bearing-type or slip-critical connections:

a) 3/4 in diameter bolt or an M20 bolt in a 22 mm hole;
b) a 7/8 in diameter bolt or an M22 bolt in a 24 mm hole; and
c) a 1 in diameter bolt or an M24 bolt in a 27 mm hole.

Oversized or slotted holes may be used with high-strength bolts 16 mm in diameter and larger when approved by the designer.

22.3.5.2

Joints that use enlarged or slotted holes shall be proportioned in accordance with Clauses 13.11, 13.12, and 23 and meet the following requirements:

a) Oversize holes shall be 4 mm larger than bolts 22 mm and less in diameter, 6 mm larger than bolts 24 mm in diameter, and 8 mm larger than bolts 27 mm and greater in diameter. Oversize holes shall not be used in bearing-type connections but may be used in any or all plies of slip-critical connections. Hardened washers shall be used under heads or nuts adjacent to the plies containing oversize holes.
b) Short slotted holes shall be 2 mm wider than the bolt diameter and have a length that does not exceed the oversize diameter requirements of Item a) by more than 2 mm. Short slotted holes may be used in any or all plies of slip-critical or bearing-type connections and without regard to direction of loading in slip-critical connections, but shall be normal to the direction of the load in bearing-type connections. For pretensioned bolts, hardened washers shall be used under heads or nuts adjacent to the plies containing the slotted holes.
c) Long slotted holes shall be 2 mm wider than the bolt diameter, shall have a length greater than that allowed in Item b) (but not more than 2.5 times the bolt diameter in only one of the connected parts at an individual faying surface of either a slip-critical or bearing-type connection), and may be used in
   i) slip-critical connections without regard to the direction of loading (slip resistance shall be decreased in accordance with Clause 13.12.2.2); and
   ii) bearing-type connections with the long dimension of the slot normal to the direction of loading, provided that structural plate washers or a continuous bar not less than 8 mm in thickness covers long slots that are in the outer plies of joints. The plate washers or bar shall have a size sufficient to completely cover the slot after installation. Plate washers or bars shall not be required for bearing-type connections in double shear.
d) When pretensioned ASTM A490 or ASTM A490M bolts greater than 26 mm in diameter are used in oversize or slotted holes, hardened washers shall be at least 8 mm thick. A 10 mm plate washer covering the hole with a standard hardened washer may be used in lieu of the 8 mm hardened washer.

22.3.5.3

The maximum and minimum edge distance for bolts in slotted or oversize holes (as permitted in Clause 22.3.5.1) shall meet the requirements of Clauses 22.3.2 to 22.3.4, assuming that the fastener can be placed at any extremity of the slot or hole.

23 Installation and inspection of bolted joints

23.1 Connection fit-up

When assembled, all joint surfaces, including those adjacent to bolt heads, nuts, and washers, shall be free of scale (tight mill scale excepted), burrs in excess of 2 mm in height, dirt, and foreign material that could prevent firm contact of the parts. Connections using high-strength bolts shall be in firm contact when assembled and shall not be separated by gaskets or compressible materials.

23.2 Surface conditions for slip-critical connections

The condition of the contact surfaces for slip-critical connections, as specified in Table 3, shall be as follows:

a) For clean mill scale, the surfaces shall be free of oil, paint, lacquer, or any other coating for all areas within the bolt pattern and for a distance beyond the edge of the bolt hole that is the greater of 25 mm or the bolt diameter.
b) For Classes A and B, the blast-cleaning and the coating application shall be the same as those used in the tests to determine the mean slip coefficient.
c) For Class C, hot-dip galvanizing shall be done in accordance with CAN/CSA-G164 and the surface subsequently roughened by hand wire-brushing. Power wire-brushing shall not be used.
d) For all other coatings, the surface preparation and coating application for the joint shall be the same as those used in the tests to determine the mean slip coefficient.

Coated joints shall not be assembled before the coatings have cured for the minimum time used in the tests to determine the mean slip coefficient.

23.3 Minimum bolt length

The length of bolts shall be such that the point of the bolt will be flush with or outside the face of the nut when completely installed.

23.4 Use of washers

23.4.1

ASTM F436 hardened washers shall be used under the turned element

a) as required by Clause 23.4.2;
b) for pretensioned ASTM F1852 and ASTM F2280 bolts; and
c) for bolt arbitration inspection procedures.

23.4.2

When high strength bolts are pretensioned, ASTM F436 hardened washers shall

a) be used to cover oversize or slotted holes (see Clause 22.3.5);
b) be used with ASTM F959 washers, as applicable;
c) be placed under the head and nut when used with steel having a specified minimum yield point of less than 280 MPa and the bolts are either ASTM A490, ASTM A490M, or ASTM F2280; and
d) be not less than 8 mm in thickness, when either ASTM A490, ASTM A490M, or ASTM F2280 bolts greater than 26 mm in diameter are used in oversize and slotted holes.

23.4.3

If necessary, washers may be clipped on one side to a point not closer than 7/8 of the bolt diameter from the centre of the washer hole.

23.4.4

ASTM F436 bevelled washers shall be used to compensate for lack of parallelism where, in the case of ASTM A325, ASTM A325M, and ASTM F1852 bolts, an outer face of bolted parts has more than a 5% slope with respect to a plane normal to the bolt axis. In the case of ASTM A490, ASTM A490M, and ASTM F2280 bolts, bevelled washers shall be used to compensate for any lack of parallelism due to the slope of the outer faces.

23.5 Storage of fastener components for pretensioned bolt assemblies

Fastener components shall

a) be stored in closed containers;
b) be returned to protected storage at the end of the work shift when not incorporated into the work;
c) not have the as-delivered condition altered in any fashion, including cleaning; and
d) not be incorporated into the work if rust or dirt resulting from plant or job site conditions accumulates unless they are cleaned, relubricated, and requalified with a bolt tension calibrator.

ASTM F1852 and ASTM F2280 bolt assemblies shall not be relubricated, except by the manufacturer.

23.6 Snug-tightened bolt assemblies

Snug-tightened bolted assemblies shall have the following two conditions:

a) High-strength fastener assemblies that are not required to be pretensioned shall be installed in properly aligned holes to a snug-tight condition as a minimum (for slotted holes, see Clause 22.3.5.2).
b) Fastener assemblies incorporating ASTM A307 bolts shall only be snug-tightened. Where so specified by the designer, additional security from working loose of ASTM A307 assemblies shall be provided by the use of lock washers, locknuts, jam nuts, thread burring, welding or other methods so approved.

23.7 Pretensioned high-strength bolt assemblies

23.7.1 Installation procedure

Pretensioned bolts shall be installed to at least the minimum bolt tensions specified in Table 7, in accordance with the following procedure:

a) After the holes in a joint are aligned, sufficient bolts shall be placed to secure the member.
b) Bolts shall be placed in the remaining open holes and snug-tightened, with joint assembly progressing systematically from the most rigid part of the joint to its free edges (re-snugging may be necessary in large joints).
c) When all bolts are snug-tight, each bolt in the joint shall be pretensioned, with pretensioning progressing systematically from the most rigid part of the joint to its free edges in a manner that will minimize relaxation of previously pretensioned bolts.

23.7.2 Turn-of-nut method

After the snug-tightening procedure is completed, each bolt in the connection shall be pretensioned additionally by the applicable amount of relative rotation specified in Table 8. During this operation there shall be no rotation of the part not turned by the wrench unless the bolt and nut are match-marked to enable the amount of relative rotation to be determined.

23.7.3 Use of ASTM F959 washers

When ASTM F959 washers are used (also known as direct tension indicator washers), the pretension of the bolt in accordance with Table 7 shall be verified using a tension calibrator. Prior to installation of ASTM F959 bolt assemblies, a sample of not fewer than three complete bolt assemblies of each combination of diameter, length, grade, and lot to be used in the work shall be placed individually in a bolt-tension calibrator at the site of installation to verify that the pretensioning method develops a tension that is equal to or greater than 1.05 times the minimum tensions specified in Table 7. The pre-installation verification procedure shall be performed at the start of the work and whenever the lot of fastener assembly is changed.

23.7.4 Use of ASTM F1852 and ASTM F2280 bolts

Prior to installation of ASTM F1852 and ASTM F2280 bolt assemblies in joints requiring pretension, a sample of not fewer than three complete bolt assemblies of each combination of diameter, length, grade, and lot to be used in the work shall be placed individually in a bolt-tension calibrator at the site of installation to verify that the pretensioning method develops a tension that is equal to or greater than 1.05 times the minimum tensions specified in Table 7. The pre-installation verification procedure shall be performed at the start of the work and whenever the lot of fastener assembly is changed.

During the snug-tightening procedure, care shall be taken to avoid severing the splined ends. Bolts with severed ends shall be replaced. After the snug-tightening procedure is completed, each bolt in the joint shall be pretensioned.

23.8 Inspection procedures

23.8.1

The inspector shall determine that the requirements of Clauses 23.1 to 23.6 are met. Tensioning of bolts shall be observed during their installation to ascertain that the proper procedures are employed. In addition, the following shall apply:

a) for snug-tight connections, the inspection need ensure only that the bolts have been tightened sufficiently to bring the connected elements into firm contact;
b) for bolts pretensioned by the turn-of-nut method, the turned element of all bolts shall be visually examined for evidence that they have been pretensioned;
c) for ASTM F959 washers, the washers shall be inspected to ensure that adequate deformations have been achieved in accordance with the manufacturer's installation procedures; and
d) for ASTM F1852 and ASTM F2280 bolt assemblies, the splined ends shall be inspected for twist-off.

Note: *For pretensioned connections, see Annex I if there is disagreement concerning the results of inspection of bolt-tensioning procedures.*

23.8.2

Bolt tensions exceeding those specified in Table 7 shall not be cause for rejection.

24 Welding

24.1 Arc welding

Arc welding shall be designed in accordance with

a) Clause 13.13 for factored resistance of welds under static loading with matching electrode (see CSA W59 for locations and conditions where non-matching is permissible); and

b) Clause 26 for resistance to fatigue loading, with matching electrode (see CSA W59 for locations and conditions where non-matching is permissible).

For all other aspects of welding, the requirements of CSA W59 shall be followed.

24.2 Resistance welding

The resistance of resistance-welded joints shall be in accordance with CSA W55.3. Quality assurance and weld process control procedures shall be as specified in CSA W55.3.

24.3 Fabricator and erector qualification

Fabricators and erectors responsible for welding structures fabricated or erected under this Standard shall be certified by the Canadian Welding Bureau to the requirements of CSA W47.1 (Division 1 or Division 2), CSA W55.3, or both, as applicable. Part of the work may be sublet to a Division 3 fabricator or erector; however, the Division 1 or Division 2 fabricator or erector shall retain responsibility for the sublet work.

25 Column bases and anchor rods

25.1 Loads

Suitable provision shall be made to transfer factored axial loads, including uplift, shears, and moments, to footings and foundations. Forces present during construction and in the finished structure shall be resisted.

25.2 Minimum number of anchor rods

Columns shall be fitted with at least four anchor rods. When four non-colinear anchor rods for erection safety are not feasible, special precautions shall be taken.

25.3 Resistance

25.3.1 Concrete in compression

The compressive resistance of concrete shall be determined in accordance with Clause 10.8 of CSA A23.3. When compression exists over the entire base plate area, the bearing pressure on the concrete may be assumed to be uniform over an area equal to the width of the base plate multiplied by the length minus $2e$, where e is the eccentricity of the column load. Where eccentricity exists about both column axes, the width of the base plate shall also be reduced by twice the eccentricity in that direction.

25.3.2 Tension

25.3.2.1 Anchor rods

The factored tensile resistance of an anchor rod shall be taken as

$$T_r = \phi_{ar} A_n F_u$$

where

ϕ_{ar} = 0.67

A_n = the tensile area of the rods

= $0.85A_g$

25.3.2.2 Pull-out

The pull-out resistance shall be determined in accordance with CSA A23.3, Annex D. Full anchorage shall be obtained when the factored pull-out resistance of the concrete is equal to or greater than the factored tensile resistance of the rods.

The determination of the pull-out value shall account for single and group anchor behaviour.

25.3.3 Shear

25.3.3.1 Shear transfer mechanisms

Shear resistance may be developed by friction between the base plate and the foundation unit or by bearing of the anchor rods or shear lugs against the concrete. The requirements of CSA A23.3, Clause 11 and Annex D shall be met for

a) anchor rods being against the concrete;
b) loads are transferred by friction;
c) shear lugs bearing against the concrete; and
d) shear acting toward a free edge of concrete.

25.3.3.2 Anchor rods in bearing

The factored bearing resistance of an anchor rod shall be determined by CSA A23.3, Annex D. The thickness of the grout layer under the base plate shall be taken into account, in accordance with CSA A23.3.

25.3.3.3 Anchor rods in shear

The factored shear resistance of an anchor rod shall be taken as

$$V_r = 0.60\phi_{ar}A_{ar}F_u$$

where
A_{ar} = cross-sectional area of the anchor rod based on its nominal diameter

When the rod threads are intercepted by the shear plane, the factored shear resistance shall be taken as $0.70V_r$.

25.3.4 Anchor rods in shear and tension

An anchor rod required to develop resistance to both tension and shear shall be proportioned so that

$$(V_f/V_r)^2 + (T_f/T_r)^2 \leq 1$$

where
V_f = the lesser of the factored shear resistance of the anchor rod or the portion of the total shear per rod resisted by bearing of the anchor rods on the concrete

T_r = the lesser of the factored tension resistance of the anchor rod or the factored pull-out resistance of the concrete

25.3.5 Anchor rods in tension and bending

An anchor rod required to develop resistance to both tension and bending shall be proportioned to meet the requirements of Clause 13.9.1. The tensile and moment resistances, T_r and M_r, shall be based on the properties of the cross-section at the critical section. M_r shall be taken as $\phi_{ar}SF_y$.

25.3.6 Moment on column base
The moment resistance of a column base shall be taken as the couple formed by the tensile resistance determined in accordance with Clause 25.3.2 and by the concrete compressive resistance determined in accordance with Clause 25.3.1.

25.4 Fabrication and erection

25.4.1 Fabrication

25.4.1.1 Base plate holes
Base plate holes may be drilled, machined, or thermally cut. The surfaces of thermally cut holes shall meet the requirements of Clause 28.2.

Holes in base plates for anchor rods shall be of sufficient size to meet or exceed the placement tolerances for anchor rods. The Designer shall provide details of corrective work if base plate holes are to be adjusted to suit as-cast locations of anchor rods.

25.4.1.2 Bases resting on masonry or concrete
The bottom surfaces of bearing plates and column bases that rest on masonry or concrete foundations and are grouted to ensure full bearing need not be planed.

25.4.1.3 Rolled steel bearing plates
Finishing of steel-to-steel contact bearing surfaces shall meet the requirements of Clauses 28.5 and 29.3.9. Plates 55 mm or less thick may be used without machining. Plates more than 55 mm thick may be straightened by pressing or machined at bearing locations.

25.4.2 Erection

25.4.2.1 Setting column bases
Column bases shall be set on level finished floors, pre-grouted levelling plates, levelling nuts, or shim packs that are adequate to transfer the construction loads. Steel shim packs may remain in place unless otherwise specified by the Designer.

25.4.2.2 Tensioning of anchor rods
Nuts on anchor rods need be installed only to a snug-tight condition unless otherwise specified by the designer. If pre-tension is required, the method of tensioning and the pre-tension value shall be defined by the designer.

26 Fatigue

26.1 General
In addition to meeting the fatigue requirements of Clause 26, all members and connections shall meet the requirements for the static load conditions using the factored loads. Specified loads shall be used for all fatigue calculations. This is calculated using ordinary elastic analysis and the principles of mechanics of materials and includes stresses that may result from bending moments due to joint eccentricities. A specified load less than the maximum specified load but acting with a greater number of cycles can govern and therefore shall be considered. Members and connections subjected to fatigue loading shall

be designed, detailed, and fabricated so as to minimize stress concentrations and abrupt changes in cross-section. The life of the structure shall be taken as 50 years, unless otherwise specified by the owner.

26.2 Proportioning

In the absence of more specific requirements by the owner or designer, the requirements of Clause 26 shall be used to proportion members and parts. Fatigue resistance shall be provided only for repetitive loads.

26.3 Live-load-induced fatigue

26.3.1 Calculation of stress range

The controlling stress feature in load-induced fatigue is the range of stress to which the element is subjected. This is calculated using ordinary elastic analysis and the principles of mechanics of materials and includes stresses that may result from the bending moments due to joint eccentricities. More sophisticated analysis shall be required only in cases not covered by Table 9, e.g., major access holes and cut-outs. Stress range is the algebraic difference between the maximum stress and minimum stress at a given location; thus, only live load induces a stress range.

The load-induced fatigue requirements of Clause 26 need be applied only at locations that undergo a net applied tensile stress. Stress ranges that are completely in compression need not be investigated for fatigue.

26.3.2 Design criteria

For load-induced fatigue and constant amplitude fatigue loading, the following design requirement shall apply:

$$F_{sr} \geq f_{sr}$$

where

F_{sr} = fatigue resistance

$$= \left(\frac{\gamma}{nN}\right)^{1/3} \geq F_{srt}$$

$$= \left(\frac{\gamma'}{nN}\right)^{1/5} \leq F_{srt}$$

where

γ and γ' = fatigue life constants (see Clause 26.3.4)

n = number of stress range cycles at given detail for each application of load

N = number of applications of load

F_{srt} = constant amplitude threshold stress range (Clauses 26.3.3 and 26.3.4)

f_{sr} = calculated stress range at the detail due to passage of the fatigue load including stresses due to eccentricities

26.3.3 Cumulative fatigue damage

The total damage that results from variable amplitude fatigue loading shall satisfy

$$\sum\left[\frac{(nN)_i}{N_{fi}}\right] \leq 1.0$$

where

$(nN)_i$ = number of expected stress range cycles at stress range level i, F_{sri}

N_{fi} = number of cycles that would cause failure F_{sri}, obtained from Figure 1 for the appropriate fatigue category. Alternatively, it may be calculated as follows:

$N_{fi} = \gamma F_{sri}^{-3}$ for $F_{sri} \geq F_{srt}$

and

$N_{fi} = \gamma' F_{sri}^{-5}$ for $F_{sri} \leq F_{srt}$

The summation shall include both stress cycles above and below F_{srt}.

The fatigue constant γ' shall be as specified in Table 10.

26.3.4 Fatigue constants and detail categories

The fatigue constants γ, γ', nN', and F_{srt} shall be as specified in Table 10 and shown in Figure 1. The detail categories shall be obtained from Table 9 and are illustrated in Figure 2.

For high-strength bolts, see also Clause 13.12.1.3.

26.3.5 Limited number of cycles

Except for fatigue-sensitive details with high stress ranges (probably with stress reversal), special considerations beyond those specified in Clause 26.1 need not apply in the event that the number of stress range cycles, nN, over the life of the structure, expected to be applied at a given detail, is less than the greater of γ / f_{sr}^3 or 20 000.

26.4 Distortion-induced fatigue

26.4.1

Members and connections shall be detailed to minimize distortion-induced fatigue that can occur in regions of high strain at the interconnection of members undergoing differential displacements. Whenever practicable, all components that make up the cross-section of the primary member shall be fastened to the interconnection member.

26.4.2

Plate girders with $h / w > 3150 / \sqrt{F_y}$ shall not be used under fatigue conditions.

26.5 High-strength bolts

A high-strength bolt subjected to tensile cyclic loading shall be pretensioned to the minimum preload specified in Clause 23.7. Connected parts shall be arranged so that prying forces are minimized. The prying force per bolt shall not exceed 30% of the externally applied load.

The permissible maximum applied nominal axial stress, including amplification by prying under specified loads, based on the nominal area of the bolt, shall not exceed 214 MPa for ASTM A325, ASTM A325M, and ASTM F1852 bolts and 262 MPa for ASTM A490, ASTM A490M, and ASTM F2280 bolts.

The total maximum cyclic service load that may be applied to a bolt is calculated as the product of the permissible maximum nominal stress above and the nominal area of a bolt. Thus calculated, the service load per bolt, including the amplification by prying, shall not exceed this maximum applied service load on a pretensioned bolt.

27 Seismic design

27.1 General

27.1.1 Scope

Clause 27 specifies requirements for the design of members and connections in the seismic-force-resisting system of steel-framed building structures. With the exception of Clause 27.11, Clause 27 applies to buildings for which seismic design loads are based on a ductility-related force modification factor, R_d, greater than 1.5. Clause 27 shall be applied with the requirements of the *NBCC*. Alternatively, the maximum anticipated seismic loads may be determined from non-linear time-history analyses using appropriate structural models and ground motions. Height restrictions shall not apply when the seismic forces are determined from non-linear time-history analyses or to buildings with specified short-period spectral acceleration ratios ($I_E F_a S_a(0.2)$) less than 0.35, unless otherwise specified in Clause 27 or the *NBCC*.

Clause 27 may be applied to structures other than building structures provided that the structure includes a clearly defined seismic-force-resisting system and that a level of safety and seismic performance comparable to that required by Clause 27 for building structures is provided.

27.1.2 Capacity design

Unless otherwise specified in Clause 27, seismic-force-resisting systems shall be designed according to capacity design principles to resist the maximum anticipated seismic loads, but such loads need not exceed the values corresponding to $R_d R_o$ = 1.3.

In capacity design,

a) specific elements or mechanisms are designed and detailed to dissipate energy;
b) all other elements are sufficiently strong for this energy dissipation to be achieved;
c) structural integrity is maintained;
d) elements and connections in the horizontal and vertical load paths are designed to resist the seismic loads;
e) diaphragms and collector elements are capable of transmitting the loads developed at each level to the vertical seismic-force-resisting system; and
f) these loads are transmitted to the foundation.

Connections along the horizontal load path that are designed for forces corresponding to $R_d R_o$ = 1.3 shall have a ductile governing ultimate limit state.

27.1.3 Seismic load path

Any element that significantly affects the load path or the seismic response shall be considered in the analysis and shown on the structural drawings.

27.1.4 Members and connections supporting gravity loads

Structural members and their connections that are not considered to form part of the seismic-force-resisting system shall be capable of supporting gravity loads when subjected to seismically induced deformations.

Splices in gravity columns not part of the seismic-force-resisting system shall have a factored shear resistance in both orthogonal axes equal to the sum of $0.2ZF_y/h_s$ of the columns above and below the splices.

Note: *The gravity loads to be supported are those considered in combination with the earthquake loading.*

27.1.5 Material requirements

27.1.5.1

Steel used in the energy-dissipating elements described in Clauses 27.2 to 27.10 shall comply with Clauses 5.1.3 and 8.3.2 a). F_y shall not exceed 350 MPa unless the suitability of the steel is determined by testing or other rational means. F_y shall not exceed 450 MPa in columns in which the only expected inelastic behaviour is at the column base. Other material may be used if approved by the regulatory authority.

Note: *F_y is the specified minimum yield stress. See Clause 5.1.2.*

27.1.5.2

When the specified short-period spectral acceleration ratio ($I_E F_a S_a(0.2)$) is greater than 0.55, rolled shapes with flanges 40 mm or thicker, or plates and built-up shapes over 51 mm in thickness, used in energy-dissipating elements or welded parts, shall have a minimum average Charpy V-notch impact test value of 27 J at 20 °C, unless it can be demonstrated that tensile stresses, including local effects, are not critical. The impact tests shall be conducted in accordance with CSA G40.21, with the following exceptions:

a) the central longitudinal axis of the test specimens in rolled shapes shall be located as near as practicable to midway between the inner flange surface and the centre of the flange thickness at the intersection with the web mid-thickness; and

b) one impact test sample shall be taken from each 15 tonnes or less of shapes produced from each heat, or from each ingot for shapes rolled from ingots.

27.1.5.3

This Clause applies to welds in primary members and connections where the specified short-period spectral acceleration ratio ($I_E F_a S_a(0.2)$) is greater than 0.35.

All welds shall be made with filler metals that have a minimum average Charpy V-notch impact test value of 27 J at a test temperature equal to or lower than –18 °C as certified in accordance with CSA W48 or a manufacturer's certificate of conformance.

In addition, demand critical welds as designated below shall be made with filler metals that have a minimum average Charpy V-notch impact test value of 54 J at +20 °C, except that where the structure in service is exposed to temperatures lower than +10 °C, the maximum testing temperature shall be 20 °C above the 2.5% January design temperature as defined in Appendix C, Division B of the *NBCC*. Demand critical welds shall include

a) groove welds in column splices;

b) welds at column-to-base plate connections when plastic hinging or net section fracture in tension is expected at the column bases;

c) except when Item e) applies, complete joint penetration groove welds joining beam flanges and beam webs to columns in moment connections for Type D and MD moment-resisting frames;

d) except when Item e) applies, complete joint penetration groove welds joining beam flanges to columns in moment connections for Type LD moment-resisting frames and Type D plate walls;

e) when moment connections are designed in accordance with the CISC *Moment Connections for Seismic Applications*, all demand critical welds designated therein;

f) welds joining link beam flanges and webs to columns in Type D eccentrically braced frames;

g) welds joining webs and flanges in built-up tubular link beams in Type D eccentrically braced frames; and

h) welds joining infill plates to perimeter frame members in Type D plate walls.

The requirements of this Clause may be waived when the specified short-period spectral acceleration ratio ($I_E F_a S_a(0.2)$) is less than or equal to 0.55 and the welds are loaded primarily in shear.

Note: *The maximum testing temperature for demand critical welds in structures exposed to low temperatures is based on a service temperature taken as* 10 °C *above the 2.5% January design temperature as defined in Appendix C, Division B of the* NBCC.

27.1.5.4

When T-joint or corner-joint details susceptible to through-thickness tensile stresses resulting from welding executed under conditions of restraint cannot be avoided, measures shall be taken to minimize the possibility of lamellar tearing in accordance with CSA W59.

27.1.6 Bolted connections

Bolted connections shall

a) have pretensioned high-strength bolts;
b) have surfaces of Class A or better, when designed as bearing-type connections;
c) not be considered to share load with welds;
d) not have long slotted holes;
e) not have short slotted holes unless the load is normal to the slot; and
f) have end distances in the line of seismic force not less than two bolt diameters when the bearing force due to seismic load exceeds 75% of the bearing resistance (see Clause 13.12.1.2).

The requirements of this Clause may be waived when fastener and connection details conform to those of a tested assembly.

27.1.7 Probable yield stress

The probable yield stress shall be taken as $R_y F_y$. The value of R_y shall be taken as 1.1 and the product $R_y F_y$ as not less than 460 MPa for HSS sections or 385 MPa for other sections, unless the probable yield stress, taken as an average yield stress, is obtained in accordance with CSA G40.20.

Width-to-thickness limits of energy-dissipating elements shall be based on F_y, with F_y taken as not less than 300 MPa for angles and 350 MPa for other sections.

27.1.8 Stability effects

27.1.8.1

The effects of notional loads and P-delta effects shall be taken into account when sizing the energy-dissipating elements or mechanisms of the seismic-force-resisting system. Notional loads and P-delta effects shall also be considered when determining the limiting forces corresponding to $R_d R_o$ = 1.3. Notional loads and P-delta effects need not be considered when determining member forces induced by yielding of the energy-dissipating elements or mechanisms of the seismic-force-resisting system.

The notional loads shall be calculated in accordance with Clause 8.4.1.

27.1.8.2

When the provisions of the *User's Guide — NBC 2015: Structural Commentaries (Part 4)* are applied in calculating P-delta effects, the value of U_2 in Clause 8.4.2 may be taken as

$$U_2 = 1 + \left(\frac{\sum C_f R_d \Delta_f}{\sum V_f h} \right)$$

Structural stiffness shall be provided such that U_2 does not exceed 1.4.

27.1.9 Protected zones

Structural and other attachments that could introduce metallurgical notches or stress concentrations shall not be used in areas designated as protected zones unless engineered and forming part of the design system or forming part of a test assembly that satisfies the physical test requirements of Clauses 27.2.5.1,27.7.8.1, and 27.8.6. Discontinuities created by fabrication or erection operations shall be repaired.

Welded shear studs and decking attachments that penetrate the beam flange shall not be placed on the beam flanges within protected zones unless approved by the designer. Arc-spot welds necessary to secure decking to beam flanges may be used.

Protected zones shall be indicated on the structural design documents and shop details (see Clauses 4.2.2,4.3.2, and 4.3.3).

27.2 Type D (ductile) moment-resisting frames, R_d = 5.0, R_o = 1.5

27.2.1 General

27.2.1.1

Ductile moment-resisting frames can develop significant inelastic deformation through plastic hinging in beams a short distance from the face of columns. Plastic hinges in columns shall be used only at the base, except at the top of columns terminating under a beam or in single-storey buildings.

Note: *Plastic deformation in joints is limited by Clause 27.2.4. See Clause 27.11.2 for cantilever column structures.*

27.2.1.2

Energy-dissipating elements shall be proportioned and braced to enable them to undergo large plastic deformations.

27.2.1.3

In Clauses 27.2.2 to 27.2.4, the effects of bearing of slabs on column flanges shall be considered in determining the flexural resistance of, and the loading produced by, composite beams.

27.2.2 Beams

Beams are expected to develop plastic hinges typically at a short distance from the face of columns (see Clause 27.2.5) and shall

a) be Class 1 sections; and
b) be laterally braced in accordance with Clause 13.7 b) unless alternative bracing is demonstrated as satisfactory in accordance with Clause 27.2.5.1. The value of κ shall be based on the bending moment distribution for combined gravity and seismic loads. The bending moments due to seismic load may be taken as varying linearly from a maximum at one end of the beam to zero at the other, unless another value can be justified.

The forces acting on other members and connections due to plastic hinging shall be calculated using $1.1R_y$ times the nominal flexural resistance, ZF_y, except when connections and associated design procedures referenced in Annex J are selected.

Beams in single-storey buildings need not meet the requirements of this Clause if plastic hinges develop near the top of columns instead of in the beams.

27.2.3 Columns

27.2.3.1

Columns shall be Class 1 or 2. When a column is expected to develop plastic hinging, it shall be Class 1 and meet the following requirements:

a) the column shall be laterally braced in accordance with Clause 13.7 b), using $\kappa = 0.0$, unless other values of κ can be justified by analysis;

b) when the specified one-second spectral acceleration ratio ($I_E F_v S_a(1.0)$) is greater than 0.30, the factored axial load shall not exceed $0.30AF_y$ for all seismic load combinations; and

c) the column shall meet the requirements of Clause 27.2.8.

Non-dissipating structural elements adjacent to plastic hinges in columns shall be able to resist forces corresponding to $1.1R_y$ times the nominal flexural resistance of the columns. This nominal flexural resistance shall be taken as $1.18M_{pc}(1 - C_f/R_yC_y)$, but shall not be greater than the nominal plastic moment resistance of the column, M_{pc}, where C_f is as specified in Clause 27.2.3.2.

27.2.3.2

Columns shall resist the gravity loads together with the forces induced by plastic hinging of the beams as projected at the column centrelines. The following shall apply at each beam-to-column intersection:

$$\sum M'_{rc} \geq \sum\left(1.1R_yM_{pb} + V_h\left(x + \frac{d_c}{2}\right)\right)$$

where

$\Sigma M'_{rc}$ = sum of the column factored flexural resistances projected at the intersection of the beam and column centrelines

and

$$M'_{rc} = 1.18\phi M_{pc}\left(1 - \frac{C_f}{\phi C_y}\right) \leq \phi M_{pc}$$

where

M_{pb} = nominal plastic moment resistance of the beam

V_h = shear acting at that beam plastic hinge location due to gravity loads on the beam plus moments equal to $1.1R_yM_{pb}$ at beam hinge locations

x = distance from the centre of a beam plastic hinge to the column face, which shall correspond to that of the assembly used to demonstrate performance in accordance with Clause 27.2.5.1

M_{pc} = nominal plastic moment resistance of the column

C_f = axial force from gravity loads plus the summation of V_h acting at and above the level under consideration

Columns in single-storey buildings need not meet the requirements of this Clause if plastic hinges develop near the top of columns.

27.2.3.3
When the axial force calculated in accordance with Clause 27.2.3.2 is tensile, column splices having partial-joint-penetration groove welds shall

a) be capable of resisting twice the calculated tensile force;
b) have flange connections that are each capable of resisting at least $0.5A_fR_yF_y$, where A_f is the flange area of the smaller column at the splice; and
c) be located at least one-fourth of the clear distance between beams but not less than 1 m from the beam-to-column joint.

27.2.4 Column joint panel zone

27.2.4.1
When plastic hinges form in adjacent beams, the panel zone shall resist forces arising from beam moments at the column faces of

$$\Sigma\left(1.1R_yM_{pb} + V_hx\right)$$

where the summation is for both beams at a joint, and M_{pb}, V_h, and x are as specified in Clause 27.2.3.2.

In single-storey buildings, when plastic hinges form near the top of columns, panel zones shall resist forces arising from moments corresponding to plastic hinge moments of $1.1R_y$ times the nominal flexural resistance of the column.

27.2.4.2
The horizontal shear resistance of the column joint panel zone shall be taken as either

a) $V_r = 0.55\phi d_c w' F_{yc}\left[1 + \frac{3\,b_c t_c^2}{d_c d_b w'}\right] \le 0.66\phi d_c w' F_{yc}$; or
b) $V_r = 0.55\phi d_c w'\, F_{yc}$

where the subscripts b and c denote the beam and the column, respectively, and w' is the thickness of the column web plus the thickness of the doubler plates, when used.

27.2.4.3
The following requirements shall also apply:

a) Where the specified short-period spectral acceleration ratio ($I_EF_aS_a$ (0.2)) is equal to or greater than 0.55, and the joint panel zones are designed in accordance with Clause 27.2.4.2 a), the sum of panel zone depth and width divided by the panel zone thickness shall not exceed 90 and the effects of panel-zone deformations on frame stability shall be accounted for.
b) Joint panel zones designed in accordance with Clause 27.2.4.2 b) shall satisfy the width-to-thickness limit of Clause 13.4.1.1 a) i).
c) Doubler plates shall be groove- or fillet-welded to the column flanges to develop their full shear resistance.
d) When doubler plates are placed against the column web and continuity plates are used, the doubler plates shall be fillet welded to the continuity plates to develop the proportion of the total force transmitted to the doubler plate. When continuity plates are not used, the doubler plates shall extend above and below the level of the beam flanges and be fillet welded across the top and bottom edges to develop the proportion of the total force transmitted to the doubler plate.

e) When doubler plates are placed away from the column web, they shall be placed symmetrically in pairs and welded to continuity plates to develop the proportions of the total force transmitted to the doubler plate.
f) In calculating width-to-thickness ratios, doubler plate thickness may be included with web thickness only when the doubler plate is connected to the column web near the centre of the panel.

27.2.4.4
Other requirements may apply for the beam-to-column connections selected in accordance with Clause 27.2.5.1.

27.2.5 Beam-to-column joints and connections

27.2.5.1
The beam-to-column joint shall maintain a strength at the column face of at least the nominal plastic moment resistance of the beam, M_{pb}, through a minimum interstorey drift angle of 0.04 radians under cyclic loading. When reduced beam sections are used, or when local buckling limits the flexural strength of the beam, the beam need only achieve $0.8M_{pb}$ at the column face when an interstorey drift angle of 0.04 radians is developed under cyclic loading.

Beam-to-column connections shall satisfy the requirements in this Clause by one of the following:
a) use of connections designed and detailed in accordance with the CISC *Moment Connections for Seismic Applications*; or
b) demonstration of the connection performance through at least two physical qualifying cyclic connection tests as described and referenced in Annex J.

27.2.5.2
The factored resistance of the beam web-to-column connection shall equal or exceed the effects of gravity loads combined with shears induced by moments of $1.1R_yZF_y$ acting at plastic hinge locations. Other requirements may apply for the beam-to-column connections selected in accordance with Clause 27.2.5.1.

27.2.5.3
In single-storey buildings, when the column frames into the underside of the beam and plastic hinging is expected near the top of a column, the connection shall meet the requirements of Clause 27.2.5.1.

27.2.6 Bracing
The following bracing requirements shall apply:
a) Beams, columns, and beam-to-column joints shall be braced by members proportioned in accordance with Clause 9.2 where $C_f = 1.1R_yF_y$ times the cross-sectional area in compression. The possibility of complete load reversals shall be considered.
b) When plastic hinges occur in the beam, lateral bracing at the joints shall be provided at least at the level of one beam flange. If bracing is not provided at the level of both beam flanges, the transverse moments produced by the forces that would otherwise be resisted by the lateral bracing shall be included in the seismic load combinations. Attachments in the hinging area shall meet the requirements of Clause 27.2.8.
c) When plastic hinges occur near the top of the column, lateral bracing at the joints shall be provided at the level of both beam flanges.

d) When no lateral support can be provided to the joint at the level considered, the following shall apply:
   i) the column maximum slenderness ratio shall not exceed 60; and
   ii) transverse moments produced by the forces otherwise resisted by the lateral bracing shall be included in the seismic load combinations.

27.2.7 Fasteners

Fasteners connecting the separate elements of built-up flexural members shall have resistance adequate to support forces corresponding to moments of $1.1R_yZF_y$ at the plastic hinge locations.

27.2.8 Protected zones

The regions at each end of the beams subject to inelastic deformations and in columns where inelastic deformations are anticipated shall be designated as protected zones and meet the requirements of Clause 27.1.9.

The protected zone of the beams shall be defined as the area from the face of the column flange to one-half of the beam depth beyond the theoretical hinge point. Abrupt changes in beam flange cross-sections shall be avoided in protected zones, unless specially detailed reduced beam sections are provided that satisfy Clause 27.2.5. Bolt holes in beam webs, when detailed in accordance with the individual connection requirements of this Standard, may be used.

Where the theoretical hinge point falls at the base of the column, the protected zone of the columns shall be defined as the area from the face of the base plate to one-half of the column depth beyond the theoretical hinge point or the column depth, whichever is greater. Where the theoretical hinge point falls within the column below the beam, the protected zone of the columns shall be defined as the area from the underside of the beam to one-half of the column depth beyond the theoretical hinge point or the column depth, whichever is greater.

27.3 Type MD (moderately ductile) moment-resisting frames, $R_d = 3.5$, $R_o = 1.5$

Moderately ductile moment-resisting frames can develop a moderate amount of inelastic deformation through plastic hinging in the beams at a short distance from the face of columns. The requirements of Clause 27.2 shall apply to such frames, except that
a) with respect to Clause 27.2.2,
   i) the beams shall be Class 1 or 2 sections; and
   ii) the bracing shall meet the requirements of Clause 13.7 a);
b) with respect to Clause 27.2.3.1 b), the factored axial load shall not exceed $0.50AF_y$; and
c) with respect to Clause 27.2.5.1, the minimum interstorey drift angle shall be 0.03 radians.

27.4 Type LD (limited-ductility) moment-resisting frames, $R_d = 2.0$, $R_o = 1.3$

27.4.1 General

Limited-ductility moment-resisting frames can develop a limited amount of inelastic deformation through plastic hinging in the beams, columns, or joints. This system may be used in buildings
a) not exceeding 60 m in height where the specified short-period spectral acceleration ratio ($I_EF_aS_a$ (0.2)) is greater than or equal to 0.35 but less than or equal to 0.75; and
b) not exceeding 30 m in height where the specified short-period spectral acceleration ratio ($I_EF_aS_a$ (0.2)) is greater than 0.75 or where the specified one-second spectral acceleration ratio ($I_EF_vS_a$ (1.0)) is greater than 0.30.

27.4.2 Beams and columns

27.4.2.1

Beams shall be Class 1 or 2. Columns shall be Class 1. Except at roof level, beams shall frame into the columns.

27.4.2.2

When the specified short-period spectral acceleration ratio ($I_E F_a S_a$ (0.2)) is greater than 0.55 or the building is greater than 60 m in height, columns shall satisfy the requirements of Clause 27.2.3.2. However, when Clause 27.2.3.2 is applied, the term $1.1 R_y M_{pb}$ may be replaced by $R_y M_{pb}$ and columns may be Class 2. In addition, the beams shall be designed so that for each storey, the storey shear resistance is not less than that of the storey above.

27.4.3 Column joint panel zone

The horizontal shear resistance of the column joint panel zone shall be that specified in Clause 27.2.4.2.

27.4.4 Beam-to-column connections

27.4.4.1

The beam-to-column joints shall meet the requirements of Clause 27.2.5.1, except that the minimum interstorey drift angle shall be 0.02 radians.

Beam-to-column connections shall satisfy the requirements in this Clause by one of the following:
a) use of connections designed and detailed in accordance with Clause 27.4.4.2;
b) use of connections designed and detailed in accordance with the CISC *Moment Connections for Seismic Applications*; or
c) demonstration of the connection performance through at least two physical qualifying cyclic connection tests as described and referenced in Annex J.

27.4.4.2

With respect to Clause 27.4.4.1 a):
a) Columns shall be I-shaped sections.
b) The beam flanges shall be directly welded to the column flanges.
c) Beam-to-column connections shall have a moment resistance equal to $R_y M_{pb}$, except that, when the controlling limit state is ductile, the moment resistance need not exceed the effect of the gravity loads combined with the seismic load multiplied by 2.0.
d) Beam-to-column connections designed for a moment resistance of $R_y M_{pb}$ shall have a welded web connection.
e) Weld backing bars and run-off tabs shall be removed and repaired with reinforcing fillet welds. Top-flange backing bars may remain in place if continuously fillet welded to the column flange on the edge below the complete joint penetration groove weld. Neither partial-joint-penetration groove welds nor fillet welds shall be used to resist tensile forces in the connections.
f) The tensile resistance of the column flange shall be taken as $0.6T_r$, as specified in Clause 21.3.
g) When columns frame under the beams, the roles of beam and column shall be reversed.

Note: *Note: Beam-to-column connections with complete-penetration groove welds made with matching electrodes in accordance with Clause 13.13.3.1 between the beam flanges and the column flanges are considered to have a moment-resistance equal to $R_y M_{pb}$.*

27.4.4.3
Beam-to-column connections shall resist shear forces resulting from the gravity load together with shears corresponding to the moments at the beam ends equal to those specified in Clause 27.4.4.2 c).

27.5 Type MD (moderately ductile) concentrically braced frames, R_d = 3.0, R_o = 1.3

27.5.1 General
Moderately ductile concentrically braced frames can dissipate moderate amounts of energy through yielding of bracing members.

27.5.2 Bracing systems

27.5.2.1 General
Moderately ductile concentrically braced frames include
a) tension-compression bracing systems (see Clause 27.5.2.3);
b) chevron braced systems (see Clause 27.5.2.4);
c) tension-only bracing systems (see Clause 27.5.2.5); and
d) other systems, provided that stable inelastic response can be demonstrated.

Knee bracing and K-bracing, including those systems in which pairs of braces meet a column on one side between floors, are not considered to be moderately ductile concentrically braced frames.

27.5.2.2 Proportioning
At all levels of any planar frame, the diagonal bracing members along any braced column line shall be proportioned in such a way that the ratio of the sum of the horizontal components of the factored tensile brace resistances in opposite directions is between 0.75 and 1.33.

27.5.2.3 Tension-compression bracing
Except where the specified short-period spectral acceleration ratio ($I_E F_a S_a$ (0.2)) is less than 0.35, tension-compression concentric bracing systems shall not exceed 40 m in height. In addition, when the height exceeds 32 m, the factored seismic forces for the ultimate limit states shall be increased by 3% per metre of height above 32 m.

Tension-compression bracing, in which pairs of braces meet a column at one or two points on one side between horizontal diaphragms, may be used provided that the columns meet the requirements of Clause 27.5.6.

27.5.2.4 Chevron bracing
Chevron bracing systems comprise pairs of braces, located either above or below a beam, that meet the beam at a single point within the middle half of the span. Chevron bracing systems shall meet the requirements of Clause 27.5.2.3.

The beams to which the chevron bracing is attached shall
a) be continuous between columns;
b) have both top and bottom flanges laterally braced at the brace connection; and
c) resist bending moments due to gravity loads (assuming no vertical support is provided by the bracing members) in conjunction with bending moments and axial forces induced by forces of T_u and C'_u in the tension and compression bracing members, respectively. In the case of buildings not exceeding four storeys, the tension brace force may be taken as $0.6T_u$, provided that the beam is a

Class 1 section. When braces are connected to the beam from above, the case where the brace compression force is equal to C_u shall also be considered.

The beam-to-column connections shall resist the forces corresponding to the loading described in Item c) for beams. However, when the tension brace force is less than T_u, the connections shall resist the gravity loads combined with forces associated with the attainment of R_y times the nominal flexural resistance of the beam at the brace connection.

The lateral braces at the brace connection shall resist a transverse load of 0.02 times the beam flange yield force.

Note: *See Clause 27.5.3.4 for the probable tensile, compressive, and post-buckling compressive resistances of bracing members, T_u, C_u, and C'_u, respectively.*

27.5.2.5 Tension-only bracing

The braces in tension-only bracing systems are designed to resist, in tension, 100% of the seismic loads and are connected at beam-to-column intersections. In addition, except where the specified short-period spectral acceleration ratio ($I_E F_a S_a(0.2)$) is less than 0.35,

a) the structure shall not exceed 20 m in height and, when the height exceeds 16 m, the factored seismic forces for ultimate limit states shall be increased by 3% per metre of height above 16 m;
b) all columns are continuous and of constant cross-section over the building height; and
c) the column splices are proportioned for the full moment resistance of the cross-section and for a shear force of $2.0ZF_y/h_s$, where Z is the plastic modulus of the column and h_s is the storey height.

Although the braces are proportioned on the basis of tension loading only, this system shall meet the other requirements of Clause 27, including Clauses 27.5.3 to 27.5.5.

27.5.3 Diagonal bracing members

Note: *Where possible, at every storey, the two discontinuous bracing members in every X-bracing bay should be fabricated and installed from the same heat.*

27.5.3.1 Brace slenderness

The slenderness ratio, KL/r, of bracing members shall not exceed 200.

When the specified short-period spectral acceleration ratio ($I_E F_a S_a$ (0.2)) is equal to or greater than 0.75 or the specified 1 s spectral acceleration ratio ($I_E F_v S_a(1.0)$) is equal to or greater than 0.30, the slenderness ratio of HSS bracing members shall not be less than 70.

Note: *The effects of translational and rotational restraints at the brace ends or along the brace length should be accounted for in the calculation of KL.*

27.5.3.2 Width (diameter)-to-thickness ratios

When the specified short-period spectral acceleration ratios ($I_E F_a S_a$ (0.2)) are equal to or greater than 0.35, width-to-thickness ratios shall not exceed the following limits:

a) when $KL/r \leq 100$:
   i) for rectangular and square HSS: $330/\sqrt{F_y}$;
   ii) for circular HSS: $10\,000/F_y$;
   iii) for legs of angles and flanges of channels: $145/\sqrt{F_y}$; and
   iv) for other elements: Class 1;
b) when $KL/r = 200$
   i) for HSS members: Class 1;

ii) for legs of angles: $170 / \sqrt{F_y}$; and
iii) for other elements: Class 2; and

c) when $100 < KL/r < 200$, linear interpolation may be used.

When the specified short-period acceleration ratio ($I_E F_a S_a$ (0.2)) is less than 0.35, HSS shall be Class 1 and all other sections shall be Class 1 or 2. The width-to-thickness ratio for legs of angles shall not exceed $170 / \sqrt{F_y}$.

Back-to-back legs of double-angle bracing members for which buckling out of the plane of symmetry governs, the width-to-thickness ratio shall not exceed $200 / \sqrt{F_y}$ irrespective of the specified short-period acceleration ratio ($I_E F_a S_a$ (2)).

27.5.3.3 Built-up bracing members

For buildings with specified short-period spectral acceleration ratios [$I_E F_a S_a$ (0.2)] equal to or greater than 0.35, the slenderness ratio of the individual parts of built-up bracing members, as defined in Clause 19.2.4,shall not be greater than 0.5 times the governing effective slenderness ratio of the member as a whole. If overall buckling of the brace does not induce shear in the stitch fasteners that connect the separate elements of built-up bracing members, the slenderness ratio of the individual parts shall not exceed 0.75 times the governing effective slenderness ratio of the member as a whole.

If overall buckling of the brace induces shear in the stitch fasteners, the stitch fasteners shall have a resistance adequate to support one-half of the yield load of the larger component being joined, with this force assumed to act at the centroid of the smaller member. Bolted stitch connections shall not be located in the anticipated plastic hinge regions of bracing members.

27.5.3.4 Probable brace resistances

For the purpose of evaluating forces on connections and other members upon yielding and buckling of the bracing members in capacity design, the probable tensile resistance of bracing members, T_u, shall be taken as equal to $A_g R_y F_y$; the probable compressive resistance of bracing members, C_u, shall be taken as equal to the lesser of $A_g R_y F_y$ and $1.2C_r/\phi$, where C_r is computed using $R_y F_y$; and the probable post-buckling compressive resistance of bracing members, C_u', shall be taken as equal to the lesser of $0.2A_g R_y F_y$ and C_r/ϕ, where C_r is computed using $R_y F_y$.

Each of the two loading conditions,

a) the compression acting braces attaining their probable compressive resistance, C_u; and
b) the compression acting braces attaining their probable buckled resistance, C_u', shall be considered as occurring in conjunction with the tension acting braces developing their probable tensile resistance, T_u.

For chevron bracing, when plastic hinging in the beam is permitted by Clause 27.5.2.4 c) or 27.6.2.2, the brace tensile force need not exceed the greater of that corresponding to plastic hinging in the beam and that corresponding to C_u of the compression brace.

When the forces corresponding to $R_d R_o = 1.3$ are computed, the redistribution of forces due to brace buckling shall be considered.

27.5.4 Brace connections

27.5.4.1 Eccentricities

Eccentricities in connections of braces to gusset plates or other supporting elements shall be minimized.

27.5.4.2 Resistance

The factored resistance of brace connections shall equal or exceed both the probable tensile resistance of the bracing members in tension, T_u, and the probable compressive resistance of the bracing members in compression, C_u, specified in Clause 27.5.3.4. For chevron bracing, the brace tension force may be reduced as specified in Clause 27.5.2.4.

The net section fracture resistance of the brace shall be adequate to resist the tension resistance, T_u. The net section factored resistance of the brace may be multiplied by R_y/ϕ_u, where R_y shall not exceed 1.2 for HSS and 1.1 for other shapes. This multiplier shall not be applied to the factored resistance of any cross-section reinforcement.

27.5.4.3 Ductile hinge rotation

Brace members or connections, including gusset plates, shall be detailed to provide ductile rotational behaviour, either in or out of the plane of the frame, depending on the governing effective brace slenderness ratio. When rotation is anticipated in the bracing member, the factored flexural resistance of the connections shall equal or exceed $1.1ZR_yF_y$ of the bracing member and the net section factored bending resistance of an unreinforced brace may be multiplied by R_y/ϕ. This requirement may be satisfied in the absence of axial load.

27.5.5 Columns, beams, and connections other than brace connections

27.5.5.1

The factored resistance of columns, beams, and connections other than brace connections shall equal or exceed the effects of gravity loads and the brace forces corresponding to the brace probable resistances specified in Clause 27.5.3.4. For chevron bracing, the beams shall be designed in accordance with Clause 27.5.2.4 and the brace tension force may be reduced as specified in Clause 27.5.3.4.

27.5.5.2

Columns in multi-storey buildings using the systems specified in Items a) to c) of Clause 27.5.2.1 shall be continuous and of constant cross-section over a minimum of two storeys, except as required by Clause 27.5.2.5.

Columns outside of the braced bays shall meet the requirements of Class 1, 2, or 3 flexural members.

Columns in braced bays shall meet the requirements of Class 1 or 2 beam-columns. Column resistances in the braced bays shall satisfy the requirements of Clause 13.8 including an additional bending moment in the direction of the braced bay of $0.2ZF_y$ in combination with the computed bending moments and axial loads. Splices in columns in braced bays shall be designed to provide the required axial, shear, and flexural resistances including the effects of the additional bending moments in the direction of the braced bays of $0.2ZF_y$ acting either in the same or the opposite directions at the column ends.

27.5.5.3

Partial-joint-penetration groove weld splices in columns subject to tension shall meet the requirements of Items a) and b) of Clause 27.2.3.3.

27.5.6 Columns with braces intersecting between horizontal diaphragms

27.5.6.1

Columns with braces intersecting at one or two points between horizontal diaphragms may be used provided that they also satisfy the requirements of this Clause.

27.5.6.2

Columns shall resist the simultaneous effects of
a) the gravity loads;
b) the axial loads, shear forces, and bending moments induced by yielding and buckling of the bracing members at the design storey drift as obtained from non-linear incremental analysis, assuming that yielding develops in the tension-acting bracing members located at any one level along the height of the storey; and
c) an out-of-plane transverse load at each brace-to-column connection equal to 2% of the factored axial compression load in the columns below the connection.

27.5.6.3

Horizontal struts shall be provided between columns at the brace-to-column connection levels in the plane of the bracing bents for transferring loads between tension-acting braces along the height of the storey assuming that the compression-acting braces attain their probable post-buckling resistance.

27.5.7 Protected zones

The protected zone of bracing members shall
a) be designated to include the full brace length;
b) be designated to include elements that connect braces to beams and columns; and
c) meet the requirements of Clause 27.1.9.

Splices shall not be used in bracing members.

27.6 Type LD (limited-ductility) concentrically braced frames, R_d = 2.0, R_o = 1.3

27.6.1 General

Concentrically braced frames of limited ductility can dissipate limited amounts of energy through yielding of bracing members. The requirements of Clause 27.5 shall be met, except as modified by Clauses 27.6.2 to 27.6.6.

27.6.2 Bracing systems

27.6.2.1 Tension-compression bracing

Except where the specified short-period spectral acceleration ratio ($I_E F_a S_a(0.2)$) is less than 0.35, tension-compression concentric bracing systems shall not exceed 60 m in height. In addition, when the height exceeds 48 m, the factored seismic forces shall be increased by 2% per metre of height above 48 m.

Tension-compression bracing, in which pairs of braces meet a column on one side between floors, may be used in limited-ductility concentrically braced frames provided that the columns meet the requirements of Clause 27.6.6.

27.6.2.2 Chevron bracing

Except where the specified short-period spectral acceleration ratio ($I_E F_a S_a$ (0.2)) is less than 0.35, tension-compression concentric bracing systems shall not exceed 60 m in height. In addition, when the height exceeds 48 m, the factored seismic forces for ultimate limit states shall be increased by 2% per metre of height above 48 m.

Structures of 20 m or less in height need not meet the requirements of Clause 27.5.2.4 c) provided that the braces and beam-to-column connections are proportioned to resist the forces that develop when buckling of the compression brace occurs and provided that when the braces are connected to the beam from below, the beam is a Class 1 section and has adequate nominal resistance to support the tributary gravity loads assuming no vertical support is provided by the bracing members.

27.6.2.3 Tension-only bracing

Except where the specified short-period spectral acceleration ratio ($I_E F_a S_a$(0.2)) is less than 0.35, tension-only systems shall

a) not exceed 40 m in height and, when the height exceeds 32 m, the factored seismic forces for ultimate limit states shall be increased by 3% per metre of height above 32 m; and
b) in multi-storey structures, have all columns fully continuous and of constant cross-section over a minimum of two storeys.

27.6.3 Diagonal bracing members

27.6.3.1

In single- and two-storey structures, the slenderness ratio of bracing members connected and designed in accordance with Clause 27.5.2.5 shall not exceed 300.

27.6.3.2

The requirements of Clause 27.5.3.2 may be modified as follows:

a) when the brace slenderness ratio exceeds 200 (as permitted by Clause 27.6.3.1), the width-to-thickness limits of Clause 27.5.3.2 need not apply; and
b) for buildings less than 40 m in height and with specified short-period spectral acceleration ratios ($I_E F_a S_a$ (0.2)) less than 0.45, braces need not be more compact than Class 2. The width-to-thickness ratio of the legs of angles shall not exceed $170 / \sqrt{F_y}$.

27.6.4 Bracing connections

The requirements of Clause 27.5.4.3 shall not apply to buildings with specified short-period spectral acceleration ratios ($I_E F_a S_a$(0.2)) less than 0.55 if the brace slenderness ratio is greater than 100.

27.6.5 Columns, beams, and other connections

For buildings with specified one-second spectral acceleration ratios ($I_E F_a S_a$(1.0)) not greater than 0.30, the design forces for column splices in Clause 27.1.4 need not be taken into account.

27.6.6 Columns with braces intersecting between horizontal diaphragms

Columns with braces intersecting at 4 points or less between horizontal diaphragms may be used provided that they meet the requirements of Clause 27.5.6.

27.7 Type D (ductile) eccentrically braced frames, R_d = 4.0, R_o = 1.5

27.7.1 General

Ductile eccentrically braced frames can dissipate energy by yielding of links.

27.7.2 Link beam

27.7.2.1

The link beam shall contain a segment (the link) designed to yield, either in flexure or in shear, prior to yielding of other parts of the eccentrically braced frame.

27.7.2.2

The link beam shall be either

a) a segment of the beam, for beams with an I-section or a built-up tubular rectangular cross-section; or

b) a modular link distinct from the rest of the beam. A modular link shall be either
   i) an end-plate connected link fabricated from a I-shaped section connected to the beam with unstiffened end-plate moment connections; or
   ii) a web connected link consisting of a built-up cross-section made of two C-sections connected back-to-back to the beam web, where the C-sections are channels or wide-flange cross-sections with the flanges cut flush with the web on one side.

27.7.2.3

A link shall be provided at least at one end of each brace. A link shall not be required in roof beams of frames over five storeys in height.

27.7.2.4

Link beams shall be Class 1 and designed for the coexisting shears, bending moments, and axial forces. Link beams may have Class 2 flanges and Class 1 webs when $e \le 1.6M_p/V_p$, where e is the length of the link and $V_p = 0.55wdF_y$, for links with wide-flange cross-sections, or $0.55(2w)dF_y$, for links with built-up tubular cross-sections.

27.7.2.5

The web or webs of the link shall be of uniform depth and have no penetrations, splices, attachments, reinforcement, or doubler plates, other than the stiffeners required by Clause 27.7.6.

For links with built-up tubular rectangular cross-sections, complete-joint-penetration groove welds shall be used to connect the webs to the flanges. Inaccessible backing bars need not be removed in these joints.

27.7.2.6

Flanges of built-up tubular links shall satisfy $b / t \le 285 / \sqrt{F_y}$, where b is the clear flange width. Webs shall satisfy $h / w \le 750 / \sqrt{F_y}$. The moment of inertia of built-up tubular links associated to horizontal, out-of-plane bending shall not be less than 0.67 times the link moment of inertia associated to bending in the vertical plane.

27.7.2.7

For web connected modular links, the flanges of the two C-sections shall be interconnected at both flange levels such that the clear longitudinal spacing between interconnections does not exceed 2.0 times the width of the flange of the individual C-sections.

When plates are used to reinforce the flanges of the C-sections in web connected modular links,

a) the flange reinforcement plates shall be continuously welded along their two longitudinal edges over the full length of the C-sections; and

b) the reinforced flanges shall satisfy Class 1 limit for flanges of I-sections in Table 2, where b_{el} is taken as the average of the C-section flange width and the flange reinforcement plate width and t is taken as the thickness of an equivalent flange having a moment of inertia for bending in the plane of the frame equal to that of the reinforced flange.

27.7.3 Link resistance

27.7.3.1 Factored link resistance

The factored shear resistance of the link shall be taken as the lesser of

$\phi V_p'$ and $2\phi M_p'/e$

where

$$V_p' = V_p\sqrt{1-\left(\frac{P_f}{AF_y}\right)^2}$$

where

V_p = $0.55wdF_y$ for links with wide-flange cross-sections

= $0.55(2w)dF_y$ for links with built-up tubular cross-sections and modular links with back-to-back C-sections

P_f = axial force in the link

= C_f or T_f

A = gross area of the link beam

$$M_p' = 1.18M_p\left(1-\frac{P_f}{AF_y}\right) \le M_p$$

e = length of the link (see Clause 27.7.4)

27.7.3.2 Probable link resistance

The nominal shear resistance of the link shall be taken equal to the lesser of V_p' and $2M_p'/e$, as defined in Clause 27.7.3.1, except that when P_f is equal to T_f, V_p' is given by

$$V_p' = V_p\sqrt{1+\left(\frac{P_f}{AF_y}\right)^2}$$

The probable shear resistance of the link shall be taken equal to 1.3 R_y times the nominal link resistance except for links with built-up tubular cross-sections for which the probable shear resistance of the link shall be taken equal to 1.45 R_y times the nominal link resistance.

27.7.4 Link length

27.7.4.1

For end-plate connected modular links, the length of the link e shall be taken as the distance between the end plates. For web connected modular links, the length of the link e shall be taken as the distance between the innermost rows of bolts or vertical welds of the web connections.

For links that consist of a segment of the beams, the length of the link e shall be taken as the distance between the brace-to-beam intersecting points. When a link is directly connected to a column, the link length is measured from the column face or from the link-to-column connection reinforcement.

27.7.4.2

The link length shall be not less than the depth of the link beam. When $P_f/(AF_y) > 0.15$, the link length shall be as follows:

a) when $\dfrac{A_w}{A} \geq 0.3\dfrac{V_f}{P_f}$:

$$e \leq \left[1.15 - 0.5\frac{P_f}{V_f}\frac{A_w}{A}\right]\left(\frac{1.6M_p}{V_p}\right)$$

b) when $\dfrac{A_w}{A} < 0.3\dfrac{V_f}{P_f}$:

$$e \leq \frac{1.6M_p}{V_p}$$

where

A_w = area of web

= $(d - 2t)w$ for links with wide-flange cross-sections

= $(d–2t)(2w)$ for links with tubular cross-sections and modular links with back-to-back C-sections

27.7.4.3

The length of modular links shall also be as follows:

$$e \leq \frac{1.6M_p}{V_p}$$

27.7.5 Inelastic link rotation

The inelastic component of the rotation of the link segment relative to the rest of the beam, the inelastic link rotation, taken as the rotation associated to an inelastic drift equal to three times the elastic drift determined under factored seismic loading, Δf, shall not exceed the following limits:

a) when $e \leq 1.6M_p/V_p$: 0.08 radians;

b) when $e \geq 2.6M_p/V_p$: 0.02 radians; and

c) when $1.6M_p/V_p < e < 2.6M_p/V_p$, linear interpolation may be used.

27.7.6 Link stiffeners

27.7.6.1 Links with wide-flange cross-sections

27.7.6.1.1
Full-depth web stiffeners shall be provided on both sides of the beam web at the ends of the link. The stiffeners shall have a combined width of not less than $b - 2w$ and a thickness of not less than $0.75w$ or 10 mm, whichever is larger.

27.7.6.1.2
Intermediate link web stiffeners shall be full depth and when
a) $e \leq 1.6M_p/V_p$, spaced at intervals not exceeding $(30w - 0.2d)$ when the inelastic link rotation is 0.08 radians or $(52w - 0.2d)$ when the inelastic link rotation is 0.02 radians or less (for intermediate inelastic link rotations, spacing shall be determined by linear interpolation);
b) $2.6M_p/V_p < e < 5M_p/V_p$, placed at a distance of $1.5b$ from each end of the link;
c) $1.6M_p/V_p < e < 2.6M_p/V_p$, provided as in Items a) and b); and
d) $e \geq 5M_p/V_p$, are not required.

27.7.6.1.3
Intermediate web stiffeners shall be required on only one side of the web for link beams less than 650 mm in depth and on both sides of the web for beams 650 mm or greater in depth. One-sided stiffeners shall have a thickness of not less than w or 10 mm, whichever is larger, and a width of not less than $0.5(b - 2w)$.

27.7.6.1.4
Fillet welds connecting stiffeners to the beam web shall be continuous and develop a stiffener force of A_sF_y. The welds shall be terminated a distance of five times the link web thickness from the transition radius between the web and the flanges of the link.

Fillet welds connecting intermediate stiffeners to the flanges shall develop a force of $0.50A_sF_y$. Welds connecting the stiffeners at the link ends to the flanges shall develop a force of A_sF_y.

27.7.6.2 Links with built-up tubular cross-sections

27.7.6.2.1
Full-depth web stiffeners shall be provided on one side of each link web at the diagonal brace connection. These stiffeners shall have a combined width not less than $(b - 2w)$ and a thickness not less than $0.75w$ or 13 mm, whichever is larger.

27.7.6.2.2
Intermediate link web stiffeners shall be full depth and when
a) $e \leq 1.6M_p/V_p$ and $0.64\sqrt{E / F_y} \leq h / w \leq 1.67\sqrt{E / F_y}$ spaced at intervals not exceeding $20w-(d-2t)/8$, on one side of each web; and
b) $h/w \leq h / w \leq 0.64\sqrt{E / F_y}$, are not required.

27.7.6.2.3
Fillet welds connecting stiffeners to the beam web shall be continuous and develop a stiffener force of A_sF_y.

27.7.6.3 Modular links

27.7.6.3.1

Link stiffeners of modular links shall be designed and detailed in accordance with Clause 27.7.6.1, except that

a) For end-plate modular links, the end-plates shall be considered as end stiffeners.

b) For web connected modular links, end stiffeners shall be provided on the exterior side of each C-section and have a width of not less than $0.5(b - 2w)$ and a thickness of not less than $0.75w$ or 10 mm, whichever is larger. End stiffeners shall be located at the location of the inner vertical weld for welded web connections and at a distance of 1.5 d inside of the innermost row of bolts for bolted web connections.

c) For web connected modular links, intermediate web stiffeners shall be provided on the exterior side of each C-section and shall have a width of not less than $0.5(b - 2w)$ and a thickness of not less than $0.75w$ or 10 mm, whichever is larger.

27.7.7 Lateral support for link

Except for links with built-up tubular rectangular cross-sections for which lateral bracing is not required, lateral support shall be provided to both top and bottom flanges at the ends of a link. These lateral supports shall have factored resistance equal to at least $0.06btR_yF_y$.

27.7.8 Link beam-to-column connection

27.7.8.1

When a link is directly connected to a column, the link-to-column connection shall be demonstrated by physical tests as being capable of undergoing cyclic inelastic rotation equal to at least 1.2 times the inelastic component of the rotation as specified in Clause 27.7.5.

Note: *Physical testing procedures to be used to demonstrate the required behaviour are referenced in Annex J.*

27.7.8.2

The demonstration of performance required by Clause 27.7.8.1 may be waived when

a) a link is separated from a column by a short distance in which the beam is reinforced to ensure elastic behaviour of the connection and the beam within this length remains elastic under the forces corresponding to the probable resistance of the link (see Clause 27.7.3.2);

b) the link length does not exceed $1.6M_p/V_p$; and

c) full-depth web stiffeners are provided at the end of the reinforced section.

27.7.8.3

Except for connections designed in accordance with Clauses 27.7.8.1 and 27.7.8.2, link beam-to-column connections may be designed for shear and torsion only. The factored torsional resistance shall equal or exceed $0.02btdF_y$.

27.7.9 Beam outside the link

27.7.9.1

The beam outside the link shall be Class 1 or 2.

27.7.9.2
The beam outside the link shall resist forces corresponding to the probable resistance of the link (see Clause 27.7.3.2). When subject to these forces, the beam resistance may be taken as the factored resistance multiplied by R_y/ϕ when the link and the beam outside the link are part of the same beam piece.

27.7.9.3
The beam outside of the link shall be provided with sufficient lateral support to maintain stability of the beam under the forces defined in Clause 27.7.9.2. If yielding is anticipated at the link end of this outer beam segment, bracing shall be provided in accordance with Clause 13.7 a). Lateral bracing shall be provided to both top and bottom flanges and have factored resistances at least equal to $0.02btR_yF_y$.

27.7.9.4
When welded shear studs are used to transfer horizontal seismic loads from a concrete slab to the beam, shear studs shall not be placed within a distance from the link end equal to
a) four times the overall slab deck thickness for solid slabs or for ribbed slabs with ribs parallel to the beam; or
b) two times the spacing of the ribs for ribbed slabs with ribs perpendicular to the beam.

27.7.10 Modular link-to-beam connections

27.7.10.1
Modular link-to-beam connection shall be demonstrated by physical tests as being capable of undergoing cyclic inelastic rotation equal to at least 1.2 times the inelastic rotation specified in Clause 27.7.5.

Note: *Physical testing procedures to be used to demonstrate the required behaviour are referenced in Annex J.*

27.7.10.2
Modular link-to-beam connections designed to resist forces corresponding to the probable resistance of the link (see Clause 27.7.3.2) may be considered as achieving the requirements of Clause 27.7.10.1 provided that they satisfy Clause 27.7.10.3 or 27.7.10.4, as applicable.

27.7.10.3 End-plate connected modular links
End connections shall be bolted unstiffened end-plate moment connections designed and detailed in accordance with the CISC *Moment Connections for Seismic Applications*. The depth limits for the beam in this publication do not apply. Effects of axial forces shall be included in the design procedure.

27.7.10.4 Web connected modular links
Webs of the back-to-back C-sections shall be connected to the web of the beam by means of either welded or bolted connections. The connections shall be designed in accordance with established design procedures for eccentrically loaded connections that account for the load-deformation response of the welds or bolts, as applicable, and the eccentricity of the load with respect to the instantaneous centre of rotation. When doubler plates are used for the webs of the C-sections, the doubler plates shall not extend into the link length.

27.7.11 Diagonal braces

27.7.11.1
Diagonal brace sections shall be Class 1 or 2.

27.7.11.2
Each diagonal brace and its end connections shall have a factored resistance to support axial force and moment produced by the link developing its probable resistance (see Clause 27.7.3.2).

27.7.12 Brace-to-beam connection
No part of the brace-to-beam connection shall extend into the link. The intersection of the brace and beam centrelines shall be at the end of or within the link. If the brace is designed to resist a portion of the link end moment, full end restraint shall be provided. The beam shall not be spliced within or adjacent to the connection between beam and brace.

27.7.13 Columns

27.7.13.1
Column sections shall be Class 1 or 2.

27.7.13.2
Columns shall be designed to resist the cumulative effect of yielding links together with the gravity loads. The link forces shall be taken as the probable resistances of the links except in storeys below the top two storeys, where the link forces may be taken as 0.90 times the probable resistance of the links.

Column sections in braced bays shall be Class 1 or 2. Column resistances in the braced bays shall satisfy the requirements of Clause 13.8 assuming an additional bending moment in the direction of the braced bay of $0.2ZF_y$ in combination with the computed bending moments and axial loads. In the top two storeys, the additional bending moment shall be taken as equal to $0.4ZF_y$.

27.7.13.3
Splices in columns in braced bays shall be designed to provide the required axial, shear, and flexural resistances including the effects of the additional bending moments in the direction of the braced bays of $0.2ZF_y$ acting either in the same or the opposite directions at the column ends.

Splices that incorporate partial-joint-penetration groove welds shall be located at least one-fourth of the clear distance between beams but not less than 1 m from the beam-to-column joints. When tension occurs in columns due to the link-induced forces, column splices having partial-joint-penetration groove welds shall be designed in accordance with Items a) and b) of Clause 27.2.3.3.

27.7.14 Protected zone
Link beams shall be designated as a protected zone. The protected zone shall extend to one-half of the depth of the beam beyond the ends of the link beams. Welding on link beams may be used for attachment of link stiffeners. The protected zone shall meet the requirements of Clause 27.1.9.

27.8 Type D (ductile) buckling restrained braced frames, R_d = 4.0, R_o = 1.2

27.8.1 General
Ductile buckling restrained braced frames can develop significant inelastic deformation through axial yielding in tension and compression of the core of the buckling restrained bracing members.

27.8.2 Bracing systems

Knee bracing and K-bracing, including systems in which pairs of braces meet a column on one side between floors, shall not be considered to be buckling-restrained braced frames.

Except where the specified short-period spectral acceleration ratio ($I_E F_a S_a(0.2)$) is less than 0.35, buckling restrained braced frames shall not exceed 40 m in height unless stable inelastic response can be demonstrated.

27.8.3 Bracing members

27.8.3.1

The braces shall consist of a structural steel core and a system that restrains the steel core from buckling. The steel core shall be designed to resist the entire axial force in the brace. The factored axial tensile and compression resistances (T_r and C_r, respectively) of the steel core to be used for design of the core shall be taken as follows:

$$T_r = C_r = \phi A_{sc} F_{ysc}$$

where

A_{sc} = cross-sectional area of the yielding segment of the steel core

F_{ysc} = specified minimum yield strength of the steel core, or actual yield strength of the steel core determined from the average of 2 coupon tests. The coupons shall be taken from the actual plate that the steel core is fabricated from. The long axis of the coupons shall be parallel to the long axis of the core. The coupons shall be tested in accordance with CSA G40.20

27.8.3.2

Splices shall not be used in the steel core. Plates used in the steel core that are 50 mm thick or greater shall satisfy the minimum notch toughness requirements of Clause 27.1.5.

27.8.3.3

The buckling restraining system shall be able to resist, without buckling, the forces and deformations that will develop in the brace at deformations corresponding to 2.0 times the seismic design storey drift.

27.8.3.4

The probable tensile, T_{ysc}, and compressive, C_{ysc}, resistances of the bracing members, including strain hardening, friction, and other effects, shall be taken as follows:

$$T_{ysc} = \omega A_{sc} R_y F_{ysc}$$

$$C_{ysc} = \beta \omega A_{sc} R_y F_{ysc}$$

where

ω = a strain hardening adjustment factor obtained by dividing the maximum tension force developed in the buckling restrained brace in the qualification testing specified in Clause 27.8.6, up to a deformation corresponding to 2.0 times the seismic design storey drift, by $A_{sc} R_y F_{ysc}$

β = a friction adjustment factor obtained by dividing the maximum compression force developed in the buckling restrained brace in the qualification testing specified in Clause 27.8.6, up to a deformation corresponding to 2.0 times the seismic design storey drift, by $\omega A_{sc} R_y F_{ysc}$

R_y may be taken as equal to 1.0 if F_{ysc} is determined from a coupon test as part of the qualification testing specified in Clause 27.8.6.

27.8.4 Brace connections

The factored resistance of brace connections shall equal or exceed the probable tensile and compressive resistances of the bracing members.

The design of connections shall include consideration of local and overall buckling and shall be consistent with the bracing forces and details considered in the qualification testing required by Clause 27.8.6.

27.8.5 Beams, columns, and connections other than brace connections

27.8.5.1

The factored resistance of beams, columns, and connections other than brace connections shall equal or exceed the effect of gravity forces and the brace connection forces specified in Clause 27.8.4, assuming the redistribution of loads when the bracing members develop their probable tensile and compressive resistances.

27.8.5.2

Columns in multi-storey buildings shall be continuous and of constant cross-section over a minimum of two storeys.

Column sections outside of the braced bays shall be Class 1, 2, or 3.

Column sections in braced bays shall be Class 1 or 2. Column resistances in the braced bays shall satisfy the requirements of Clause 13.8 assuming an additional bending moment in the direction of the braced bay of $0.2ZF_y$ in combination with the computed bending moments and axial loads. Splices in columns in braced bays shall be designed to provide the required axial, shear, and flexural resistances including the effects of the additional bending moments in the direction of the braced bays of $0.2ZF_y$ acting either in the same or the opposite directions at the column ends.

27.8.5.3

Partial-joint-penetration groove weld splices in columns subject to tension shall meet the requirements of Items a) and b) of Clause 27.2.3.3.

27.8.6 Testing

Individual buckling restrained brace members and buckling restrained braced frames shall be able to develop their resistance without buckling and with positive strain hardening up to deformations corresponding to 2.0 times the seismic design storey drift and shall exhibit values of ω and β greater than 1.0. Satisfaction of these requirements shall be demonstrated by physical testing as described in Annex J. Qualifying test results shall consist of at least two successful cyclic tests, one a test of a brace subassemblage (including brace connection rotational demands at the specified performance) and the other a uniaxial or subassembly test. Both requirements may be based on

a) tests reported in research or documented tests performed for other projects; or
b) tests conducted specifically for the project.

27.8.7 Protected zone

The steel core of bracing members and the elements that connect the steel core to beams and columns shall be designated as protected zones and shall meet the requirements of Clause 27.1.9.

27.9 Type D (ductile) plate walls, R_d = 5.0, R_o = 1.6

27.9.1 General

Ductile plate walls are composed of infill plates framed by rigidly connected columns and beams. They can develop significant inelastic deformation by the yielding of the infill plates and plastic hinging in beams a short distance from the face of columns. Plastic hinges in columns shall be allowed only at the base and shear yielding in columns shall be prevented.

The requirements of Clause 20 shall apply unless otherwise specified by Clause 27.9.

27.9.2 Infill plates

27.9.2.1 Shear resistance

The infill plate shall be designed to resist 100% of the applied factored storey shear force. The factored shear resistance of infill plates shall be taken as

$$V_r = 0.4\phi F_y w L \sin 2\alpha$$

27.9.2.2 Probable yield force

The forces acting on other members and connections due to yielding of the infill plates shall be calculated as R_y times the tension yield resistance of the infill plates, but these forces need not exceed the value corresponding to $R_d R_o$ = 1.3.

27.9.2.3 Perforated infill plates

Unreinforced circular perforations may be located in infill plates provided that

a) the perforations are of equal diameter, D, and are regularly spaced vertically and horizontally over the entire area of the infill plates to form a regular grid of staggered holes to allow development of continuous diagonal tension fields at 45°;
b) the shortest centre-to-centre distance between the perforations, S_{diag}, is such that $D/S_{diag} \le 0.6$;
c) the distance between the first holes and infill plate connections to the surrounding beams and columns is at least D, but does not exceed $(D + 0.7S_{diag})$; and
d) a minimum of four horizontal and four vertical lines of holes is used.

The factored shear resistance of infill plates with circular perforations shall be taken as

$$V_r = 0.4(1 - 0.7D/S_{diag})\phi F_y w L_i$$

27.9.2.4 Infill plates with corner cut-outs

Quarter-circular cut-outs may be located at the upper corners of the infill plates if

a) the infill plates are connected to a reinforcement arching plate that follows the edge of the cut-outs and are designed to allow development of the full strength of the solid infill plate;
b) the radius of the corner cut-outs is less than one-third of the infill plate clear height; and
c) beams and columns are designed to resist the compression or tension axial forces acting at the end of the arching reinforcement.

27.9.3 Beams

27.9.3.1

Beams shall be Class 1 sections braced in accordance with Clause 13.7 b).

27.9.3.2
Beams at every storey shall have sufficient flexural resistance such that at least 25% of the applied factored storey shear force is resisted by beams and columns forming a moment-resisting frame. Axial loads in beams and gravity load effects on beams need not be considered in calculating this resistance.

27.9.3.3
Beam resistances shall meet the requirements of Clause 13.8, considering the axial loads and bending moments induced by the gravity and lateral loads and the tension force in the infill plate determined in accordance with Clause 27.9.2.2. The effects of the tension force in the infill plate acting on the beams and the columns shall be considered in the calculation of the beam axial loads.

27.9.4 Columns

27.9.4.1
Columns shall be Class 1 sections braced in accordance with Clause 13.7 b).

27.9.4.2
Columns shall resist the effects of gravity loads together with the axial loads, shear forces, and bending moments due to the tension forces in the infill plates as determined in accordance with Clause 27.9.2.2, as well as the forces induced by the beams as determined in accordance with Clause 27.9.7.2.

27.9.4.3
Column splices shall develop the full flexural resistance of the smaller section at the splice, together with the shear force consistent with plastic hinging at column ends, assuming double curvature. Splices shall be located as close as practicable to one-fourth of the storey height above the floor.

27.9.4.4
The columns shall be stiffened so that plastic hinging forms in the columns above the base plate or foundation beam.

27.9.5 Minimum stiffness for beams and columns
Beams and columns shall have sufficient flexural stiffness so that the entire infill plate is yielded at the design storey drift.

The requirement of this Clause may be satisfied by applying Clauses 20.5.1 and 20.5.2.

27.9.6 Column joint panel zones
The horizontal shear resistance of the column joint panel zone shall meet the requirements of Clauses 27.2.4.2 and 27.2.4.3.

27.9.7 Beam-to-column joints and connections

27.9.7.1
Beam-to-column joints and connections shall meet the requirements of Clause 27.4.4, except that the moment resistance in Clause 27.4.4.2 c) shall be taken equal to $1.1R_yM_{pb}$.

27.9.7.2
The factored resistance of the beam web-to-column connection shall equal or exceed the effects of gravity loads and tension forces in the infill plates, as determined in accordance with Clause 27.9.2.2, acting above and below the beams, combined with shears induced by moments of $1.1R_yM_{pb}$ acting at plastic hinge locations. The moments acting in the beam plastic hinges may be taken as $1.18(1.1R_yM_{pb})(1–C_f/\phi C_y)$, where C_f is the beam axial load due to the tension forces in the infill plates and C_y is the axial yield resistance of the beam.

27.9.8 Protected zones
Infill plates, the region at each end of the beams subject to inelastic straining, and column bases where inelastic deformations are anticipated shall be designated as protected zones and shall meet the requirements of Clause 27.1.9. The protected zone of beams shall be defined as the area from the face of the column flange to one-half of the beam depth beyond the theoretical hinge point. Bolt holes in beam webs, when detailed in accordance with the individual connection requirements of this Standard, may be used.

27.10 Type LD (limited-ductility) plate walls, R_d = 2.0, R_o = 1.5

27.10.1 General
Limited-ductility plate walls are composed of infill plates framed by columns and beams that may be connected rigidly or by simple connections. They can develop limited inelastic deformation by the yielding of the infill plates and plastic hinging in the beams, columns, or joints. Except where the specified short-period spectral acceleration ratio ($I_EF_aS_a$ (0.2)) is less than 0.35, the height of the structure shall be limited to 60 m

The requirements of Clause 20 and Clause 27.9.8 apply unless otherwise specified by Clause 27.10.

27.10.2 Infill plates

27.10.2.1
The factored shear resistance of infill plates shall be determined in accordance with Clause 27.9.2.1 and the forces acting on other members and connections due to yielding of the infill plates shall be determined in accordance with Clause 27.9.2.2.

27.10.2.2
Infill plate splices shall be designed to resist forces determined in accordance with Clause 27.9.2.2 and proportioned so as not to inhibit the formation of a uniform tension field in the panel.

27.10.3 Beams

27.10.3.1
Beams shall be Class 1 or Class 2 sections braced in accordance with Clause 13.7 a).

27.10.3.2
Beams shall meet the requirements of Clause 27.9.3.3.

27.10.4 Columns
Clause 27.9.4 shall apply, except that in applying Clause 27.9.4.2 the forces induced by the beams shall be determined using Clause 27.10.6.2.

Where the specified one-second spectral acceleration ratio $I_E F_a S_a$ (1.0) is less than 0.30, the design forces for the column splices in Clause 27.1.4 need not be taken into account.

27.10.5 Column joint panel zones
If rigid beam-to-column connections are used, the horizontal shear resistance of the column joint panel zone shall meet the requirements of Clause 27.2.4.2.

27.10.6 Beam-to-column joints and connections

27.10.6.1
If rigid beam-to-column connections are used, they shall have a moment resistance equal to $R_y M_{pb}$.

27.10.6.2
The factored resistance of the beam web-to-column connection shall equal or exceed the effects of gravity loads and tension forces in the infill plates, as determined in accordance with Clause 27.9.2.2, acting above and below the beams. If rigid beam-to-column connections are used, the design forces shall include shears induced by moments of $R_y M_{pb}$ acting at plastic hinge locations and these moments may be taken as $1.18(R_y M_{pb})(1 - C_f/\phi C_y)$, where C_f and C_y are as defined in Clause 27.9.7.2.

27.11 Conventional construction, R_d = 1.5, R_o = 1.3

27.11.1
Structural systems in this category have some capacity to dissipate energy through localized yielding and friction that inherently exists in traditional design and construction practices. Except as otherwise specified in Clause 27.11, the requirements of Clauses 27.1 to 27.10 and 27.12 shall not apply to these systems.

Diaphragms and connections of primary framing members and diaphragms of the seismic-load-resisting system of steel-framed buildings with specified short-period spectral acceleration ratios ($I_E F_a S_a$(0.2)) greater than 0.45 designed to resist seismic loads based on a force reduction factor, R_d, of 1.5 shall be
a) proportioned so that the expected connection failure mode is ductile; or
b) designed to resist gravity loads combined with the seismic load multiplied by R_d.

The connection design load need not exceed the gross section strength of the members being joined, as determined using the probable yield stress $R_y F_y$.

27.11.2
Cantilever column structures composed of single or multiple beam-columns fixed at the base and pin-connected or free at the upper ends shall
a) have Class 1 section columns;
b) have U_2 not greater than 1.25; and
c) have base connections designed to resist a moment of $1.1R_y$ times the nominal flexural resistance of the column, but need not exceed the value corresponding to $R_d R_o$ = 1.0.

27.11.3
When the specified short-period spectral acceleration ratio ($I_E F_a S_a(0.2)$) is greater than or equal to 0.35, seismic force resisting systems other than cantilever column structures as specified in Clause 27.11.2 and not part of an assembly occupancy building as specified in the *NBCC*, may exceed 15 m in height if

a) all factored seismic forces for ultimate limit states are increased linearly by 2% per metre of height above 15 m, without exceeding forces corresponding to $R_d R_o$ = 1.3;
b) the height does not exceed 40 m when the specified short-period acceleration ratio ($I_E F_a S_a(0.2)$) is greater than 0.75 or the specified one-second spectral acceleration ratio ($I_E F_v S_a(1.0)$) is greater than 0.30;
c) the height does not exceed 60 m when the specified short-period spectral acceleration ratio ($I_E F_a S_a(0.2)$) is greater than or equal to 0.35 but less than or equal to 0.75;
d) the seismic forces and deformations are determined using the Dynamic Analysis Procedure described in the *NBCC*;
e) the requirements of Clauses 27.1.3 to 27.1.8 are satisfied;
f) beams, columns, and I-shaped or HSS bracing members are Class 1 or Class 2 sections;
g) for bracing members with slenderness equal to or less than 200, the width-to-thickness ratios is less than $170/\sqrt{F_y}$ for the legs of angles and flanges of channels and $670/\sqrt{F_y}$ for the webs of channels;
h) the columns are designed to resist in compression the effects of gravity loads combined with 1.30 times the member factored seismic forces, where the seismic induced axial loads for columns that are part of two or more intersecting seismic-force-resisting systems are obtained from analysis of the structure independently in any two orthogonal directions for 100% of the earthquake loads applied in one direction plus 30% of the earthquake loads in the perpendicular direction;
i) connections are designed to resist the effects of gravity loads combined with 1.30 times the member factored seismic forces, without exceeding the gross section strength of the members being joined, as determined using the probable yield stress $R_y F_y$;
j) connections are designed and detailed such that the governing failure mode is ductile when the member gross section strength does not control the connection design loads;
k) the factored seismic forces for diaphragms are determined for forces corresponding to $R_o R_d$ = 1.3; and
l) compression members of the seismic-force-resisting system that are intersected by bracing members at an unbraced location are designed for an additional out-of-plane transverse force equal to 10% of the axial load carried by the compression members at that intersection point.

27.12 Special seismic construction
Other framing systems and frames that incorporate special bracing, ductile truss segments, seismic isolation, or other energy-dissipating devices shall be designed on the basis of published research results or design guides, observed performance in past earthquakes, or special investigation. A level of safety and seismic performance comparable to that required by Clause 27 shall be provided.

28 Shop and field fabrication and coating

28.1 Cambering, curving, and straightening
Cambering, curving, and straightening may be done by mechanical means, local application of heat, or both. The temperature of heated areas as measured by approved methods shall not exceed the limits specified in CSA W59.

28.2 Thermal cutting

Thermal cutting shall be performed by guided machine where practicable. Thermally-cut edges shall meet the requirements of CSA W59. Re-entrant corners shall be free from notches and have the largest practicable radii, with a minimum radius of 14 mm.

28.3 Sheared or thermally cut edge finish

28.3.1

Planing or finishing of sheared or thermally cut edges of plates or shapes shall not be required unless noted on the drawings or included in a stipulated edge preparation for welding.

28.3.2

The use of sheared edges in the tension area shall be avoided in locations subject to plastic hinge rotation at factored loading. Sheared edges, if used, shall be finished smooth by grinding, chipping, or planing. The requirements of this Clause shall be noted on design drawings and on shop details where applicable.

28.3.3

All burrs over 2 mm in height shall be removed. Projections and burrs under 2 mm in height shall be removed

a) when needed for proper fit-up for welding; and
b) when they create a hazard during or after construction.

28.4 Fastener holes

28.4.1 Drilled and punched holes

Unless otherwise shown on design documents or as specified in Clause 22.3.5, holes

a) shall be made 2 mm larger than the nominal diameter of the fastener;
b) may be punched when the thickness of the material is not greater than the nominal fastener diameter plus 4 mm;
c) shall be either drilled from the solid or sub-punched or sub-drilled and reamed when the material is greater than the nominal fastener diameter plus 4 mm; and
d) shall be drilled in CSA G40.21-700Q or ASTM A514 steels more than 13 mm thick.

28.4.2 Holes at plastic hinges

In locations subject to plastic hinge rotation at factored loading, fastener holes in the tension area shall be either sub-punched and reamed or drilled full size. This requirement shall be noted on design drawings and shop details.

28.4.3 Thermally cut holes

Thermally cut holes produced by guided machine may be used in statically loaded structures if the actual hole size does not exceed the nominal hole size by more than 1 mm. Gouges not exceeding 1.5 mm deep may be permitted along edges of thermally cut slots. Manually cut fastener holes may be permitted only with the approval of the designer.

28.4.4 Alignment

Drifting done during assembly to align holes shall not distort the metal or enlarge holes. Holes in adjacent parts shall match well enough to permit easy entry of bolts. Holes, except oversize or slotted holes, may be enlarged to admit bolts by a moderate amount of reaming. However, gross mismatch of holes shall be cause for rejection.

28.5 Joints in contact bearing

Joints in compression that depend on contact bearing shall have the bearing surfaces prepared to a common plane by milling, sawing, or other suitable means. Surface roughness shall have a roughness height rating not exceeding 500 (12.5 µm), as specified in CSA B95, unless otherwise specified by the designer.

When shop assembled, such joints shall have at least 75% of the entire contact area in bearing. A separation not exceeding 0.5 mm shall be considered acceptable as bearing. The separation of any remaining portion shall not exceed 1 mm. A gap of up to 3 mm may be packed with non-tapered steel shims to meet the requirements of this Clause. Shims need not be other than mild steel, regardless of the grade of the main material.

28.6 Member tolerances

28.6.1

Structural members consisting primarily of a single rolled shape shall be straight within the tolerances allowed in CSA G40.20, except as specified in Clause 28.6.4.

28.6.2

Built-up bolted structural members shall be straight within the tolerances allowed for rolled wide-flange shapes in CSA G40.20, except as specified in Clause 28.6.4.

28.6.3

Dimensional tolerances of welded structural members shall be those specified in CSA W59, unless otherwise specified by the designer.

28.6.4

The out-of-straightness of fabricated compression members shall not exceed 0.001 of the axial length between points that are to be laterally supported.

28.6.5

Beams with bow within the straightness tolerance shall be fabricated so that, after erection, the bow due to rolling or fabrication shall be upward.

28.6.6

Completed members shall be free from twists, bends, and open joints. Sharp kinks or bends shall be cause for rejection.

28.6.7

A variation of 1 mm is permissible in the overall length of members with both ends finished for contact bearing.

28.6.8
Members without ends finished for contact bearing that are to be framed to other steel parts of the structure may have a variation from the detailed length not greater than 2 mm for members 10 m or less in length and not greater than 4 mm for members more than 10 m in length.

28.7 Cleaning, surface preparation, and shop coating

28.7.1 General
Steelwork need not be coated unless required by Clause 6.6 or otherwise specified by the designer.

28.7.2 Uncoated steel

28.7.2.1
Steelwork need not be cleaned of oil, grease, dirt, and other foreign matter unless encased in concrete or otherwise specified by the designer.

28.7.2.2
Steelwork to be encased in concrete need not be coated. Steelwork that is designed to act compositely with reinforced concrete and depends on natural bond for interconnection shall not be coated.

28.7.3 Coated steel

28.7.3.1 General
The requirements of the coating system, including surface preparation, minimum finished coating thickness, and coating or performance specifications, shall be specified to meet service conditions. The primer and subsequent coats shall be compatible. Coatings shall be applied thoroughly and evenly to dry, clean surfaces.

28.7.3.2 Surface preparation
Steelwork shall be cleaned of all loose mill scale, loose rust, weld slag and flux deposit, oil, grease, dirt, other foreign matter, and excessive weld spatter prior to application of the coating. When specified, special surface preparation prior to coating shall meet the requirements of SSPC SP 1; SSPC SP 2; SSPC SP 3; SSPC SP 5/NACE No. 1; SSPC SP 6/NACE No. 3; SSPC SP 7/NACE No. 4; SSPC SP 10/NACE No. 2; SSPC SP 11; SSPC SP 12; or SSPC SP 14, as applicable.

28.7.3.3 One-coat systems
Steelwork to be coated shall, at a minimum, be given a one-coat paint intended to withstand exposure to an essentially non-corrosive atmosphere for a period not exceeding six months in accordance with CISC/CPMA 1-73a, unless otherwise specified.

A one-coat shop primer intended to withstand exposure to an essentially non-corrosive atmosphere for a period not exceeding 12 months shall comply with CISC/CPMA 2-75, unless otherwise specified by the designer.

28.7.3.4 Inaccessible surfaces
Surfaces that will be inaccessible after assembly shall be cleaned or cleaned and coated, as necessary, prior to assembly. Inside surfaces of enclosed spaces that will be entirely sealed off from any external source of oxygen need not be coated.

28.7.3.5 Field coating
Unless otherwise specified by the designer, the cleaning of steelwork in preparation for field coating, touch-up of shop coat, spot-coating of field fasteners, and general field coating shall not be considered part of the erection work.

28.7.4 Special surfaces

28.7.4.1
Coated-faying surfaces in high-strength bolted slip-critical joints shall meet the requirements of Clause 23.3.

28.7.4.2
For members in compression, surfaces that are finished to bear shall be cleaned before assembly but shall not be coated unless otherwise specified by the designer.

28.7.4.3
Joints that are to be welded shall be kept free of all foreign matter, including paint, primer, or other coatings that could be detrimental to achieving a sound weldment.

28.7.5 Metallic zinc coatings

28.7.5.1
Material to be hot-dip galvanized shall comply with CAN/CSA-G164.

28.7.5.2
Material to be zinc metallized shall comply with CSA G189.

29 Erection

29.1 Temporary conditions

29.1.1 General
Suitable provisions shall be made in accordance with this Standard to ensure that an adequate margin of safety exists in the uncompleted structure and members during erection. (See also Clause 4.3.4.)

29.1.2 Temporary loads
Suitable provisions shall be made to ensure that the loads incurred during steel erection can be safely sustained for their duration and without permanent deformation or other damage to any member of the steel frame and other building components supported thereby.

Temporary loads can include but are not limited to loads due to wind, equipment, equipment operation, and storage of construction materials.

29.1.3 Temporary bracing

Temporary bracing shall be employed whenever necessary to withstand all loads to which the structure may be subject during steel erection. Temporary bracing shall be left in place undisturbed as long as necessary for the safety and integrity of the structure.

29.1.4 Adequacy of temporary connections

As erection progresses, the work shall be securely bolted or welded to resist safely all dead, wind, and erection loads and to provide necessary structural integrity.

29.2 Alignment

Permanent welding or bolting shall not be performed until as much of the structure as will be stiffened thereby has been suitably aligned.

29.3 Erection tolerances

29.3.1 General

The steel framework shall be erected true and plumb within the specified tolerances. The tolerances specified in Clauses 29.3.2 to 29.3.11 are the maximum allowable tolerances for a given member.

Note: *A member tolerance can be limited to less than the allowed tolerance due to a stricter tolerance controlling the member to which it is framed into or to a member that it supports.*

29.3.2 Elevation of base plates

Column base plates shall be considered to be at their proper elevation if the following tolerances are not exceeded:

a) for single- and multi-storey buildings designed as simple construction as specified in Clause 8.3: ±5 mm from the specified elevation; and
b) for single- and multi-storey buildings designed as continuous construction as specified in Clause 8.2 or as partially restrained construction as specified in Clause 8.4: ±3 mm from the specified elevation.

29.3.3 Plumbness of columns

Unless otherwise specified by the designer, columns shall be considered plumb if their verticality does not exceed the following tolerances:

a) for exterior columns of multi-storey buildings: 1/1000, but not more than 25 mm toward or 50 mm away from the building line in the first 20 storeys, plus 2 mm for each additional storey, up to a maximum of 50 mm toward or 75 mm away from the building line over the full height of the building;
b) for columns adjacent to elevator shafts: 1/1000, but not more than 25 mm in the first 20 storeys, plus 1 mm for each additional storey, up to a maximum of 50 mm over the full height of the elevator shaft; and
c) for all other columns: 1/500.

Column plumbness shall be measured from the actual column centreline at the base of the column to its centreline at the next adjacent storey. Deviation from straightness of the erected column shall meet the requirements of Clause 28.6.

29.3.4 Horizontal alignment of members

Unless otherwise specified by the designer, spandrel beams shall be considered aligned when the offset of one end relative to the other from the alignment shown on the drawings does not exceed $L/1000$. However, the offset need not be less than 3 mm and shall not exceed 6 mm.

Other members shall be considered aligned when the offset of one end relative to the other from the alignment shown on the drawings does not exceed $L/500$. However, the offset need not be less than 3 mm and shall not exceed 12 mm.

29.3.5 Elevations of members

The elevations of the ends of members shall be within 10 mm of the specified member elevation. Allowances shall be made for initial base elevation, column shortening, differential deflections, temperature effects, and other special conditions, but the maximum deviation from the specified slope shall not exceed $L/500$. The difference from the specified elevation between member ends that meet at a joint shall not exceed 6 mm.

29.3.6 Crane runway beams

Unless otherwise required by the operational characteristics of the crane, crane runway beams and monorail beams shall be erected within the following tolerances:

a) The slope of a member shall not exceed $L/1000$. However, the difference in elevation of the ends need not be less than 3 mm and shall not exceed 6 mm. The difference in elevation of opposite points on two parallel runway beams shall not exceed 1/1000 of the distance between the runway beams and shall not exceed 6 mm.
b) The offset of one end of the member relative to the other from the horizontal alignment shown on the drawings shall not exceed $L/500$. However, the offset need not be less than 3 mm and shall not exceed 8 mm.
c) The distance between the ends of two parallel runway beams shall not deviate by more than 1/500 of the span of the runway beam. However, the difference in the distances between the runway beam ends need not be less than 3 mm and shall not exceed 10 mm.

29.3.7 Alignment of braced members

Members such as columns, beams, trusses, and open web steel joists that are braced between their supports shall be erected in such a way that the fabrication tolerances specified in this Standard are maintained.

29.3.8 Members with adjustable connections

Members with adjustable connections (e.g., shelf angles, sash angles, and lintels) shall be considered to be within tolerances when the following requirements are met:

a) Each piece shall be level within $L/1000$; however, the difference in elevation of the ends need not be less than 3 mm and shall not exceed 6 mm.
b) Adjoining ends of members shall be aligned vertically and horizontally within 2 mm.
c) The location of the members both vertically and horizontally shall be within 10 mm of the location established by the dimensions on the drawings.

29.3.9 Column splices

Column splices and other compression joints that depend on contact bearing as part of the splice resistance shall, after alignment, have a maximum allowable separation of 6 mm. Any gap exceeding

1.5 mm shall be packed with non-tapered steel shims. Shims need not be other than mild steel, regardless of the grade of the main material.

29.3.10 Welded joint fit-up

The fit-up of joints that are to be field-welded shall be within the tolerances shown on the erection diagrams and shall not exceed the tolerances specified in CSA W59 when welding is completed.

29.3.11 Bolted joint fit-up

Bolted joint fit-up shall meet the requirements of Clause 28.4.4.

30 Inspection

30.1 General

Material and quality of work shall at all times be subject to inspection by qualified inspectors who represent and are responsible to the designer. The inspection shall cover shop work and field erection work to ensure compliance with this Standard.

30.2 Co-operation

Insofar as possible, all inspections shall be made in the fabricator's shop. The fabricator shall co-operate with the inspector and permit access for inspection to all places where work is being done. The inspector shall co-operate in avoiding undue delay in the fabrication or erection of the steelwork.

30.3 Rejection

Material or quality of work not meeting the requirements of this Standard may be rejected at any time during the progress of work once non-compliance is established.

30.4 Inspection of high-strength bolted joints

The inspection of high-strength bolted joints shall be performed in accordance with Clause 23.8.

30.5 Third-party welding inspection

When third-party welding inspection (visual and/or NDE) is specified by the owner, the welding inspection shall be performed by firms certified to CSA W178.1, except that visual inspection may also be performed by persons certified to Level 2 or 3 of CSA W178.2.

30.6 Identification of steel by marking

In the fabricator’s plant, steel used for main components shall at all times be marked to identify its specification (and grade, if applicable). This shall be done by suitable markings or by recognized colour coding, except that cut pieces identified by piece mark and contract number need not continue to carry specification identification markings when it has been satisfactorily established that such cut pieces meet the required material specifications.

Table 1
Maximum width (or diameter)-to-thickness ratios: Elements in axial compression
(See Clauses 11.2, 13.3.1, 13.3.3.1, 13.3.5, and 14.4.2.)

Description of elements	Limits
Elements supported along one edge such as Flanges of I-sections, T-sections, and channels Stiffeners of Plate-girders	$\frac{b_{el}}{t} \le \frac{200}{\sqrt{F_y}}$
Legs of angles	$\frac{b_{el}}{t} \le \frac{250}{\sqrt{F_y}}$
Element supported along one edge and restrained by a plate that is substantially stiffer than the element itself, such as: Stems of T-sections	$\frac{b_{el}}{t} \le \frac{340}{\sqrt{F_y}}$
Element supported along two edges, such as: Flanges of rectangular hollow sections Flange cover plates and diaphragm plates between lines of fasteners or welds, web of I-shape sections. Web supported on both edges	$\frac{b_{el}}{t} \le \frac{670}{\sqrt{F_y}}$
Perforated cover plates	$\frac{b_{el}}{t} \le \frac{840}{\sqrt{F_y}}$
Circular hollow sections	$\frac{D}{t} \le \frac{23\ 000}{F_y}$

Table 2
Maximum width (or diameter)-to-thickness ratios: Elements in flexural compression
(See Clauses 11.2 and 27.7.2.7.)

Description of elements	Section classification limits		
	Class 1	**Class 2**	**Class 3**
Element supported along one edge and under flexural compression, such as Flanges of I-sections or T-sections under bending about the major axis Plates projecting from element in compression elements Outstanding legs of pairs of angles in continuous contact with an axis of symmetry in the plane of loading	$\frac{b_{el}}{t} \leq \frac{145}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \leq \frac{170}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \leq \frac{200}{\sqrt{F_y}}$
Element supported along one edge under compressive stress due to flexural bending but with a part in traction, such as Stems of T-sections Flange of I-section under flexure around the minor axis	$\frac{b_{el}}{t} \leq \frac{145}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \leq \frac{170}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \leq \frac{340}{\sqrt{F_y}}$
Element supported along two edges mainly under compressive stress due to flexural bending such as Flanges of rectangular hollow sections	$\frac{b_{el}}{t} \leq \frac{420}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \leq \frac{525}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \leq \frac{670}{\sqrt{F_y}}$
Element supported along two edges mainly under compressive stress due to flexural bending such as Flanges of box sections Web of I-section under bending about the minor axis Flange cover plates and diaphragm plates between lines of fasteners or welds	$\frac{b_{el}}{t} \leq \frac{525}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \leq \frac{525}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \leq \frac{670}{\sqrt{F_y}}$

(Continued)

Table 2 (Concluded)

Description of elements	Section classification limits		
	Class 1	Class 2	Class 3
Element supported along two edges and subject to compression under combined flexure about the major axis and an axial force such as Webs of I-sections	$\frac{h}{w} \le \frac{1100}{\sqrt{F_y}}\left(1-0.39\frac{C_f}{\phi C_y}\right)$	$\frac{h}{w} \le \frac{1700}{\sqrt{F_y}}\left(1-0.61\frac{C_f}{\phi C_y}\right)$	$\frac{h}{w} \le \frac{1900}{\sqrt{F_y}}\left(1-0.65\frac{C_f}{\phi C_y}\right)$
Circular hollow sections	$\frac{D}{t} \le \frac{13\,000}{F_y}$	$\frac{D}{t} \le \frac{18\,000}{F_y}$	$\frac{D}{t} \le \frac{66\,000}{F_y}$

Note: *Elements with ratios exceeding Class 3 limits are Class 4 sections.*

Table 3
Values of k_s and c_s
(See Clauses 13.12.2.2 and 23.2.)

Contact surface of bolted parts			c_s		
			Turn-of-nut		Other
Class	Description	k_s	A325 and A325M* bolts	A490 and A490M* bolts	F959, F1852, and F2280
A	Unpainted clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel or hot-dipped galvanized and roughened surfaces	0.30	1.00	0.92	0.78
B	Unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel	0.52	1.04	0.96	0.81

* *Bolts are installed by the turn-of-nut method.*

Notes:
1) *Class A and Class B coatings are those coatings that provide a mean slip coefficient, k_s, of not less than 0.33 and 0.50, respectively.*
2) *Values of c_s for 5% probability of slip for values of k_s not specified in this Table are given as Slip Factor D in the RCSC's Guide to Design Criteria for Bolted and Riveted Joints.*

Table 4
Matching electrode ultimate tensile strengths for CSA G40.21 steels
(See Clause 13.13.1.)

Matching electrode ultimate tensile strength* MPa	G40.21 Grades, MPa						
	260	300	350	380	400	480	700
430	X	X†					
490	X	X	X‡	X			
550					X‡		
620						X	
820							X

* *The electrode ultimate tensile strength is ten times the first two digits of the electrode classification in CSA W48.*
† *For HSS only.*
‡ *For unpainted applications using "A" or "AT" steels where the deposited weld metal is to have atmospheric corrosion resistance or colour characteristics, or both, similar to the base metal, the requirements of Clauses 5.2.1.4 and 5.2.1.5 of CSA W59 shall apply.*

Note: *For matching conditions of ASTM steels, see Table 11-1 or 12-1 of CSA W59.*

Table 5
Maximum intermediate transverse stiffener spacing
(See Clause 14.5.2.)

Web depth-to-thickness ratio, h/w	Maximum distance between stiffeners, a, in terms of clear web depth, h
≤ 150	$3h$
> 150	$\dfrac{67\,500\,h}{(h/w)^2}$

Table 6
Minimum edge distance for bolt holes, mm
(See Clauses 22.3.2 and 22.3.4.)

Bolt diameter		**Minimum edge distance**	
mm	**in**	**At sheared edge**	**At rolled or sawn edges, or edges cut by gas*, plasma, laser, or water jet**
—	5/8	28	22
16	—	28	22
—	3/4	32	25
20	—	34	26
—	7/8	38†	28
22	—	38	28
24	—	42	30
—	1	44†	32
27	—	48	34
—	1-1/8	51	38
30	—	52	38
—	1-1/4	57	41
36	—	64	46
Over 36	Over 1-1/4	1.75 × diameter	1.25 × diameter

* *Gas-cut edges shall be smooth and free from notches. The edge distance in this column may be decreased by 3 mm when the hole is at a point where calculated stress under factored loads is not more than 0.3 of the yield stress.*
† *At the ends of beam-framing angles, this distance may be* 32 mm.

Table 7
Minimum bolt tension, kN
(See Clauses 23.7.1, 23.7.3, 23.7.4, 23.8.2, and I.1.)

Bolt diameter		Minimum bolt tension*	
mm	**in**	**A325, A325M, and F1852 bolts**	**A490, A490M, and F2280 bolts**
—	1/2	53	67
—	5/8	85	107
16	—	91	114
—	3/4	125	157
20	—	142	178
—	7/8	174	218
22	—	176	220
24	—	205	257
—	1	227	285
27	—	267	334
—	1-1/8	249	356
30	—	326	408
—	1-1/4	316	454
—	1-3/8	378	538
36	—	475	595
—	1-1/2	458	658

** Equal to 70% of the specified minimum tensile strength.*

Table 8
Nut rotation from snug-tight condition*
(See Clauses 23.7.2and I.1.)

Disposition of outer faces of bolted parts	Bolt length†	Turn
Both faces normal to bolt axis or one face normal to axis and other face sloped 1:20 max. (bevelled washers not used)‡	Up to and including 4 diameters	1/3
	Over 4 diameters and not exceeding 8 diameters or 200 mm	1/2
	Exceeding 8 diameters or 200 mm	2/3
Both faces sloped 1:20 max. from normal to bolt axis (bevelled washers not used)‡	All lengths of bolts	3/4

** Nut rotation is rotation relative to a bolt regardless of whether the nut or bolt is turned. The tolerance on rotation is 30° over or under. This Table applies to coarse-thread heavy-hex structural bolts of all sizes and lengths used with heavy-hex semi-finished nuts.*
† Bolt length is measured from the underside of the head to the extreme end of point.
‡ Bevelled washers are necessary when A490, A490M, or F2280 bolts are used.

Table 9
Detail categories for load-induced fatigue
(See Clauses 26.3.1 and 26.3.4.)

General condition	Situation	Detail category	Illustrative example (see Figure 2)
Plain members	Base metal		
	• with rolled or cleaned surfaces. Flame-cut edges with a surface roughness not exceeding 1000 (25 µm) as specified by CSA B95	A	1, 2
	• of unpainted weathering steel	B	
	• at re-entrant corners of copes with a radius ≥ 35 mm and ground smooth	E1	2a
	• at net section of eyebar heads and pin plates	E	
Built-up members	Base metal and weld metal in components, without attachments, connected by		3, 4, 5, 7
	• continuous full-penetration groove welds with backing bars removed; or	B	
	• continuous fillet welds parallel to the direction of applied stress;	B	
	• continuous full-penetration groove welds with backing bars in place;or	B1	
	• continuous partial-penetration groove welds parallel to the direction of applied stress.	B1	
	Base metal at ends of partial-length cover plates		
	• bolts in slip-critical connections;	B	22
	• narrower than the flange, with or without end welds, or wider than the flange with end welds		
	– flange thickness ≤ 20 mm	E	7
	– flange thickness > 20 mm	E1	7
	• wider than the flange without end welds	E1	7
Groove-welded splice connections with weld soundness established by NDT and all required grinding in the direction of the applied stresses	Base metal and weld metal at full-penetration groove-welded splices		
	• of plates of similar cross-sections with welds ground flush	B	8, 9
	• with 600 mm radius transitions in width with welds ground flush	B	11
	• with transitions in width or thickness with welds ground to provide slopes not steeper than 1.0 to 2.5		10, 10a
	• G40.21-700Q and 700QT base metal	B1	
	• other base metal grades	B	
	• with or without transitions having slopes not greater than 1.0 to 2.5, when weld reinforcement is not removed	C	8, 9, 10, 10a

(Continued)

Table 9 (Continued)

General condition	Situation	Detail category	Illustrative example (see Figure 2)
	• at weld access holes		
	– of rolled members	C	
	– of built-up members	D	
Longitudinally loaded groove-welded attachments	Base metal at details attached by full- or partial-penetration groove welds		
	When the detail length in the direction of applied stress is		
	• less than 50 mm	C	6, 18
	• between 50 mm and 12 times the detail thickness, but less than 100 mm	D	18
	• greater than either 12 times the detail thickness or 100 mm		
	– detail thickness < 25 mm	E	18
	– detail thickness ≥ 25 mm	E1	18
	• with a transition radius, *R*, with the end welds ground smooth, regardless of detail length		12
	– *R* ≥ 600 mm	B	
	– 600 mm > *R* ≥ 150 mm	C	
	– 150 mm > *R* ≥ 50 mm	D	
	– *R* < 50 mm	E	
	• with a transition radius, *R*, with the end welds not ground smooth	E	12
Transversely loaded groove-welded attachments with weld soundness established by NDT and all required grinding transverse to the direction of stress	Base metal at detail attached by full-penetration groove welds with a transition radius, *R*		12
	• to flange, with equal plate thickness and weld reinforcement removed		
	– *R* ≥ 600 mm	B	
	– 600 mm > *R* ≥ 150 mm	C	
	– 150 mm > *R* ≥ 50 mm	D	
	– *R* < 50 mm	E	
	• to flange, with equal plate thickness and weld reinforcement not removed, or to web		
	– *R* ≥ 150 mm	C	
	— 150 mm > *R* ≥ 50 mm	D	
	– *R* < 50 mm	E	
	• to flange, with unequal plate thickness and weld reinforcement removed		
	– *R* ≥ 50 mm	D	
	– *R* < 50 mm	E	

(Continued)

Table 9 (Continued)

General condition	Situation	Detail category	Illustrative example (see Figure 2)
	– to flange, for any transition radius with unequal plate thickness and weld reinforcement not removed	E	
Fillet-welded connections with welds normal to the direction of stress	Base metal		
	• at details other than transverse stiffener-to-flange or transverse stiffener-to-web connections	C*	19
	• at the toe of transverse stiffener-to-flange and transverse stiffener-to-web welds	C1	6
Fillet-welded connections with welds normal and/or parallel to the direction of stress	Shear stress on weld throat	E	16
Longitudinally loaded fillet-welded attachments	Base metal at details attached by fillet welds		
	• when the detail length in the direction of applied stress is		
	– less than 50 mm, and stud-type shear connectors	C	13, 14, 15, 18, 20
	– between 50 mm and 12 times the detail thickness, but less than 100 mm	D	14, 18, 20
	– greater than either 12 times the detail thickness or 100 mm		7, 14, 16, 18, 20
	▪ detail thickness < 25 mm	E	
	▪ detail thickness ≥ 25 mm	E1	
	• with a transition radius, *R*, with the end of welds ground smooth, regardless of detail length		12
	– $R \geq 50$ mm	D	
	– $R < 50$ mm	E	
	• with a transition radius with the end of welds not ground smooth	E	12
Transversely loaded fillet-welded attachments with welds parallel to the direction of primary stress	Base metal at details attached by fillet welds		12
	• with a transition radius, *R*, with the end of welds ground smooth		
	– $R \geq 50$ mm	D	
	– $R < 50$ mm	E	
	• with any transition radius with end of welds not ground smooth	E	
Mechanically fastened connections	Base metal		17
	• at gross section of high-strength bolted slip-critical connections, except axially loaded joints in which out-of-plane bending is induced in connected materials	B	
	• at net section of high-strength bolted non-slip-critical connections	B	
	• at net section of non-pretensioned bolted connections	D	

(Continued)

Table 9 (Concluded)

General condition	Situation	Detail category	Illustrative example (see Figure 2)
	• at net section of riveted connections	D	
Anchor rods and threaded parts	Tensile stress range on the tensile stress area of the threaded part, including effects of bending	E	
Fillet-welded HSS to base plate	Shear stress on fillet weld	E1	21
A325, A325M, and F1852 bolts in axial tension	Tensile stress on area A_b	See Clause 13.12.1.3	
A490, A490M, and F2280 bolts in axial tension	Tensile stress on area A_b		

Note: *The fatigue resistance of fillet welds transversely loaded is a function of the effective throat and plate thickness. See Frank and Fisher (1979).*

$$F_{sr} = F_{sr}^{c}\left[\left(0.06 + 0.79H / t_p\right) / \left(0.64 t_p^{\ 1/6}\right)\right]$$

where

H = *weld leg size*

C_{sr} = *fatigue resistance for Category C as determined in accordance with Clause 26.3.3. This assumes no penetration at the weld root*

t_p = *plate thickness*

Table 10
Fatigue constants for detail categories
(See Clauses 26.3.3 and 26.3.4.)

Detail category	Fatigue life constant, γ	Constant amplitude threshold stress range, F_{srt}, MPa	nN'	Fatigue life constant, γ'
A	8190×10^9	165	1.82×10^6	223×10^{15}
B	3930×10^9	110	2.95×10^6	47.6×10^{15}
B1	2000×10^9	83	3.50×10^6	13.8×10^{15}
C	1440×10^9	69	4.38×10^6	6.86×10^{15}
C1	1440×10^9	83	2.52×10^6	9.92×10^{15}
D	721×10^9	48	6.52×10^6	1.66×10^{15}
E	361×10^9	31	12.1×10^6	0.347×10^{15}
E1	128×10^9	18	21.9×10^6	0.0415×10^{15}

Figure 1
Fatigue constants for detail categories
(See Clauses 26.3.3 and 26.3.4.)

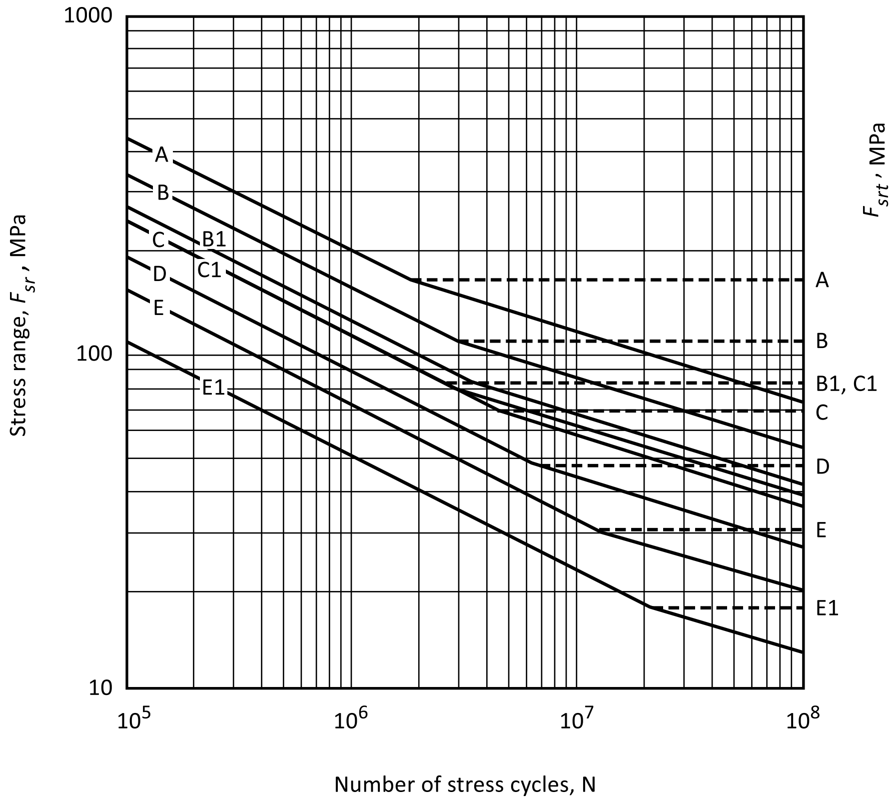


Figure 2
Illustrative examples of detail categories

(See Clause 26.3.4 and Table 9.)

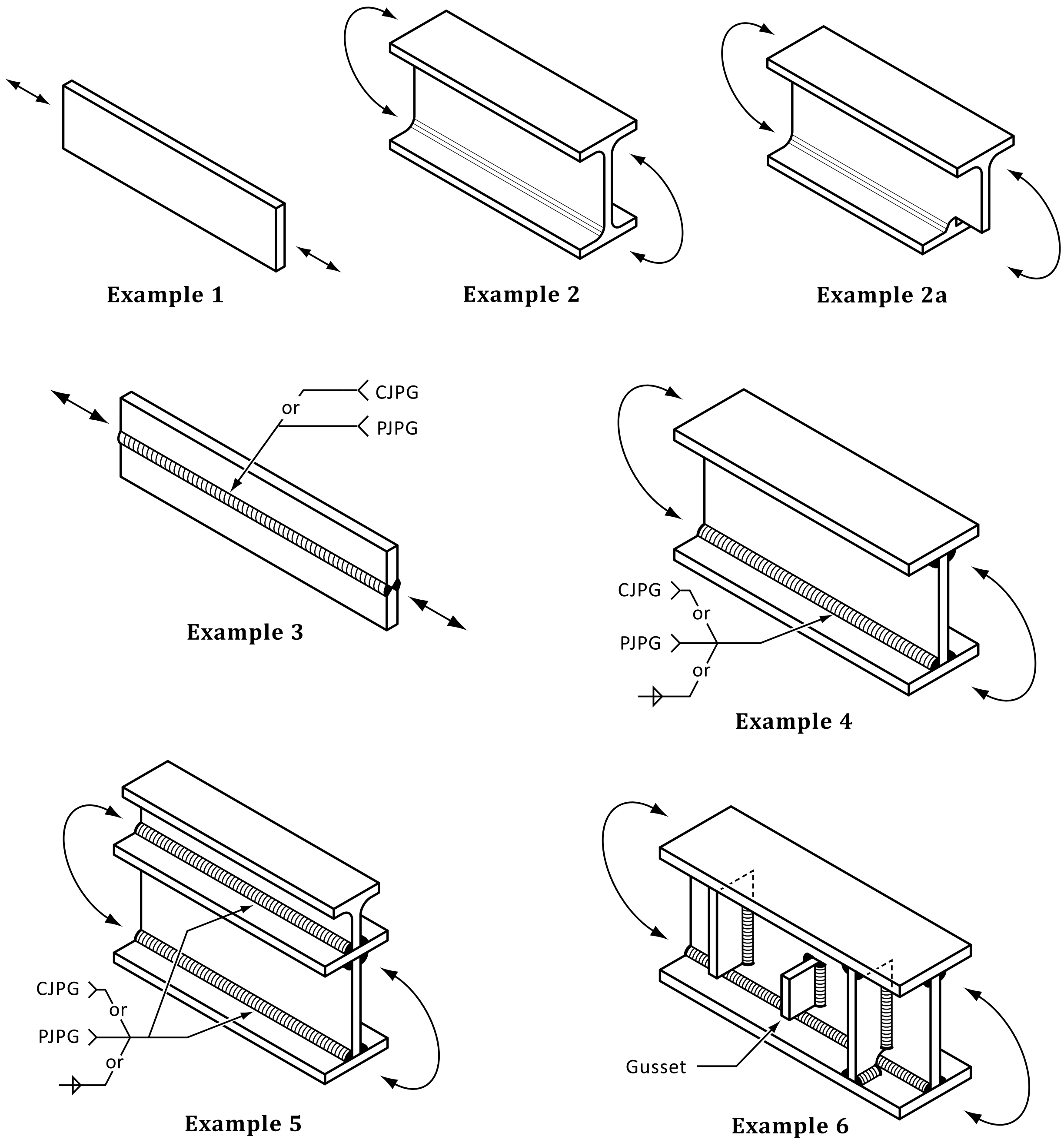


Figure 2 (Continued)

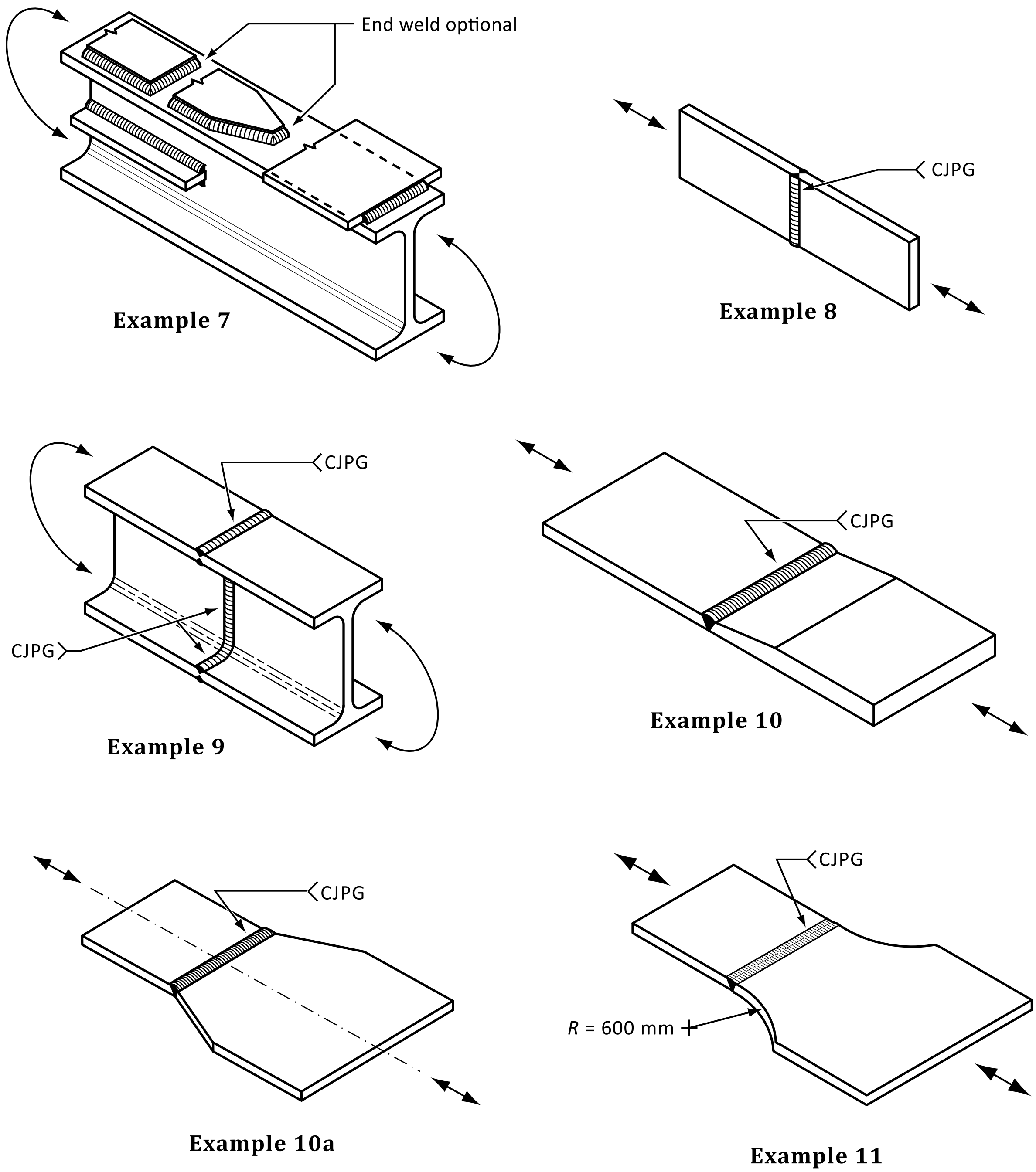
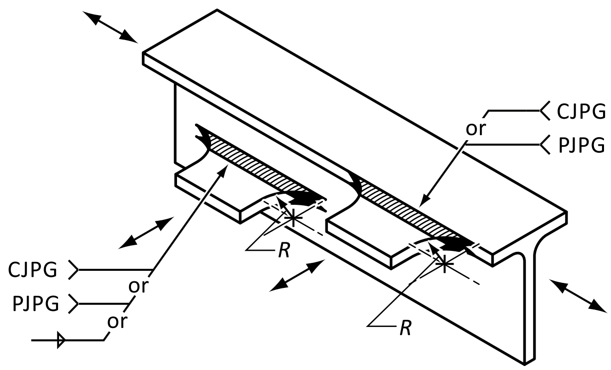
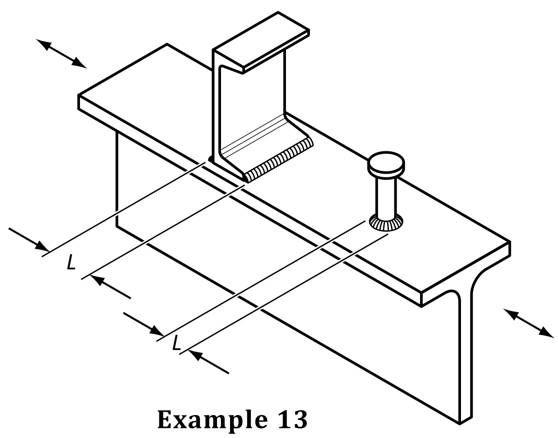


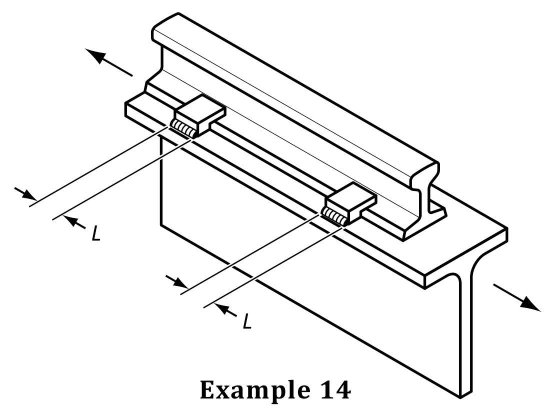
Figure 2 (Continued)



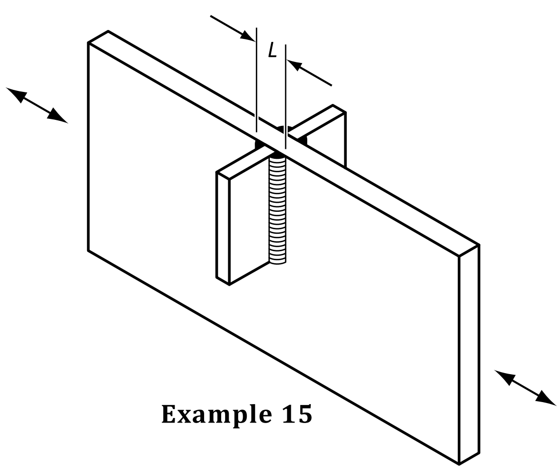
Example 12



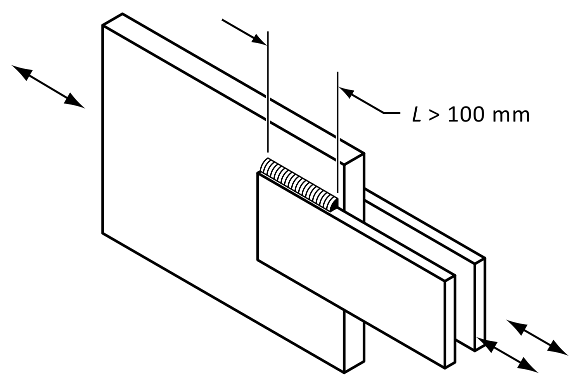
Example 13



Example 14

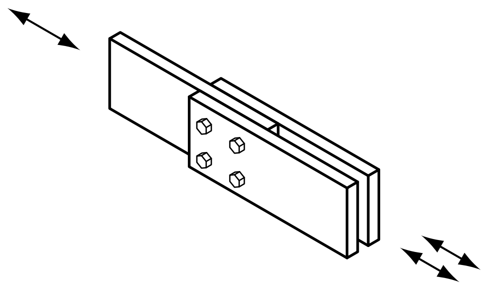


Example 15

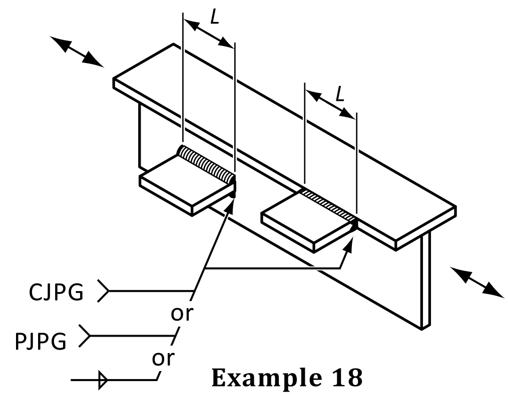


Example 16

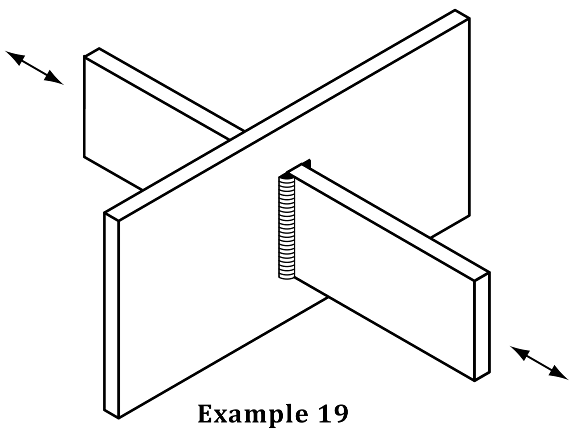
Figure 2 (Concluded)



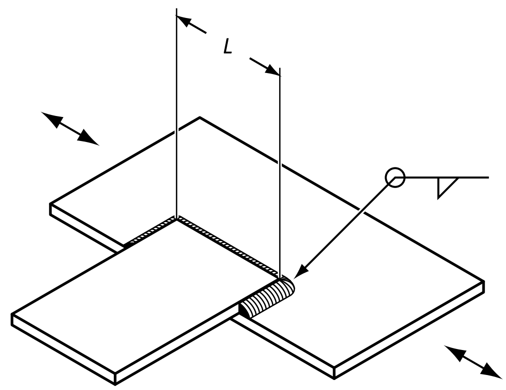
Example 17



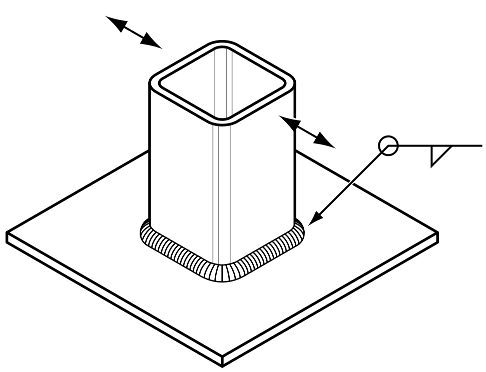
Example 18



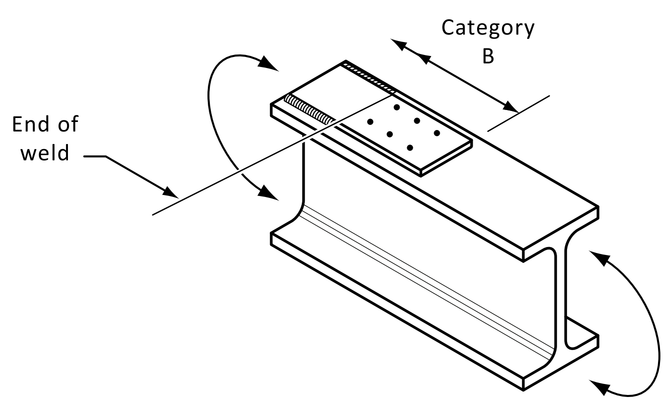
Example 19



Example 20



Example 21



Example 22

Annex A (informative)
Standard practice for structural steel

Note: *This informative Annex has been written in mandatory language to facilitate adoption by anyone wishing to do so.*

A.1 General

Matters concerning standard practice not covered by this Standard but pertinent to the fabrication and erection of structural steel (e.g., classification of material and contract documents) shall be in accordance with the CISC's *Code of Standard Practice for Structural Steel* unless otherwise clearly specified in the plans and specifications issued to the bidders.

Annex B (informative)
Margins of safety

Note: *This Annex is an informative (non-mandatory) part of this Standard.*

B.1

Code writers now use limit states design to provide a practical level of reliability over the lifetime of a structure. One of the advantages of limit states design is that by using load and resistance factors based on the statistical variation of the loads and resistances, a relatively uniform degree of reliability is obtained in the design of structures across a variety of configurations and load conditions. At the same time, economies accrue in limit states design since structures or portions of them are not designed for excessive safety, either due to the unrealistic load combinations or inaccurate modelling based on the assumed elastic behaviour of structural steel components used in the past. Moreover, by changing the reliability index in limit states design, greater or lesser safety can be assigned on a quantitative basis to entire structures or to components.

B.2

The load and resistance factors in limit states design, derived to give the desired reliability index, are related to the calculated probability of failure and are based on the statistical variations of the loads and resistances.

B.3

Limit states design was first introduced in the *NBCC*, 1975, where the reliability index for steel buildings as a whole was taken as 3.0. A greater reliability index was used for connectors so that the probability of the connector failing before the member as a whole was reduced and the more ductile mode of failure of the member was favoured. This was done to make the connections stronger than the members they joined. In the current *NBCC* and this edition of this Standard, the reliability index for steel buildings as a whole remains 3.0, and indices greater than this value are used for connections.

B.4

The development of member resistance factors used in the first limited states design standard, CSA S16.1-1974, is discussed in Kennedy and Gad Aly (1980) and others in Kennedy and Baker (1984). Since then, other resistance factors have been introduced based on statistical analyses of the resistances. That of 0.67 for welds was confirmed in the 1994 edition of CAN/CSA-S16.1, when the strength of transverse fillet welds was recognized to be 1.50 times that of longitudinal fillet welds (Lesik and Kennedy, 1990). Other resistance factors have been introduced for shear connectors, anchor rods, bearing of bolts on steel, and reinforcing bars, as well as ϕb = 0.80 for high-strength bolts in shear and tension (Kennedy, 1999a) and ϕbi = 0.80 and ϕbe = 0.75 for bearing on webs of interior loads and end reactions, respectively (Kennedy, et al., 1998; Kennedy, 1999b). Enhanced target reliability indices for calculating resistance factors of 4.5 were used for welds and bolts and 3.5 for bearing on webs. In this edition of the Standard, a new resistance factor for the block shear, net section rupture, and bolt tear-out limit states, ϕu = 0.75, has been introduced based on recent research by Driver et al. (2006) and Cai and Driver (2010). A review of resistance factors used in this Standard is presented by Schmidt and Bartlett (2002).

B.5

Cai, Q. and Driver, R.G. (2010). “Prediction of bolted connection capacity for block shear failures along atypical paths”. *AISC Engineering Journal* Fourth Quarter, 213–221 .

Driver, R.G., Grondin, G.Y., and Kulak, G.L. (2006). *"Unified block shear equation for achieving consistent reliability"*. *Journal of Constructional Steel Research*, 62 (3), March, 210–222.

Kennedy, D.J.L. (1999a). *Bolts in bearing type connections, basis for increasing the resistance factor for high strength bolts to 0.80*. S16 Committee Communication.

Kennedy, D.J.L. (1999b). *Web crippling and yielding*. S16 Committee Communication.

Kennedy, D.J.L. and Baker, K.A. (1984). "Resistance factors for steel highway bridges".*Canadian Journal of Civil Engineering*, 11 (2), June, 324–334.

Kennedy, D.J.L. and Gad Aly, M. (1980). "Limit states design of steel structures — performance factors". *Canadian Journal of Civil Engineering*, 7 (1), March, 45–77.

Kennedy, S.J., Kennedy, D.J.L., and Medhekar, M.S. (1998). "The bearing resistance of webs: Further studies of the post-buckling strength". *Proceedings of the Annual Conference*, Structural Stability Research Council, Atlanta, GA, September 21–23, 25–41.

Lesik, D.F. and Kennedy, D.J.L., (1990). "Ultimate strength of fillet welded connections loaded in plane". *Canadian Journal of Civil Engineering*, 17 (1), February, 55–67.

Schmidt, B.J. and Bartlett, F.M. (2002). "Review of resistance factor for steel: Resistance distributions and resistance factor calibration". *Canadian journal of Civil Engineering*, 29 (1), February, 109–118.

Annex C (normative)
Crane-supporting structures

Note: *This Annex is a normative (mandatory) part of this Standard.*

C.1 General

Steel structures that support overhead cranes and hoists require special consideration in order to provide safe and serviceable structures. Electrically-operated top-running overhead travelling cranes, underslung cranes, and monorails impose repetitive loads that can lead to the development and propagation of fatigue cracks in the crane-supporting structure. These loads shall be accounted for in the design and construction of the crane-supporting structure. Conditions that apply to these steel structures, where any component is subjected to fatigue loads as specified in Clause 26, are given in this Annex.

The requirements of this Standard for design for fatigue shall apply. The structural design shall take into account, among other factors, appropriate methods of analysis, rotational restraints at crane runway beam supports, crane load eccentricities, distortion leading to fatigue cracking, welded details, built-up column section details, bracing systems, deflections, and details related to crane rails. The construction specifications shall include (but not necessarily be limited to) requirements for materials, detailing, fabrication, erection, bearing and contact surfaces, dimensional tolerances, crane rail installation, and shop and field inspection.

The designer shall determine the loading parameters and the appropriate number of loading cycles at each level of load by analyzing the duty cycles for the design life of the structure, in addition to other crane details that are necessary to design the structure. This information shall be included in the structural design documents.

Note: *For design information and information to be shown on the structural design documents, see the CISC's* Crane-Supporting Steel Structures: Design Guide*.*

Annex D (informative)
Recommended maximum values for deflections for specified design live, snow, and wind loads

Note: *This Annex is an informative (non-mandatory) part of this Standard.*

D.1 General

Table D.1 provides deflection criteria for floor or roof members as a fraction of the span and for lateral drift as a fraction of the storey height. These criteria are related to the serviceability limit states. Although the criteria refer to specified live, snow, and wind loads, the designer should consider the inclusion of specified dead loads in some instances. For example, non-permanent partitions, which are classified by the *NBCC* as dead load, should be part of the loading considered under this Annex if they are likely to be applied to the structure after the completion of finishes susceptible to cracking.

D.2 Wind

Some building materials augment the rigidity provided by the steelwork; therefore, the deflections calculated for bare steel structures under wind loads can be somewhat reduced. The more common structural and non-structural elements that contribute to the stiffness of a building are masonry walls, certain types of curtain walls, masonry partitions, and concrete around steel members. Provided that the materials augmenting rigidity are accounted for in the analysis for wind loads, the deflections for comparison to the limits in Table D.1 can be reduced by a maximum of 15%. The deflections used for strength and stability calculations should not be reduced. In tall and slender structures (height greater than four times the width), the wind effects should be determined by means of dynamic analysis or wind tunnel tests.

Table D.1
Deflection criteria
(See Clauses D.1 and D.2.)

Building type	Deflection	Specified loading	Application	Maximum
Industrial	Vertical	Live, snow	Members supporting inelastic roof coverings	$L/240$
		Live, snow	Members supporting elastic roof coverings	$L/180$
		Live, snow	Members supporting floors	$L/300$
		Maximum wheel loads (no impact)	Crane runway girders for crane capacity of 225 kN and over	$L/800$
		Maximum wheel loads (no impact)	Crane runway girders for crane capacity under 225 kN	$L/600$
	Lateral	Crane lateral	Crane runway girders	$L/600$
		Crane lateral or wind	Storey drift*	$h/400$ to $h/200$
All others	Vertical	Live, snow	Members of floors and roofs supporting construction and finishes susceptible to cracking	$L/360$
		Live, snow	Members of floors and roofs supporting construction and finishes not susceptible to cracking	$L/300$
	Lateral	Wind	Building drift due to all effects	$h/400$
		Wind	Storey drift (relative horizontal movement of any two consecutive floors) in buildings in cladding and partitions without special provision to accommodate building frame deformation	$h/500$
		Wind	Storey drift, with special provision to accommodate building frame deformation	$h/400$

Legend:

h = storey height

L = length or span

* *The permissible drift of industrial buildings depends on such factors as wall construction, building height, and the effect of deflection on the operation of the crane. Where the operation of the crane is sensitive to lateral deflections, a lateral deflection of less than h/400 may be necessary.*

Annex E (informative)
Floor vibrations

Note: *This informative Annex has been written in mandatory language to facilitate adoption by anyone wishing to do so.*

E.1 General

The development of floors of lighter construction, longer spans, and less inherent damping can sometimes result in disturbing floor vibrations during normal human activity. The specific vibration characteristics of the floor should be evaluated by the building designer.

Such an evaluation shall, at a minimum, consider the following:

a) the characteristics and nature of the forcing excitations, e.g., walking and rhythmic activities (see also the *NBCC*);
b) acceptance criteria for human comfort (depending on the use and occupancy of the floor area);
c) a determination of the natural frequency of the floor framing systems, including the effect of continuity;
d) the modal damping ratio; and
e) the effective floor weights.

For guidance, see Murray, et al. (1997) and Commentary I, *User's Guide — NBC 2010: Structural Commentaries (Part 4)*.

E.2 Light-framed construction

For guidance on vibrations due to walking on light-framed construction made of light steel members and wood deck, see Applied Technology Council (1999).

E.3 Bibliography

Applied Technology Council (1999). *Minimizing floor vibration*. ATC Design Guide 1, Applied Technology Council, Redwood City, California.

Murray, T.M., Allen, D.E. and Ungar, E.E. (1997). *Floor vibrations due to human activity.* Steel Design Guide Series 11. American Institute of Steel Construction, Chicago; Canadian Institute of Steel Construction, Toronto.

Annex F (informative)
Effective lengths of columns

Note: *This Annex is an informative (non-mandatory) part of this Standard.*

F.1

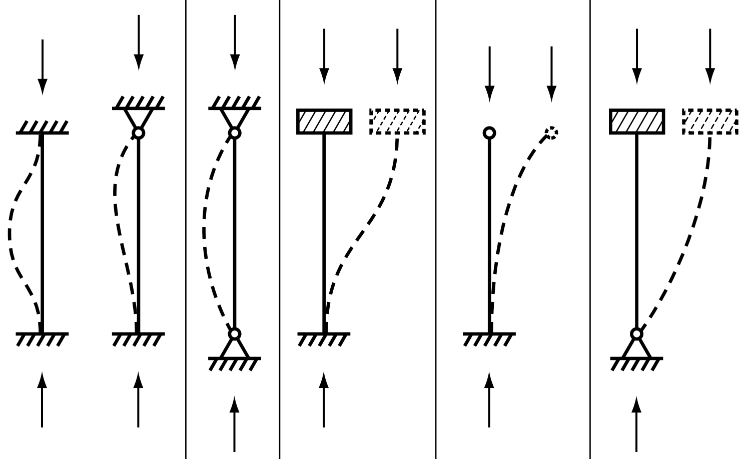
The slenderness ratio of a member whose failure mode involves buckling is defined as the ratio of the effective length to the applicable radius of gyration. The effective length, *KL*, may be thought of as the actual unbraced length, *L*, multiplied by a factor, *K*, so that the product, *KL*, is equal to the length of a pin-ended column of equal capacity to the actual member. The effective length factor, *K*, of a column of finite unbraced length therefore depends on the conditions of restraint afforded to the column at its braced locations.

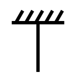
F.2

A variation in *K* between 0.65 and 2.0 will apply to the majority of cases likely to be encountered in actual structures. Figure F.1 illustrates six idealized cases in which joint rotation and translation are either fully realized or non-existent.

Figure F.1
Effective lengths of columns

(See Clause F.2.)

	a)	b)	c)	d)	e)	f)
Buckled shape of column is shown by dashed line						
Theoretical *K* value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.0	1.2	2.0	2.0

End condition code		Rotation fixed, translation fixed
		Rotation free, translation fixed
		Rotation fixed, translation free
		Rotation free, translation free

Annex G (informative)
Criteria for estimating effective column lengths in continuous frames

Note: *This Annex is an informative (non-mandatory) part of this Standard.*

G.1
Because this Standard requires the in-plane behaviour of beam columns to be based on their actual lengths (provided that, when applicable, the sway effects are included in the analysis of the structure [see Clause 8.4]), this Annex applies only to cases related to buckling, i.e., to axially loaded columns and beam columns failing by out-of-plane buckling.

G.2
Figure G.1 is a nomograph applicable to cases in which the equivalent I/L of adjacent girders that are rigidly attached to the columns is known; it is based on the assumption that all columns, in the portion of the framework considered, reach their individual critical load simultaneously. This is a conservative assumption made in the interest of simplification.

G.3
The equation on which the nomograph is based is as follows:

$$\frac{G_U G_L}{4}(\pi / K)^2 + \frac{G_U + G_L}{2}\left(1 - \frac{\pi / K}{\tan\pi / K}\right) + 2\left[\frac{\tan\pi / 2K}{\pi / K}\right] = 1$$

Subscripts U and L refer to the joints at the two ends of the column section being considered and

$$G = \frac{\Sigma I_c / L_c}{\Sigma I_g / L_g}$$

where

Σ = summation for all members rigidly connected to that joint and lying in the plane in which buckling of the column is being considered

I_c = moment of inertia of the column about the axes perpendicular to the plane of buckling

L_c = unsupported length of a column

I_g = moment of inertia of the girder about the axes perpendicular to the plane of buckling

L_g = unsupported length of a girder

G.4
For column ends supported by, but not rigidly connected to, a footing or foundation, G may be taken as 10 for practical designs. If the column end is rigidly attached to a properly designed footing, G may be taken as 1.0. Smaller values may be used if justified by analysis.

G.5
Refinements in girder I_g/L_g may be made when conditions at the far end of any particular girder are known definitely or when a conservative estimate can be made. For the case with no sideway, multiply girder stiffnesses by the following factors:

a) 1.5 if the far end of the girder is hinged; and

b) 2.0 if the far end of the girder is fixed against rotation (i.e., rigidly attached to a support that is itself relatively rigid).

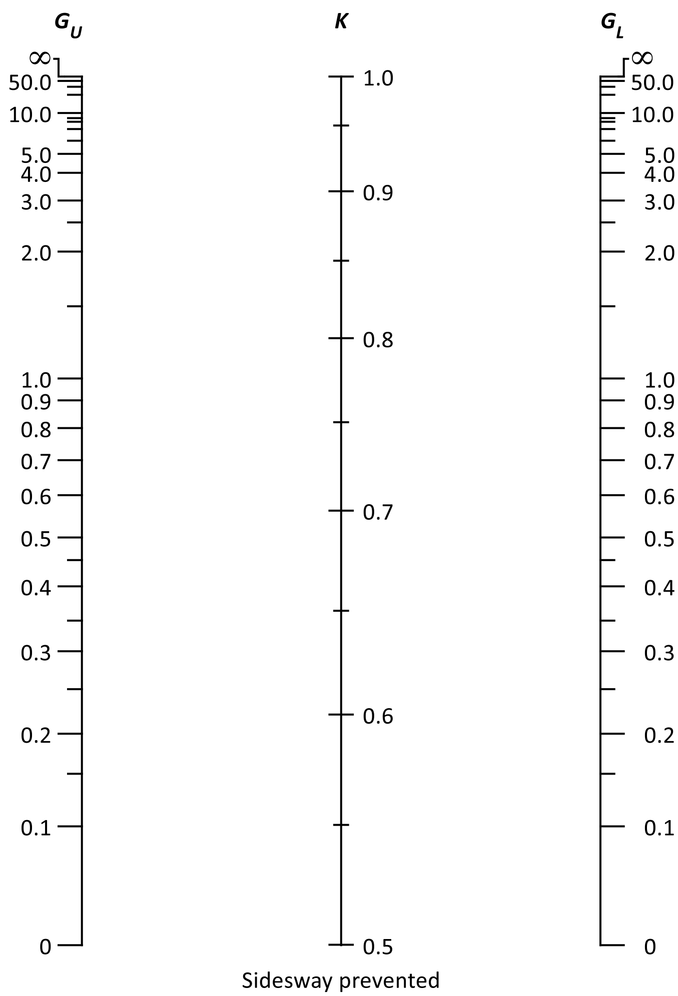
G.6

Having determined G_U and G_L for a column section, the effective length factor, K, is determined at the intersection of the straight line between the appropriate points on the scales for G_U and G_L with the scale for K.

G.7

The nomograph may be used to determine the effective length factors for the in-plane behaviour of compression members of trusses designed as axially loaded members even though the joints are rigid. In this case, there should be no in-plane eccentricities and all members of the truss meeting at the joint should not reach their ultimate load simultaneously. If it cannot be shown that all members at the joint do not reach their ultimate load simultaneously, the effective length factor of the compression members should be taken as 1.0.

Figure G.1
Nomograph for effective lengths column in continuous frames
(See Clause G.2.)



Annex H (informative)
Deflections of composite beams, joists, and trusses due to shrinkage of concrete

Note: *This Annex is an informative (non-mandatory) part of this Standard.*

H.1
Shrinkage-induced deflections result from the following process. Concrete decreases in volume as it cures, at first rapidly and then at a decreasing rate. When restrained, tensile strains and therefore tensile stresses can develop in the concrete. (It can even crack if the tensile strength is reached.)

A curing slab is restrained by the steel shape to which it is connected.

H.2
Figure H.1 shows the shrinkage strains that develop through the depth for a composite beam and the corresponding equilibrium conditions for unshored construction. It is evident that unshored composite members will deflect downward. (Shoring reduces the shrinkage deflection substantially, especially in the early stages when the rate of shrinkage is the greatest.)

H.3
Branson's (1964) method is used in this Standard to determine shrinkage deflections. As illustrated in Figure H.2 a), the first step in the method is to assume temporarily that the shrinkage of the concrete slab is not restrained by connection to the steel beam. The connection between the concrete slab and the beam is accounted for in two additional steps. First, a tensile force is applied to the centroid of the unrestrained slab so that the displacement of the slab under the force is equal to the unrestrained shrinkage displacement [see Figure H.2 b)]. Compatibility is satisfied in this step. Second, equilibrium is satisfied by applying an equal and opposite force to the composite section [see Figure H.2 c)].

The method does not account for the cracking of concrete in tension, the non-linear stress-strain relationship of concrete, and other factors. To account for these factors and match theory with test results, the free shrinkage of the concrete is multiplied by an empirical coefficient.

The method gives reasonable results when an appropriate value is used for the empirical coefficient and suitable values are used for the free shrinkage and modular ratio.

H.4
The shrinkage deflection is directly proportional to the assumed free shrinkage strain. The free shrinkage strain depends on concrete properties such as the water/cement ratio, percentage of fines, entrained air, cement content, and curing conditions. A value of 583×10^{-6} may be used if other data are not available. This value was determined for composite beams supporting 75 mm concrete topping on 75 mm deck (150 mm total thickness) for inside conditions (see Ghali, et al. (2002), Annex A.2).

H.5
The modular ratio is calculated from the age-adjusted effective modulus of concrete, which in turn depends on the aging and creep coefficients. These coefficients may be taken as 0.73 and 2.7, respectively, if other data are not available. These coefficients were determined for the composite beams described in Clause H.4, assuming the age at loading is 7 days (see Ghali, et al. (2002), Annexes A.7 and

A.2, respectively). The shrinkage deflection is not sensitive to the modular ratio because both the transformed moment of inertia of the composite beam and the distance, y, vary with it.

H.6

The procedure in this Standard is used for determining the shrinkage deflections of simply supported composite beams, joists, and trusses. For many structural configurations, moments develop at the ends of beams, joists, and trusses as a result of partial or full continuity with adjacent members. It is often appropriate to account for continuity with adjacent members when determining shrinkage deflections.

H.7

Kennedy and Brattland (1992) propose an alternative method to determine shrinkage deflections. The method uses strain compatibility between steel and concrete, and a time-dependent modulus of elasticity of concrete in tension [see Shaker and Kennedy (1991)]. It is iterative because the concrete response is non-linear. It is more difficult to use than the method specified in this Standard; however, the tensile stress-strain relationship of the concrete is satisfied.

H.8

Montgomery et al. (1983) give an example where the shrinkage deflections were excessive. Jent (1989) provides information on shrinkage effects on continuous composite beams.

H.9

Branson, D.E. (1964). *Time-dependent effects on composite concrete beams*. *Proceedings, American Concrete Institute Journal,* 61, 212–229.

Ghali, A., Favre, R. and Elbadry, M. (2002). *Concrete structures: Stresses and deformations,* 3rd ed. London: Spon Press.

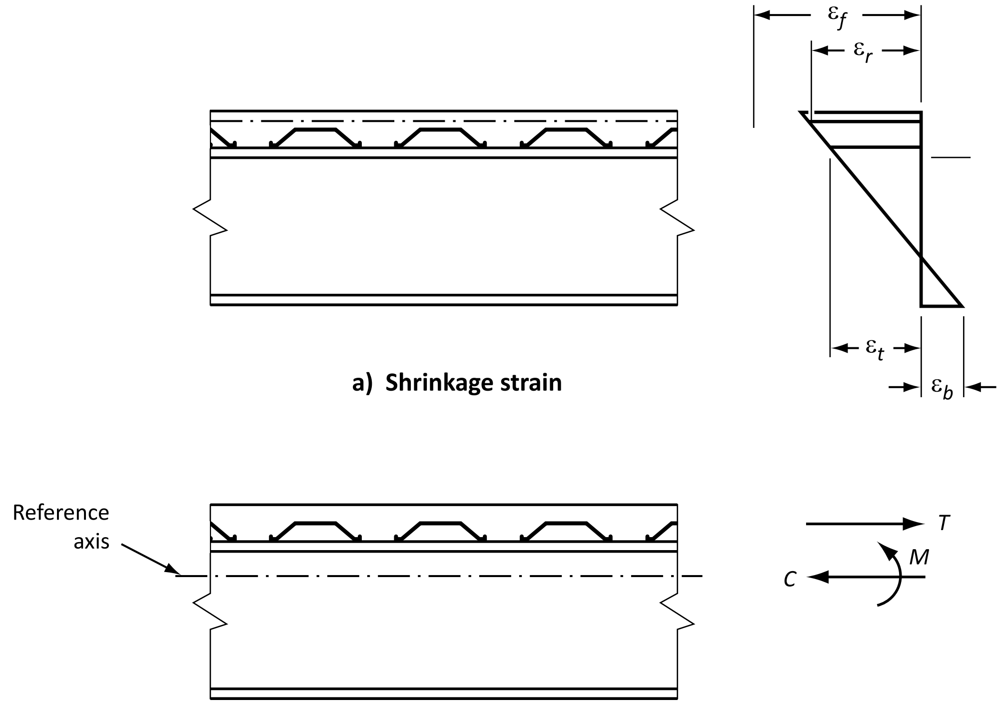
Jent, K.A. (1989). *Effects of shrinkage, creep and applied loads on continuous deck-slab composite beams*. M.Sc. thesis, Queen's University, Kingston, Ontario.

Kennedy, D.J.L. and Brattland, A. (1992). "Shrinkage tests of two full-scale composite trusses". *Canadian Journal of Civil Engineering,* 19 (2), 296–309.

Montgomery, C.J., Kulak, G.L. and Shwartsburd, G. (1983). "Deflection of a composite floor system". *Canadian Journal of Civil Engineering,* 10 (2), 192–204.

Shaker, A.F. and Kennedy, D.J.L. (1991). *The effective modulus of elasticity of concrete in tension*. Structural Engineering Report 172, Department of Civil Engineering, University of Alberta, Edmonton.

Figure H.1
Composite beam subject to shrinkage forces
(See Clause H.2.)



a) Shrinkage strain

b) Free-body diagram

Legend:

ε_f = free shrinkage strain of the concrete

ε_r = resulting restrained shrinkage strain

ε_t = compressive strain at top of steel beam

ε_b = tensile strain at bottom of steel beam

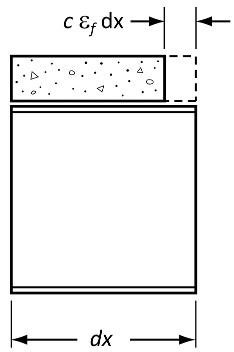
T = tensile force in concrete

C = compressive force in steel beam

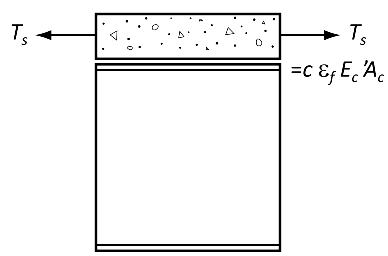
M = moment in steel beam required for equilibrium about reference axis

Figure H.2
Composite beam subject to shrinkage forces

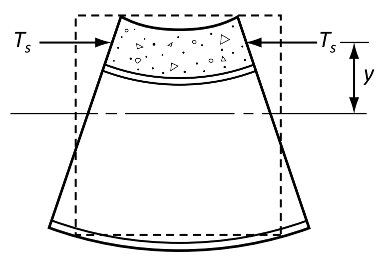
(See Clause H.3.)



a) Unrestrained shrinkage of concrete slab



b) Enforce compatibility



c) Satisfy equilibrium

Legend:

c = empirical coefficient used to match theory with test results, which may be taken as 0.5

T_s = tensile force applied at centroid of unrestrained slab

A_c = effective area of concrete slab (for metal deck spanning perpendicular to the beam, the concrete area is taken above the flutes, and for metal deck parallel to the beam, the full concrete area is taken)

y = distance from centroid of effective area of concrete slab to the centroidal axis of the composite steel beam

E_c' = age-adjusted effective modulus of elasticity of concrete

E_f = unrestrained shrinkage strain of the concrete slab

Annex I (informative)
Arbitration procedure for pretensioning connections

Note: *This informative Annex has been written in mandatory language to facilitate adoption by anyone wishing to do so.*

I.1 General

For pretensioned connections, when there is disagreement concerning the results of inspection of bolt pretensioning procedures, the following arbitration procedure shall be used unless an alternative has been specified:

a) The inspector shall use a manual or power torque inspection wrench capable of indicating a selected torque value.

b) Three bolts of the same grade and diameter as those under inspection and representative of the lengths and conditions of those in the structure shall be placed individually in a calibration device that indicates bolt tension. There shall be a washer under the part turned if washers are so used in the structure or, if no washer is used, the material abutting the part turned shall be of the same specification and condition as that in the structure.

c) When the inspection wrench is a manual wrench, each bolt specified in Item b) shall be pretensioned in the calibration device by any convenient means to an initial tension of approximately 15% of the required bolt tension and then to the minimum tension specified for its size in Table 7. Tightening beyond the initial condition shall not produce greater nut rotation than that permitted by Table 8. The inspection wrench shall then be applied to the tightened bolt, and the torque necessary to turn the nut or head an additional 5° shall be determined. The average torque measured in the tests of three bolts shall be taken as the job inspection torque to be used in the manner specified in Item e).

d) When the inspection wrench is a power wrench, it shall first be applied to produce an initial tension of approximately 15% of the required fastener tension and then adjusted so that it will tighten each bolt specified in Item b) to a tension of 5% to 10% greater than the minimum tension specified for its size in Table 7. This setting of the wrench shall be taken as the inspection torque to be used in the manner specified in Item e). Tightening beyond the initial condition shall not produce greater nut rotation than that permitted by Table 8.

e) Bolts represented by the sample prescribed in Item b) that have been tightened in the structure shall be inspected by applying, in the tightening direction, the inspection wrench and its job inspection torque to 10% of the bolts, but not less than two bolts, selected at random in each connection. If no nut or bolt head is turned by this application of the job inspection torque, the connection shall be accepted as properly tightened. If any nut or bolt head is turned by the application of the job inspection torque, this torque shall be applied to all bolts in the connection and all bolts whose nut or head is turned by the job inspection torque shall be tightened and re-inspected. Alternatively, the fabricator or erector may choose to retighten all the bolts in the connection and then resubmit the connection for the specified inspection.

Annex J (normative)
Qualification testing provisions for seismic moment connections and buckling restrained braces

Notes:

1) *Where the physical testing alternative as permitted in Clauses 27.2.5.1 b), 27.4.4.1 c), or 27.7.8.1 is chosen as the basis for design of these connections, this Annex serves as a normative (mandatory) part of the Standard.*

2) *This Annex serves as a normative (mandatory) part of the Standard to describe the qualification testing of buckling restrained braces prescribed in Clause 27.8.6.*

J.1 Seismic moment connections

J.1.1

Clause J.1 specifies testing protocols that aim to demonstrate the deformation and strength characteristics of moment-resisting connections in moment-resisting frames, eccentrically braced frames, and plate walls that permit the frames to achieve specified interstorey drift capacity when the connections are designed using the full-scale physical testing alternative as provided in Clause 27.2.5.1 b),Clause 27.4.4.1 c), or Clause 27.7.8.1.

J.1.2

Extensive physical testing and analytical studies conducted over the last two decades have advanced the knowledge of behaviour of several connection types now used for construction of ductile moment-resisting frames (ANSI/AISC 341 and CISC 2014). However, availability of physical test data for other connection types and configurations, and link-to-column connections used in eccentrically braced frames lags behind. Clause J.1 provides the requirements and guidance for such physical tests.

J.1.3

The test assemblies shall represent the size, detailing, and fabrication of the prototype, in recognition of the effects of size, bracing arrangements, welding details, and welding procedures on the inelastic cyclic behaviour of the connection type. The test loading shall represent both the deformation magnitude and cyclic nature expected in a severe seismic event. These tests shall comply with the requirements provided in Section K2 of ANSI/AISC 341, except that the criteria of acceptance as pertain to interstorey drift capacity shall comply with the appropriate clauses in this Standard. However, the provisions for welds and welding in accordance with CSA W59 and W48, instead of those in AWS D1.1 where referenced in ANSI/AISC 341, might apply. Existing test data for successful tests conducted in accordance with testing protocols as given in publications by the U.S. Applied Technology Council (ATC-24) and U.S. Federal Emergency Management Agency (FEMA350) shall nonetheless remain valid.

J.2 Buckling restrained braces

J.2.1

Clause J.2 specifies testing protocols for full-scale qualification testing of bracing members in buckling restrained braced frames, as prescribed in Clause 27.8.6.

Note: *The cyclic inelastic response of bracing members in buckling restrained braced frames heavily depends on the design, detail and fabrication of the buckling restrained braces. Physical testing is prescribed to demonstrate the cyclic inelastic performance of the members and obtain design values for the maximum tension and compression forces that are expected to develop in the buckling restrained member at maximum anticipated axial deformations.*

This Clause provides the requirements for such physical tests and references to test data. The information given in this Clause may be used for the testing of other bracing members designed to dissipate seismic input energy through nonlinear axial response.

J.2.2

The test specimens shall represent the size, detailing, and fabrication of the prototypes, in recognition of the effects of cross-section size, shape and orientation of the steel core and the material and method of separation between the steel core and the buckling restraining mechanism on the inelastic cyclic behaviour of the buckling restrained braces. The test loading shall represent both the deformation magnitude and cyclic nature expected in a severe seismic event. The tests shall comply with the requirements provided in Section K3 in Chapter K of ANSI/AISC 341, except that the criteria of acceptance as pertain to deformation capacity shall comply with Clause 27.8.6.

J.3 Bibliography

AISC. (2010). ANSI/AISC 341-10, *Seismic provisions for structural steel buildings,* American Institute of Steel Construction (AISC),Chicago, Illinois.

AISC. (2011). ANSI/AISC 358-10 and ANSI/AISC 358s1-11, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications,* including Supplement No. 1,American Institute of Steel Construction (AISC),Chicago, Illinois.

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Annex K (normative)
Structural design for fire conditions

Note: *This Annex is a normative (mandatory) part of the Standard.*

K.1 General

K.1.1 Scope

This Annex specifies criteria for the design and evaluation of structural steel components, systems and frames for fire conditions. These criteria provide for the determination of the heat input, thermal expansion, and degradation in mechanical properties of materials that cause progressive decreases in strength and stiffness of structural components and systems at elevated temperatures.

K.1.2 Definitions

This Annex uses the following terms in addition to the terms defined in Clause 2:

Active fire protection — building materials and systems that are activated by a fire to mitigate adverse effects or to notify people to take some action to mitigate adverse effects.

Convective heat transfer — the transfer of thermal energy from a point of higher temperature to a point of lower temperature through the motion of an intervening medium.

Design-basis fire — a set of conditions that define the development of a fire and the spread of combustion products throughout a building or portion thereof.

Elevated temperatures — heating conditions experienced by building elements or structures as a result of fire, which are in excess of the anticipated ambient conditions.

Fire — destructive burning, as manifested by one or more of light, flame, heat, or smoke.

Fire endurance — a measure of the elapsed time during which a material or assembly continues to exhibit fire resistance.

Fire resistance — the property of assemblies that prevents or retards the passage of excessive heat, hot gases, or flames under conditions of use and enables them to continue to perform a stipulated function.

Fire resistance rating — the period of time a building element, component, or assembly maintains the ability to contain a fire, continues to perform a given structural function, or both, as determined by test or methods based on tests.

Fire separation — a construction assembly that acts as a barrier against the spread of fire and whose construction is formed of fire-resisting materials and tested in accordance with CAN/ULC-S101, or another approved standard fire resistance test, to demonstrate compliance with requirements prescribed by the regulatory authority.

Flashover — the rapid transition to a state of total surface involvement in a fire of combustible materials within an enclosure.

Heat flux — radiant energy per unit surface area.

Heat release rate — the rate at which thermal energy is generated by a burning material.

Passive fire protection — building materials and systems whose ability to resist the effects of fire does not rely on any outside activating condition or mechanism.

Performance-based design or objective-based design — an engineering approach to structural design that is based on agreed-upon performance goals and objectives, engineering analysis, and quantitative assessment of alternatives against the performance goals and objectives using accepted engineering tools, methodologies, and performance criteria.

Prescriptive design — design methods, e.g., specific technical requirements or deemed-acceptable solutions that document specific compliance with general criteria established by the regulatory authority.

Restrained construction — floor and roof assemblies and individual beams in buildings where the surrounding or supporting structure is capable of resisting substantial thermal expansion throughout the range of anticipated elevated temperatures.

Unrestrained construction — floor and roof assemblies and individual beams in buildings that are assumed to be free to rotate and expand throughout the range of anticipated elevated temperatures.

K.1.3 Performance objectives

Structural components, members, and building frame systems shall be designed so as to maintain their load-bearing function during the design-basis fire and to satisfy other performance requirements specified for the building occupancy.

Deformation criteria shall be applied where the means of providing structural fire resistance or the design criteria for fire barriers require consideration of the deformation of the load-carrying structure.

Within the compartment of fire origin, forces and deformations from the design-basis fire shall not cause a breach of horizontal or vertical fire separation.

K.1.4 Design by engineering analysis

The analysis methods specified in Clause K.2 may be used to document the anticipated performance of steel framing when subjected to design-basis fire scenarios. These methods provide evidence of compliance with the performance objectives established in Clause K.1.3.

The analysis methods specified in Clause K.2 may be used to demonstrate an equivalency for an alternative material or method, as permitted by the regulatory authority.

K.1.5 Load combination and required resistance

The required resistance of the structure and its elements shall be determined based on the following gravity load combination specified in *User's Guide — NBC 2010: Structural Commentaries (Part 4)* Commentary A, Paragraph 25 ("Load Combination for Determination of Fire Resistance"):

$D + T_S + (\alpha L \text{ or } 0.25S)$

where

D = specified dead load, as given in Clause 6.2.1

T_S = effects due to expansion, contraction, or deflection caused by temperature changes due to the design-basis fire specified in Clause K.2.2. T_S can be taken equal to zero for statically determinate structures or for structures that have sufficient ductility to allow the redistribution of temperature forces before collapse

α = 1.0 for storage areas, equipment areas, and service rooms and 0.5 for other occupancies

L = specified occupancy live load, as given in Clause 6.2.1

S = specified variable load due to snow, as given in Clause 6.2.1

Notional lateral loads, in accordance with Clause 8.4.1, shall be applied in combination with this gravity load combination.

K.2 Structural design for fire conditions by analysis

K.2.1 General

Structural members, components, and building frames may be designed for elevated temperatures due to fire in accordance with this Clause.

K.2.2 Design-basis fire

K.2.2.1 General

A design-basis fire shall be identified to describe the heating conditions for the structure. These heating conditions shall relate to the fuel commodities and compartment characteristics present in the assumed fire area. The fuel load density based on the occupancy of the space shall be considered when determining the total fuel load. Heating conditions shall be specified in terms of a heat flux or temperature of the upper gas layer created by the fire. The variation of the heating conditions with time shall be determined for the duration of the fire.

When the analysis methods specified in Clause K.2 are used to demonstrate an equivalency as an alternative material or method as permitted by the regulatory authority, the design-basis fire shall be determined in accordance with CAN/ULC-S101.

K.2.2.2 Localized fire

Where the heat release rate from the fire is insufficient to cause flashover, a localized fire exposure shall be assumed. In such cases, the fuel composition, arrangement of the fuel array, and floor area occupied by the fuel shall be used to determine the radiant heat flux from the flame and smoke plume to the structure.

K.2.2.3 Post-flashover compartment fires

Where the heat release rate from the fire is sufficient to cause flashover, a post-flashover compartment fire shall be assumed. The determination of the temperature versus time profile resulting from the fire shall include fuel load, ventilation characteristics to the space (natural and mechanical), compartment dimensions, and thermal characteristics of the compartment boundary.

K.2.2.4 Exterior fires

The exposure of exterior structure to flames projecting from windows or other wall openings as a result of a post-flashover compartment fire shall be considered along with the radiation from the interior fire through the opening. The shape and length of the flame projection shall be used along with the distance between the flame and the exterior steelwork to determine the heat flux to the steel. The method specified in Clause K.2.2.3 shall be used for describing the characteristics of the interior compartment fire.

K.2.2.5 Fire duration

The fire duration in a particular area shall be determined by considering the total combustible mass, i.e., fuel load, available in the space. In the case of a localized fire or post-flashover compartment fire, the time duration shall be determined as the total combustible mass divided by the mass loss rate, except where determined from Clause K.2.2.3.

K.2.2.6 Active fire protection systems

The effects of active fire protection systems shall be considered when describing the design-basis fire.

Where automatic smoke and heat vents are installed in non-sprinklered spaces, the resulting smoke temperature shall be determined from calculation.

K.2.3 Temperatures in structural systems under fire conditions

Temperatures within structural members, components, and frames due to the heating conditions posed by the design-basis fire shall be determined by a heat transfer analysis.

K.2.4 Material properties at elevated temperatures

K.2.4.1 General

Material properties at elevated temperatures shall be determined from test data. In the absence of such data, the material properties specified in Clause K.2.4 may be used. These reduction factors shall not apply to steels with a yield strength in excess of 450 MPa or concretes with specified compression strength in excess of 55 MPa.

K.2.4.2 Thermal elongation

The following thermal elongation requirements shall apply:

a) Thermal expansion of structural and reinforcing steels: for calculations at temperatures above 65 °C, the coefficient of thermal expansion shall be 1.4×10^{-5}/°C.
b) Thermal expansion of normal weight concrete (NWC): for calculations at temperatures above 65 °C, the coefficient of thermal expansion shall be 1.8×10^{-5}/°C.
c) Thermal expansion of lightweight concrete (LWC): for calculations at temperatures above 65 °C, the coefficient of thermal expansion shall be 7.9×10^{-6}/°C.

K.2.4.3 Mechanical properties at elevated temperatures

The deterioration in strength and stiffness of structural members, components, and systems shall be taken into account in the structural analysis of the frame. The values F_{ym}, F_{pm}, F_{um}, E_m, f'_{cm}, E_{cm}, ε_{cu}, F_{ubm}, and F_{sbm} at elevated temperature to be used in structural analysis, expressed as the ratio with respect to the property at ambient temperature (assumed to be 20 °C), shall be as specified in Tables K.1, K.2, and K.3. Interpolation between these values may be used. Table K.1 specifies the reduction factors for the stress-strain relationship for steel at the elevated temperatures shown in Figure K.1.

K.2.5 Structural design

K.2.5.1 General structural integrity

The structural frame shall be capable of providing adequate strength and deformation capacity to withstand, as a system, the structural actions developed during the fire within the prescribed limits of deformation. The structural system shall be designed to sustain local damage, with the structural system as a whole remaining stable.

Continuous load paths shall be provided to transfer all forces from the exposed region to the final point of resistance. The foundation shall be designed to resist the forces and to accommodate the deformations developed during the design-basis fire.

K.2.5.2 Strength requirements and deformation limits

Conformance of the structural system to the requirements of this Annex shall be demonstrated by constructing a mathematical model of the structure based on principles of structural mechanics and evaluating this model for the internal forces and deformations in the members of the structure developed by the temperatures from the design-basis fire.

Individual members shall be provided with adequate strength to resist the shears, axial forces, and moments determined in accordance with this Annex.

Connections shall develop the strength of the connected members or the forces specified in this Clause. Where the means of providing fire resistance necessitates consideration of deformation criteria, the deformation of the structural system, or members thereof, under the design-basis fire shall not exceed the prescribed limits.

K.2.5.3 Methods of analysis

K.2.5.3.1 Advanced methods of analysis

The advanced methods of analysis may be used for the design of steel building structures for fire conditions. The design-basis fire exposure shall be that determined in accordance with Clause K.2.2. The analysis shall include both a thermal response and the mechanical response to the design-basis fire.

The thermal response shall produce a temperature field in each structural element as a result of the design-basis fire and shall incorporate temperature-dependent thermal properties of the structural elements and fire-resistive materials in accordance with Clause K.2.3.

The mechanical response results in forces and deflections in the structural system subjected to the thermal response calculated from the design-basis fire. The mechanical response shall explicitly take into account the deterioration in strength and stiffness with increasing temperature, the effects of thermal expansions, and large deformations. Boundary conditions and connection fixity shall represent the proposed structural design. The material properties shall be as specified in Clause K.2.4.

The resulting analysis shall consider all relevant limit states, e.g., excessive deflections, connection fractures, and overall or local buckling.

K.2.5.3.2 Simple methods of analysis

The simple methods of analysis specified in this Clause are applicable to the evaluation of the performance of individual members at elevated temperatures during exposure to fire.

The support and restraint conditions (forces, moments, and boundary conditions) applicable at normal temperatures may be assumed to remain unchanged throughout the fire exposure.

The thermal response may be modeled using a one-dimensional heat transfer equation with heat input as directed by the design-basis fire specified in Clause K.2.2. The maximum steel temperature, T, obtained from this analysis shall be assumed constant through the member cross-section and shall be used to determine the factored resistances of the members in Items a) to f) as follows:

a) Tension members: the factored resistance of a tension member shall be determined as specified in Clause 13.2, using steel properties as specified in Clause K.2.4, with the temperature equal to the maximum steel temperature.

b) Compression members: the factored resistance of a compression member shall be determined as specified in Clause 13.3 using steel properties as specified in Clause K.2.4; however, for steel temperatures equal to or greater than 200 °C, the factored compressive resistance for flexural buckling shall be determined as follows:

$C_r(T) = (1+\lambda(T)^{2dn})^{-1/dn} A F_y(T)$

where

$C_r(T)$ = the factored compressive resistance at temperature, T

$$\lambda(T) = \frac{KL}{r}\sqrt{\frac{F_y(T)}{\pi^2 E(T)}} = \sqrt{\frac{F_y(T)}{F_e(T)}}$$

d = 0.6

n = as specified in Clause 13.3.1

c) Flexural members: the factored shear and moment resistance of a flexural member shall be as specified in Clauses 13.4 to 13.6 using steel properties specified in Clause K.2.4; however, for steel temperatures equal to or greater than 200 °C, the bending strength for lateral-torsional buckling of laterally unsupported doubly-symmetric members shall be determined as follows:

$$M_r(T) = C_K M_p(T) + (1 - C_K) M_p(T)\left(1 - \left(\frac{C_K M_p(T)}{M_u(T)}\right)^{0.5}\right)^{C_z(T)}$$

where

C_K = 0.12

$M_p(T)$ = the plastic moment at elevated temperatures determined using $F_y(T)$

$M_u(T)$ = the elastic critical load at elevated temperatures, determined as follows:

$$M_u(T) = \frac{\omega_2 \pi}{L}\sqrt{E(T) I_y G(T) J + I_y C_w \left(\frac{\pi E(T)}{L}\right)^2}$$

where

ω_2 = as defined in Clause 13.6

$$C_z(T) = \frac{T + 800}{500} \le 2.4$$

d) Combined axial force and flexure: the factored resistance of a member required to resist both bending moments and an axial tensile or compression forces shall be determined as specified in Clauses 13.8 and 13.9 using steel properties specified in Clause K.2.4 and flexural and axial strengths as specified in Clause K.2.5.3.2 a) to c).

e) Composite floor members: the thermal response of flexural elements supporting a concrete slab may be modelled using a one-dimensional heat transfer equation to calculate the maximum temperature of the bottom flange of the steel section. This temperature shall be taken as constant between the bottom flange to the mid-depth of the web and shall decrease linearly from the mid-depth of the web to the top flange of the steel beam by no more than 25%.
The factored resistance of a composite flexural member shall be determined as specified in Clause 17 using steel properties specified in Clause K.2.4.

f) Other components and connections: the factored resistance of other components and connections shall be as specified in Clause 13. Factored resistances shall be calculated using steel properties specified in Clause K.2.4 at the maximum temperature determined by the design-basis fire.

K.3 Bibliography

European Committee for Standardization
EN 1992-1-2:2004
Eurocode 2: Design of concrete structures — Part 1-2: General rules — Structural fire design

EN 1993-1-2:2005
Eurocode 3: Design of steel structures — Part 1-2: General rules — Structural fire design

EN 1994-1-2:2005
Eurocode 4: Design of composite steel and concrete structures — Part 1-2: General rules — Structural fire design

Other publications

Takagi, J., and Deierlein, G. 2009. *Proposed design equations for CAN/CSA-S16* Annex K *provisions for steel members at high temperatures*. Report prepared for the Canadian Institute of Steel Construction, Markham, ON.

Table K.1
Reduction factors for stress-strain relationship of steel at elevated temperatures (Eurocode 3 and Eurocode 4)
(See Clause K.2.4.3.)

Steel temperature, T_{steel}, °C	Reduction factors at temperature, T_{steel}, relative to the value of F_y or E at 20 °C			
	Reduction factor (relative to E) for the slope of the linear elastic range, $k_E = E_m/E$	Reduction factor (relative to F_y) for proportional limit, $k_p = F_{pm}/F_y$	Reduction factor (relative to F_y) for effective yield strength, $k_y = F_{ym}/F_y$	Reduction factor (relative to F_y) for effective tensile strength, $k_u = F_{um}/F_y$
20	1.00	1.00	1.00	1.25
100	1.00	1.00	1.00	1.25
200	0.90	0.807	1.00	1.25
300	0.80	0.613	1.00	1.25
400	0.70	0.420	1.00	1.00
500	0.60	0.360	0.78	0.78
600	0.31	0.180	0.47	0.47
700	0.13	0.075	0.23	0.23
800	0.09	0.050	0.11	0.11
900	0.0675	0.0375	0.06	0.06
1000	0.0450	0.0250	0.04	0.04
1100	0.0225	0.0125	0.02	0.02
1200	0.00	0.00	0.00	0.00

Legend:

E = elastic modulus of steel (200 000 MPa assumed; earthquake loads and effects)

E_m = slope of the linear elastic range for steel at elevated temperature T_{steel}

F_{pm} = proportional limit for steel at elevated temperature T_{steel}

F_{um} = effective tensile strength of steel at elevated temperature T_{steel}

F_y = specified minimum yield stress, yield point, or yield strength

F_{ym} = effective yield strength of steel at elevated temperature T_{steel}

k_E = slope of linear elastic range, relative to slope at 20 °C

k_p = proportional limit, relative to yield strength at 20 °C

k_u = effective tensile strength, relative to yield strength at 20 °C

k_y = effective yield strength, relative to yield strength at 20 °C

Table K.2
Values for the main parameters of the stress-strain relationships of normal weight concrete (NWC) and lightweight concrete (LWC) at elevated temperatures (Eurocode 2 and Eurocode 4)

(See Clause K.2.4.3.)

Concrete temperature, $T_{concrete}$, °C	Reduction factor (relative to f'_c) for effective compressive strength, $k_c = f'_{cm}/f'_c$		E_{cm}/E_c	ε_{cu}, %
	NWC	LWC		NWC
20	1.00	1.00	1.00	0.25
100	1.00	1.00	0.92	0.40
200	0.95	1.00	0.75	0.55
300	0.85	1.00	0.59	0.70
400	0.75	0.88	0.43	1.00
500	0.60	0.76	0.26	1.50
600	0.45	0.64	0.10	2.50
700	0.30	0.52	0.083	2.50
800	0.15	0.40	0.067	2.50
900	0.08	0.28	0.050	2.50
1000	0.04	0.16	0.033	2.50
1100	0.01	0.04	0.017	2.50
1200	0.00	0.00	0.00	—

Legend:

E_c = elastic modulus of concrete

E_{cm} = tangent modulus of the stress-strain relationship of the concrete at elevated temperature $T_{concrete}$

f'_cf = specified compressive strength of concrete at 28 days

f'_{cm} = effective value for the compressive strength of concrete at elevated temperature $T_{concrete}$

k_c = effective compressive strength relative to compressive strength at 20 °C

ε_{cu} = concrete strain corresponding to f'_{cm}

Note: *For LWC, values of ε_{cu} shall be obtained from tests.*

Strain range	Stress, σ	Tangent modulus
$\varepsilon \le \varepsilon_{pm}$	εE_m	E_m
$\varepsilon_{pm} < \varepsilon < \varepsilon_{ym}$	$F_{pm} - c + (b/a)\,[a^2 - (\varepsilon_{ym} - \varepsilon)^2]^{0.5}$	$\dfrac{b(\varepsilon_{ym} - \varepsilon)}{a\left[a^2 - (\varepsilon_{ym} - \varepsilon)^2\right]^{0.5}}$
$\varepsilon_{ym} \le \varepsilon \le \varepsilon_{tm}$	F_{ym}	0

(Continued)

(Concluded)

Strain range	Stress, σ	Tangent modulus
$\varepsilon_{tm} \le \varepsilon \le \varepsilon_{um}$	$F_{ym}\,[1-(\varepsilon-\varepsilon_{tm})/(\varepsilon_{um}-\varepsilon_{tm})]$	—
$\varepsilon = \varepsilon_{um}$	0.00	—

Notes:

1) *Parameters:*
- a) $\varepsilon_{pm} = F_{pm}/E_m$
- b) $\varepsilon_{ym} = 0.02$
- c) $\varepsilon_{tm} = 0.15$
- d) $\varepsilon_{um} = 0.20$

2) *Functions:*
- a) $a^2 = (\varepsilon_{ym}-\varepsilon_{pm})\left(\varepsilon_{ym}-\varepsilon_{pm}+c/E_m\right)$
- b) $b^2 = c(\varepsilon_{ym}-\varepsilon_{pm})E_m + c^2$
- c) $c = \dfrac{\left(F_{ym}-F_{pm}\right)^2}{\left(\varepsilon_{ym}-\varepsilon_{pm}\right)E_m - 2\left(F_{ym}-F_{pm}\right)}$

Table K.3
Properties of A325M/A325 and A490M/A490 high strength bolts at elevated temperatures

(See Clause K.2.4.3.)

Steel temperature, T_{bolt}, °C	F_{ubm} / F_{ub} **or** F_{sbm} / F_{sb}
20	1.00
100	0.97
200	0.93
300	0.89
400	0.75
500	0.54
600	0.27
700	0.12
800	0.07
900	0.03
1000	0.03
1100	0.00

Legend:

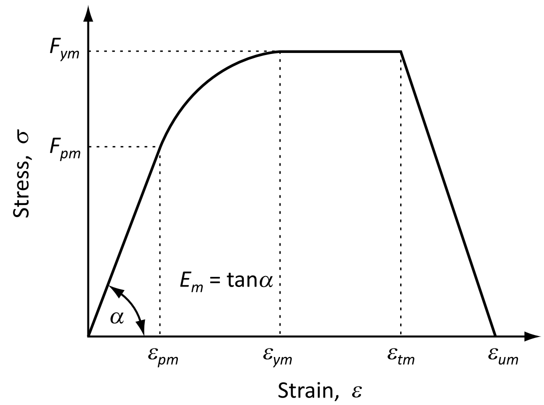
F_{ubm} = effective tensile strength of bolt at elevated temperature

F_{ub} = effective tensile strength of bolt

F_{sbm} = effective shear strength of bolt at elevated temperature

F_{sb} = effective shear strength of bolt

Figure K.1
Stress-strain relationship for steel at elevated temperatures (Eurocode 3)
(See Clause K.2.4.3.)



Legend:
E_m = slope of the linear elastic range
F_{pm} = proportional limit
F_{ym} = effective yield strength
ε_{pm} = strain at proportional limit
ε_{tm} = limiting strain for yield strength
ε_{um} = ultimate strain
ε_{ym} = yield strain

Annex L (informative)
Design to prevent brittle fracture

Note: *This Annex is an informative (non-mandatory) part of this Standard.*

L.1 General

Brittle fracture is a fracture mechanism accompanied by limited or no plastic deformation. Consequently, it is sudden and occurs with little to no warning, which makes it an undesirable failure mode that should be avoided by the adoption of a fracture control plan.

The design guidelines presented in this Annex are applicable to members and structural components subjected to tensile stresses arising from direct tension or bending when the rate of applied loading is high, e.g., dynamic or impact loading. Members and connections that contain notches, fabrication discontinuities, or other stress raisers need particular attention. Structures that are exposed to low temperatures are more susceptible to brittle fracture than those that are not. Although relatively uncommon, brittle fracture can also occur at normal temperatures, when fracture-sensitive details or metals (base or weld) with low notch toughness are subjected to dynamic tensile stresses.

The protected zones of seismically loaded structures should be designed to control brittle fracture.

When plates or heavy rolled sections are subjected to tensile stresses in the through-thickness direction, additional consideration needs to be given to the selection of the steel quality (see Clause L.3).

Statically loaded structures that are subjected to low temperature do not normally require the use of notch-tough steel. In these structures, brittle fracture can normally be avoided by following the design and fabrication criteria provided in this Standard and CSA W59. Special attention should be paid to anchor rods (see ASTM F1554) special loading conditions during construction, use of thick steel plates, fabrication procedures that give rise to high tensile residual stresses, and details that give rise to stress concentrations.

Designers should be aware that the availability of notch-tough steel is somewhat limited (see Clause L.3).

L.2 Material selection

The potential for brittle fracture depends mainly on the following factors (Barsom and Rolfe, 1999):

a) steel strength;
b) material thickness;
c) loading rate;
d) minimum service temperature;
e) material toughness; and
f) type of structural element.

These factors should be considered when selecting steel with appropriate notch toughness (Barsom, 1975; Barsom, 2002). Connection details and the presence of stress raisers also needs to be considered. Required notch toughness is expressed in terms of the test temperature at which the Charpy V-notch energy has a minimum value of 20 J or 27 J or 34 J, as specified in CSA G40.21.

The approach presented in Tables L.1 to L.4 consists of defining the Charpy V-notch energy level and the testing temperature for four different service temperature ranges. Figure L.1 can be used to determine the minimum service temperature appropriate for structural steel exposed to outdoor conditions. The testing temperature can be significantly different from the service temperature to account for the difference in strain rate between the Charpy impact test and the strain rate applied to the structure.

Dynamic loading and impact loading are recognized in Tables L.1 to L.4. Dynamic loading is applicable to intermediate strain rates such as those occurring in structures subjected to seismic ground motions, wave loads, or wind-induced vibration or truck traffic loading on highway bridges. The strain rates for such applications are typically around 10^{-3} sec^{-1}. Impact loading is applicable to high strain rates that occur, for example in explosive, crash conditions, or impact forces when large weights are dropped on structures. The strain rates for impact conditions are around 10 sec^{-1}. The selection of the applicable toughness level should be based on a judicious evaluation of the applicable strain rate.

When specifying steel for a specific application, the engineer should consider the probability of low temperature and extreme loading conditions occurring simultaneously. Care should also be taken not to specify excessively high fracture toughness since specification of higher than necessary fracture toughness can cause delays with sourcing the material.

The consequence of brittle fracture is recognized in the material selection. Fracture-critical members or joints are those for which local failure would cause complete structural collapse with serious consequences to life or very high cost. Primary tension members (tension and bending members or joints) are those for which failure would be restricted to localized areas not resulting in structural collapse. The fracture toughness of secondary framing members need not be considered.

The impact energy for the weld metal needs to be higher than for the base metal because welds usually have discontinuities, stress raisers, and high tensile residual stresses, which make weld metals more susceptible to brittle fracture. Given that the cost of weld metals is small relative to that of the structure, it is good practice to specify high-toughness filler metal to lower the risk of brittle fracture.

Table L.1
Recommended test temperatures and Charpy V-notch impact test values for primary tension members under dynamic loading

(See Clause L.2.)

Steel Grade	Base metal test temperature, °C, for minimum service temperature, T_s, °C				Weld metal test temperature, °C, for minimum service temperature, T_s, °C		
	Minimum average energy, J	$T_s > 0$	0 $T_s > –30$	$-30 > T_s >$ – 52	Minimum average energy, J	$T_s > –40$	$-30 > T_s >$ – 52
260 WT	20	20	0	–20	20	–30	–30
300 WT	20	20	0	–20	20	–30	–30
350 WT and AT	27	20	0	–20	27	–30	–30
400 WT and AT	27	20	0	–20	27	–30	–30
480 WT and AT	27	10	–10	–30	27	–30	–40
700 QT	48	0	–20	–35	48	–30	–40

Table L.2
Recommended test temperatures and Charpy V-notch impact test values for primary tension members under impact loading

(See Clause L.2.)

Steel Grade	Base metal test temperature, °C, for minimum service temperature, T_s, °C				Weld metal test temperature, °C, for minimum service temperature, T_s, °C		
	Minimum average energy, J	$T_s > 0$	0 $T_s > –30$	$–30 > T_s >$ – 52	Minimum average energy, J	$T_s > -40$	$–30 > T_s > –52$
260 WT	20	–30	–50	–70	20	–65	–75
300 WT	20	–25	–45	–65	20	–60	–70
350 WT and AT	27	–20	–40	–60	27	–55	–65
400 WT and AT	27	–20	–40	–60	27	–55	–65
480 WT and AT	27	–20	–40	–60	27	–55	–65
700 QT	48	–10	–30	–40	48	–40	–45

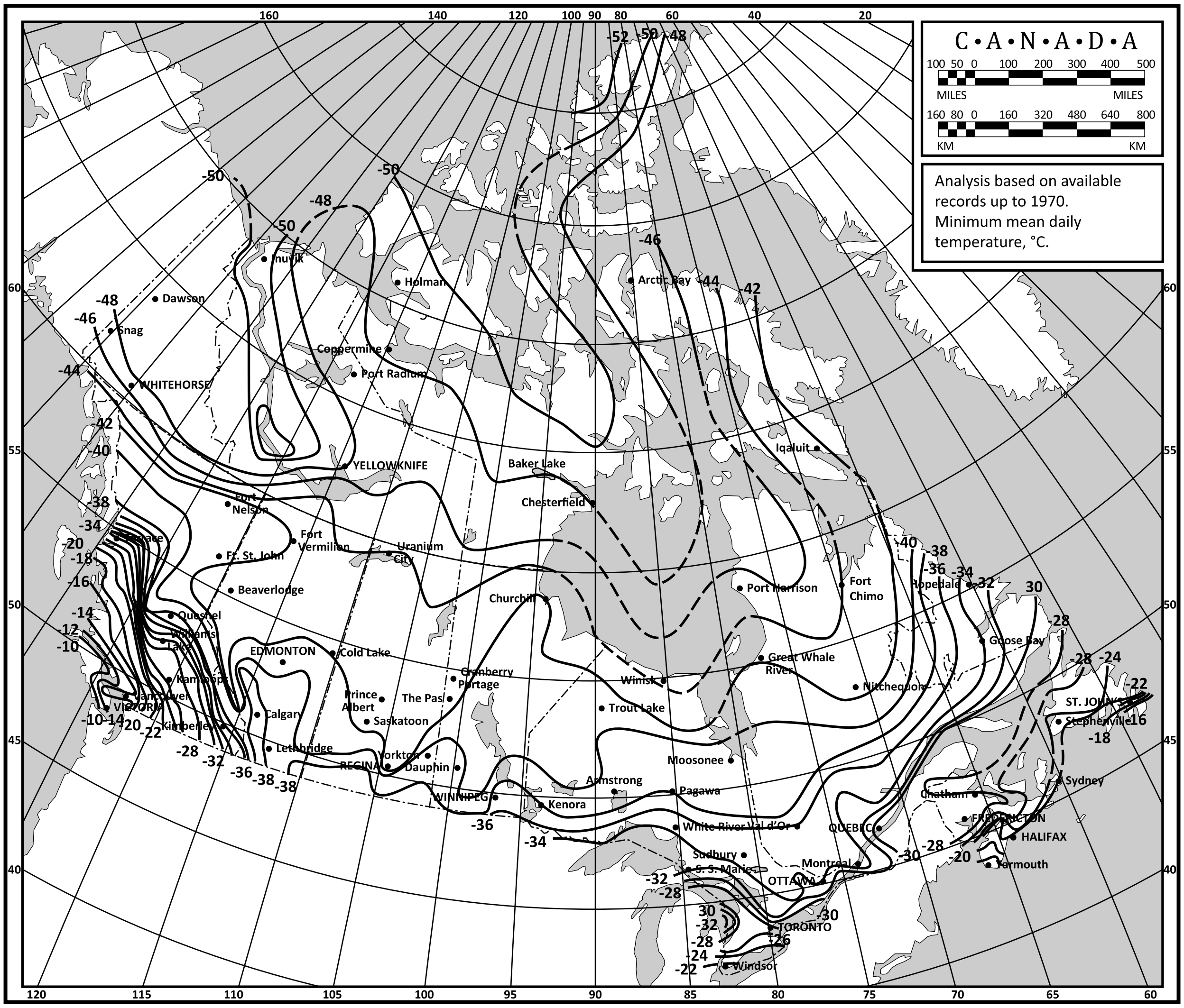
Table L.3
Recommended test temperatures and Charpy V-notch impact test values for fracture critical members under dynamic loading
(See Clause L.2.)

Steel Grade	Base metal test temperature, °C, for minimum service temperature, T_s, °C				Weld metal test temperature, °C, for minimum service temperature, T_s, °C		
	Minimum average energy, J	$T_s > 0$	$0\ T_s > -30$	$-30 > T_s > -52$	Minimum average energy, J	$T_s > -40$	$-30 > T_s > -52$
260 WT	20	0	–20	–30	20	–30	–30
300 WT	20	0	–20	–30	20	–30	–30
350 WT and AT	27	0	–20	–30	27	–30	–30
400 WT and AT	27	0	–20	–30	27	–30	–40
480 WT and AT	27	0	–20	–40	27	–40	–50
700 QT	Not permitted						

Table L.4
Recommended test temperatures and Charpy V-notch impact test values for fracture critical members under impact loading
(See Clause L.2.)

Steel Grade	Base metal test temperature, °C, for minimum service temperature, T_s, °C				Weld metal test temperature, oC, for minimum service temperature, T_s, °C		
	Minimum average energy, J	$T_s > 0$	$0\ T_s > -30$	$-30 > T_s > -52$	Minimum average energy, J	$T_s > -40$	$-30 > T_s > -52$
260 WT	20	–45	–65	–70	20	–70	–70
300 WT	20	–45	–60	–70	20	–70	–70
350 WT and AT	27	–35	–50	–70	27	–70	–70
400 WT and AT	27	–35	–50	–65	27	–70	–70
480 WT and AT	27	–30	–45	–65	27	–70	–70
700 QT	Not permitted						

Figure L.1
Minimum mean daily temperatures
(See Clause L.2.)



L.3 Steel availability

When selecting materials for structures at risk of brittle fracture, it is recommended to consult with suppliers regarding the availability of notch-tough materials. In general for Grade WT material, plates are more readily available than rolled shapes.

The inherent toughness of Grade W steel is often sufficient to prevent brittle fracture at low temperatures, although the required toughness is not guaranteed. CSA G40.21 type W steel may be substituted for CSA G40.21 type WT steel only when the Charpy impact energy requirements are verified by the submission of test documentation.

L.4 Control of discontinuities

In addition to specifying materials with appropriate toughness, the control of discontinuities is equally important to provide an acceptably low probability of brittle fracture.

Discontinuities can result from deliberate changes in geometry from the design or unintentional such as flaws in welded connections. Although it is impossible to avoid all flaws in welded construction, Welded structures should meet the requirements of CSA W59 for acceptable size of discontinuities.

Connection details should be designed to minimize stress raisers such as sharp corners and abrupt changes of stiffness resulting from changes in cross-section. Thick or high-strength materials are generally more susceptible to cold cracking in the heat-affected zones of welds and in areas of high residual stresses. In such cases, the choice of appropriate welding procedures is as important as the selection of the material.

When selecting construction details, the designer needs to account for the fact that some materials cannot be welded, or can be welded only under strict conditions:

a) Prestressing steels, anchor rods, and high-strength bolts cannot be welded.
b) High-carbon steels can be welded only under specific conditions. For welding of reinforcing steel, see CAN/CSA-G30.18.

L.5 Bibliography

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Annex M (informative)
Seismic design of industrial steel structures

Note: *This informative (non-mandatory) Annex has been written in normative (mandatory) language to facilitate adoption where users of the Standard or regulatory authorities wish to adopt it formally as additional requirements to this Standard.*

M.1 General

M.1.1
This Annex applies to industrial type structures that are expected not to respond to seismic ground motions in a fashion similar to conventional buildings because of non-uniform distribution of mass, strength and stiffness in the building, absence of clearly defined floors, or reduced damping due to limited architectural components. The intended use of these structures are essentially to support equipment and material for an industrial process that may significantly affect the structure seismic response, and do not include the shelter of persons. These provisions do not apply to warehouses or to office buildings for industrial complexes. These provisions do not apply to nuclear facilities.

M.1.2
All requirements in this Standard shall apply except as otherwise specified in this Annex.

M.2 Seismic force resisting systems

M.2.1 System restrictions
The seismic force resisting system shall be chosen from Table M.1.

M.2.2 System redundancy
When $I_E F_a S_a(0.2)$ is greater than 0.45 or $I_E F_a S_a(1.0)$ is greater than 0.30, structures that exceed 40 m in height and that are designed to resist seismic loads based on a ductility-related force modification factor, R_d, greater than 1.5 shall be either

a) configured in such a way that failure of any brace or brace connection, or any rigid beam-to-column joint, does not increase earthquake effects by more than 33% in the remaining members of the seismic force resisting system; or

b) designed to resist gravity loads combined with 1.3 times the seismic loads.

M.3 Analysis

M.3.1 Methods of analysis
Analysis for design earthquake actions shall be carried out in accordance with the Dynamic Analysis Procedure described in the *NBCC*, except that Sentences of 4.1.8.12.(6), 4.1.8.12.(8), 4.1.8.12.(9) and 4.1.8.12.(11) do not apply.

M.3.2 Damping coefficient
The design spectrum S(T) in the *NBCC* shall be multiplied by the damping coefficient $\beta = (0.05/\xi)^{0.4}$, where $\xi = 0.02$ for welded structures and 0.03 for bolted structures.

M.3.3 Effective mass
The effective mass corresponds to the seismic weight as defined in the *NBCC*, including the mass of the operational contents of tanks, vessels, bins, hoppers, piping, and other similar equipment.

When determining vertical earthquake effects, the effective mass must include 100% of the mass resulting from the probable accumulation of equipment and storage of materials.

The design of the supporting tower or pedestal, anchorage, and foundation for seismic overturning shall assume the material stored is a rigid mass positioned at its centre of gravity. The effects of fluid-structure interaction may be considered in determining the forces, effective period, and mass centroids of the system if the following requirements are met:
a) the sloshing period, T_c is greater than $3T$ where T = natural period of the tank with confined liquid (rigid mass) and supporting structure; and
b) the sloshing mechanism (i.e., the percentage of convective mass and centroid) is determined for the specific configuration of the container by detailed fluid-structure interaction analysis or testing.

M.3.4 Number of modes
When the modal response spectrum analysis method is used, the number of modes in each direction of analysis shall be sufficient to accurately represent the seismic dynamic response of the structure. As a minimum, the combined participating mass of all the modes included in the analysis should total at least 95% of the total mass.

M.3.5 Direction of analysis
When $I_E F_a S_a(0.2)$ is greater than 0.35 or $I_E F_v S_a(1.0)$ is greater than 0.30, columns that form part of two or more intersecting seismic force resisting systems shall be designed to resist 100% of the earthquake effects from the seismic loads applied in one direction plus 30% of the earthquake effects from seismic loads applied in the perpendicular direction, with the combination requiring the greater element strength being used in the design.

M.3.6 Vertical earthquake effects
When $I_E F_a S_a(0.2)$ is greater than 0.35 or $I_E F_v S_a(1.0)$ is greater than 0.30, vertical earthquake effects shall be considered for
a) bars and rods supporting hanging equipment, including their supports, and steel beams when the dead load of the equipment represents more than 75% of the total tributary gravity loads;
b) foundations, anchors, and bearings for structures and equipment; and
c) structures that can be affected by vertical seismic motion such as cantilevered structures.

Vertical earthquake effects shall be determined using the modal response spectrum analysis method with 2/3 the design spectrum S(T) in the *NBCC*, as multiplied by the damping coefficient defined in Clause M.2.2,and $R_d R_o$= 1.3.

M.4 Anchorage

M.4.1 Anchorage strength
Anchor rods shall be designed for earthquake forces corresponding to $R_d R_o$ = 1.3, without exceeding forces corresponding to the probable resistance of the connected components determined using the probable yield stress $R_y F_y$.

M.4.2 Anchorage detailing

Where anchor rods are not threaded over their full length, they shall

a) be detailed such that $A_{aru}F_y \le 0.75\ A_{ar}F_u$, where A_{aru} is the cross-section area of the unthreaded portion;
 and

b) have a minimum length of 75 mm of thread is left under the nut.

For anchorage of towers, chimneys, or cantilevered structures designed with R_d equal to or greater than 1.5, or anchorage of fixed base columns of structures designed with R_d equal to or greater than 1.5, the anchor rods shall have a stretch length sufficient to accommodate the expected inelastic elongation, but not less than the larger of 250 mm and 8 times the diameter of the anchor rod.

M.5 Special requirements

M.5.1
Design of columns in braced frames

When two orthogonal braced bays share a column, the column shall be design for the forces resulting from the braces in both orthogonal directions reaching their probable resistances simultaneously.

M.5.2 Tanks and vessels supported by buildings

Tanks and vessels that are supported within buildings or are incidental to the primary function of the tower are considered mechanical equipment and shall be designed in accordance with the *NBCC* Chapter 4.

M.5.3 Welded steel water storage structures

Welded steel elevated water storage structures shall be designed and detailed in accordance with the seismic requirements of AWWA D100 with the height limits imposed in Table M.1.

Table M.1
Seismic force resisting systems

Structure type	Clause*	Restrictions†				
		Cases where $I_E F_a S_a(0.2)$				Cases where $I_E F_v S_a(1.0)$
		< 0.2	≥ 0.2 to < 0.35	≥ 0.35 to ≤ 0.75	> 0.75	> 0.3
Elevated tanks, vessels, bins, or hoppers supported on symmetrically braced legs	27.5	NL	NL	60	60	60
Elevated tanks, vessels, bins or hoppers supported on asymmetrically braced legs	27.6	NL	NL	60	60	60
Elevated tanks, vessels, bins or hoppers supported on a single pedestal or skirt	27.11	NL	NL	60‡	60‡	60‡
Horizontal saddle supported welded steel vessels	27.5	NL	NL	NL	NL	NL
All other distributed mass cantilever structures not covered above including stacks, chimneys, silos and skirt supported vertical vessels	27.11	NL	NL	NL‡	NL‡	NL‡
Trussed towers (free standing or guyed)	27.6	NL	NL	NL	NL	NL
Guyed stacks and chimneys	27.6	NL	NL	NL	NL	NL
Cooling towers	27.5	NL	NL	60	60	60
	27.11	NL	NL	NL‡	NL‡	NL‡
Pole towers	27.11	NL	NL	NL‡	NL‡	NL‡
All other self-supporting structures, tanks or vessels not covered above		NL	NL	NL	NL	NL

* *Structures shall meet the requirements of this Clause, including the applicable factors R_d and R_o.*
† *NP = system is not permittedNL = system is permitted and not limited in height as an SFRS. Numbers in this Table are maximum height limits in m. The most stringent requirement governs.*
‡ *Earthquake effects shall be determined with R_dR_o = 1.0 when the height exceeds* 40 m.

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