Part 1, Appendix A: Detailed Summary of Reinforced Concrete Punctured Shear Wall Studies

A.1 Introduction

As described in Part 1, Chapter 1, the Working Group 1, Linear effort focused on refinements and improvements to the linear procedure limitation provision ASCE/SEI 41. This involved a number of case study efforts. The main effort focused on studies involving a reinforced concrete shear wall building. This Appendix A1 provides a more detailed summary of the punctured reinforced concrete shear wall building case studies. It includes a description of the prototype buildings, the linear and nonlinear models, and results from investigation of the weak story irregularity limitation of ASCE/SEI 41-17 Section 7.3.1.1 which prohibits use of linear analysis methods if there is a weak story irregularity and if the DCR of any wall pier exceeds the lesser of 3.0 and the *m*-factor for the component action. There are three key questions:

- How closely do the results of the linear analyses match those of the nonlinear analyses?
- Does the limitation provision appropriately prevent a situation where the linear results would be overly unconservative?
- How do the results from ASCE/SEI 41-17 compare with ASCE/SEI 7-16 (ASCE, 2017)?

Conclusions are provided, as well as proposed next steps.

A.2 Reinforced Concrete Shear Wall Building Description

A.2.1 Source of Building Geometry: SEAW and FEMA P-2006 Case Studies

The existing building that was selected for this investigation is a reinforced concrete shear wall building built in the 1960s and designed in accordance with the 1961 Uniform Building Code (ICBO, 1961). It has been modified from the concrete shear wall design example in FEMA P-2006, *Example Application Guide for ASCE/SEI 41-13 Seismic Evaluation and Retrofit of Existing Buildings with Additional Commentary for ASCE/SEI 41-17* (FEMA, 2018). The FEMA P-2006 example was adapted from a presentation for the Structural Engineers Association of Washington (SEAW).

A.2.2 Overall Building Description and Geometry

The building has six 20-ft bays in the east-west longitudinal direction and three 20-ft bays in the north-south transverse direction. The baseline story height is 14 ft, but it is subject to change for parametric studies. The building does not have a basement. The building is used as an office building, and the Risk Category is II. The building is categorized as Type C2 (Concrete Shear Walls with Stiff Diaphragms) according to Table 3-1 of ASCE/SEI 41-17.

A.2.3 Structural Elements

The existing floor system consists of 4" non-prestressed concrete slab supported by joists in east-west direction and beams in the north-south direction. The floor system is supported by rectangular reinforced concrete columns, and the columns bear on isolated square spread footings. The existing lateral force-resisting system is composed of perforated reinforced concrete shear walls supported by strip footings.

A.2.4 Material Properties

The nominal (lower-bound) and expected grades of the *existing* materials of the building are:

- Existing concrete: f'_{cL} = 2,500 psi, f'_{cE} = 1.5(f'_{cL}) = 3,750 psi
- Existing reinforcing steel: f_{yL} = 40,000 psi, f_{yE} = 1.25(f_{yL}) = 50,000 psi

The nominal (lower-bound) and expected grades of *retrofit* materials are:

- New concrete: f'_{cL} = 5,000 psi, f'_{cE} = 1.3(f'_{cL}) = 6,500 psi

  (The factor to translate lower-bound or design concrete strength to expected compressive strength for new concrete is not specifically addressed in ASCE/SEI 41-17. A factor of 1.3 is chosen for this purpose, as recommended by ACI 319-19 Appendix A, Table A.9.1. ASCE/SEI 41-17 Table 10-1 applies to existing materials.)

- New reinforcing steel: f_{yL} = 60,000 psi, f_{yE} = 1.25(f_{yL}) = 75,000 psi

No retrofit materials are used in this case study.

A.2.5 Applied Loads

The uniform floor and roof loads are:

- Roof live load: 20 psf
- Floor live load: 125 psf (due to office light storage loads).

- Floor dead load (self-weight plus superimposed dead load) is not specified herein but is varied for the parametric studies, and the final seismic weight per level is summarized in tables for each study.

The building site is not subject to any geologic hazard such as liquefaction, lateral spreading, slope failure or tsunami.

Figure A-1 and Figure A-2 illustrate the building floor plan and south / north elevation. The floor plan remains the same for different building variants, but the elevation is subject to change for the building variants used in parametric studies. Only the east-west direction loading is being studied, and thus north-south oriented shear walls are not included in the model.

For each case described in Section 4, the geometry of the shear walls, story height and seismic story weight will be varied to meet the intended structural and loading conditions of the case study.

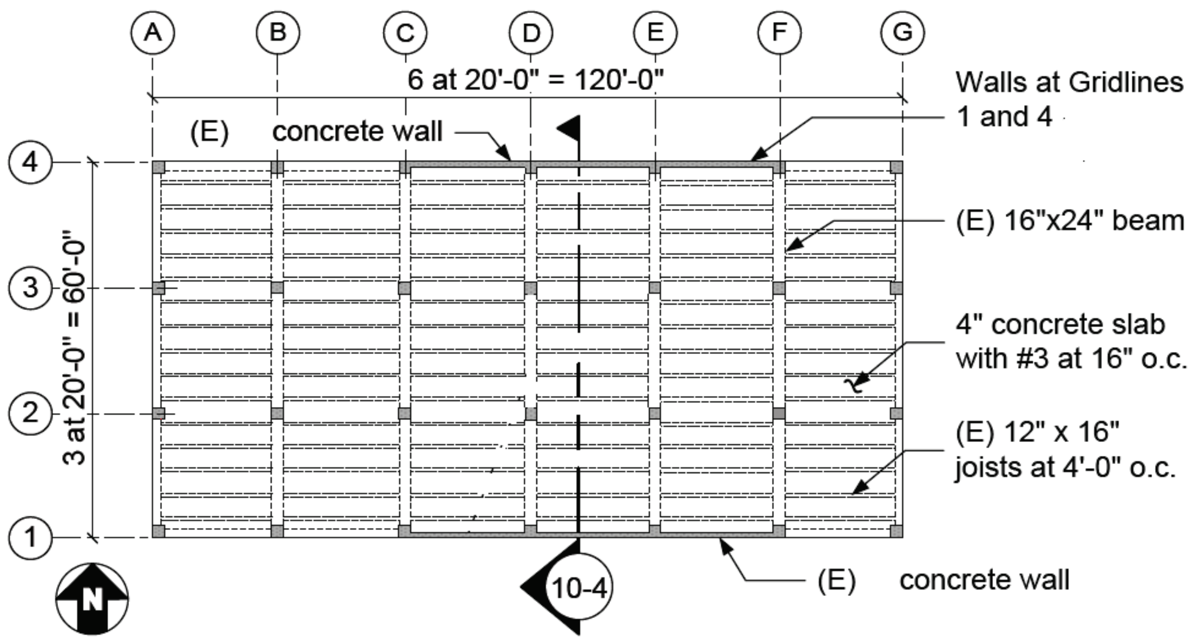

Figure A-1. Floor plan of prototype buildings.

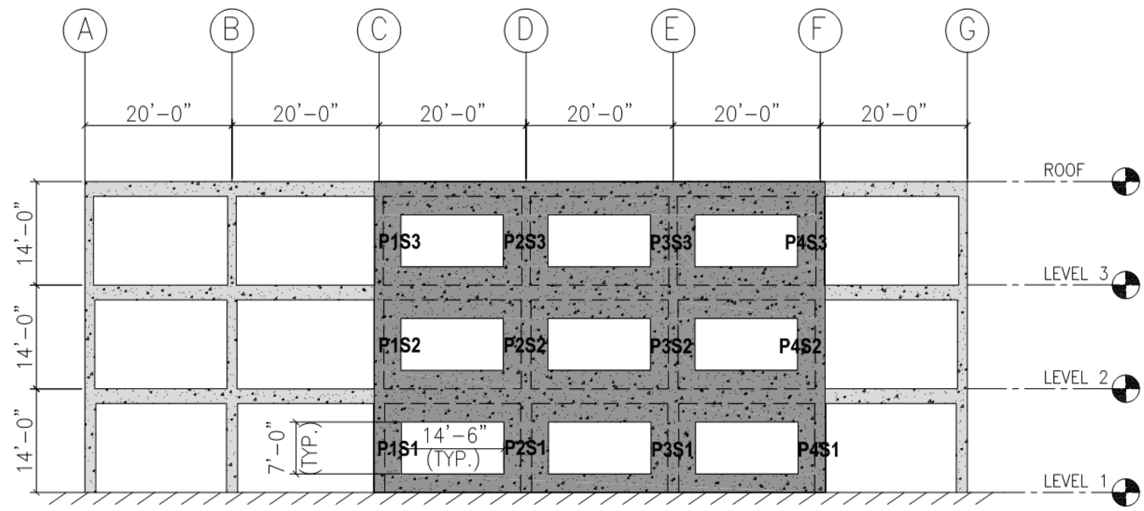

Figure A-2. **Example south and north elevation of three-story prototype building.**

A.3 Structural Models and Analysis Procedures

A.3.1 Linear Static and Linear Dynamic Procedures

Linear structural models for the prototype buildings have been created using ETABS following the provisions in Section 7.2.3 and Chapter 10 of ASCE/SEI 41-17. Some of the characteristics of the models are listed as follows:

- The structural models are three-dimensional.
- Section 7.2.3.3 of ASCE/SEI 41-17 states that the total initial lateral stiffness of secondary components, i.e., gravity-carrying frame, shall not exceed 25% of the total initial lateral stiffness of the primary components. Accordingly, the percentage of lateral loads resisted by the primary components has been checked to make sure that less than 25% of the total lateral loads are resisted by the secondary components.
- Expected material properties are assigned to structural components in the model.
- Effective sectional stiffness has been assigned to structural components according to Table 10-5 of ASCE/SEI 41-17 and other applicable specifications such as ACI 318-14 (ACI, 2014).
- Rigid diaphragms are used.
- Pinned supports are assigned to the base of shear walls. No soil-structure interaction effect is being considered at this stage.
- P-Delta effects are included.
- Gravity loads are uniformly distributed over the diaphragms.

- Seismic mass of floors is uniformly distributed over the diaphragms.
- Accidental torsional effects are checked per Section 7.2.3.2 of ASCE/SEI 41-17 and Section 12.8.4.2 of ASCE/SEI 7-16 and included if it turns out to be required.
- Load combinations for ASCE/SEI 41-17 are per Section 7.2.2 for linear analysis which includes:
  - $1.1D + 1.1 \times (0.25L) + 1.0E = 1.1D + 0.275L + 1.0E$, where L is the unreduced design live load from ASCE/SEI 7
  - $0.9D + 1.0E$

Linear static and response spectrum analysis have been performed according to the linear static procedure (LSP) and linear dynamic procedure (LDP) outlined in Section 7.4 of ASCE/SEI 41-17. The equivalent lateral force procedure and modal response spectrum analysis outlined in ASCE/SEI 7-16 have also been studied. Some features of the analysis are listed as follows

- Response spectra are scaled to reach the desired seismic load level.
- Seismic loads are parallel to the east-west direction of interest.

The following images illustrate the general layout of the ETABS finite element models that have been studied.

For simplicity, detailed comparisons focused on the LSP runs, and they are summarized in this appendix. LDP analyses are not summarized.

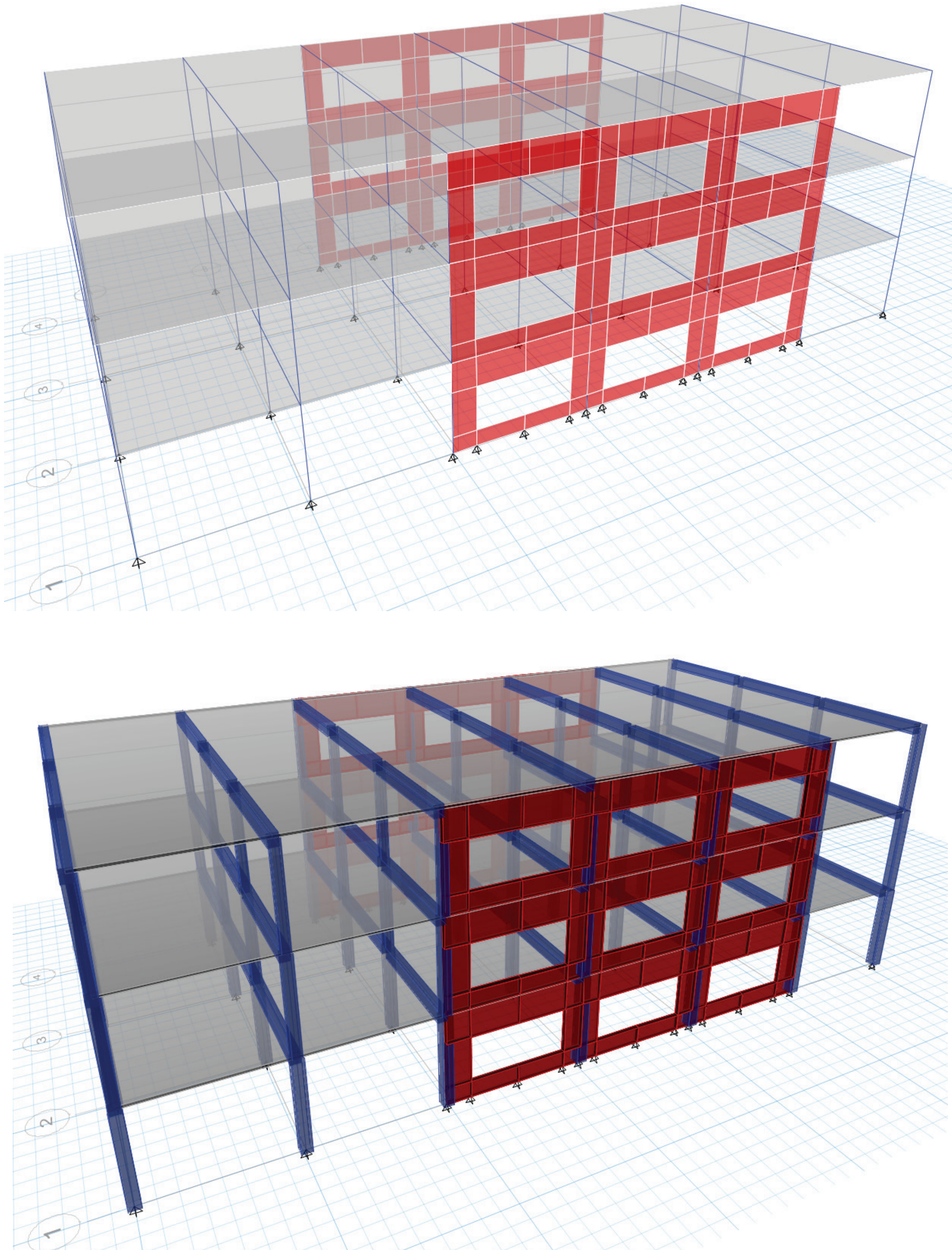

Figure A-3. Three-dimensional views of the ETABS model showing the shell elements in planar and extruded versions.

A.3.2 Nonlinear Static and Nonlinear Dynamic Procedures

Nonlinear structural models have been created using PERFORM-3D Version 7.0.0 following the provisions in Section 7.2.3 and Chapter 10 of ASCE/SEI 41-17. Some of the characteristics of the models are listed as follows:

- The structural models are three-dimensional.
- According to Section 7.2.3.3 of ASCE/SEI 41-17, both primary and secondary components, i.e., shear walls, diaphragms, and columns, are included in the model.
- Uniaxial stress-strain relations of reinforcing steel and concrete materials are created for implementing fiber-discretized sections.
- Fiber-discretized sections for capturing inelastic axial-flexural interactions are assigned to selected wall piers. Columns embedded in the walls have been incorporated into the fiber sections.
- Shear stress-strain relations have been determined according to Table 10-20 of ASCE/SEI 41-17 and assigned to selected wall piers and spandrels.
- Rigid diaphragms are used. Gravity columns are included to help capture P-Delta effects.
- Pinned supports are assigned to the base of shear walls and columns. No soil-structure interaction effects are being considered at this stage.
- P-Delta effects are considered.
- Gravity loads are applied to the top of columns according to the tributary area.
- Lateral seismic floor mass is lumped at the master node of the rigid floor constraint.
- Accidental torsional effects were checked.
- A linear elastic gravity analysis is conducted prior to any nonlinear seismic analysis and the gravity loads at the end of the gravity analysis remain constant in the seismic analysis.
- Seismic displacements are applied parallel to the east-west direction of interest.
- Vertical seismic effects are not considered.
- The load combination for ASCE/SEI 41-17 is per Section 7.2.2 for nonlinear analysis which is:
    - $1.0D + 0.25L + 1.0E$, where L is the unreduced design live load from ASCE/SEI 7
- Nonlinear static pushover analysis is conducted per the nonlinear static procedure (NSP) outlined in Section 7.4 of ASCE/SEI 41-17.

- For the NSP, response spectra are scaled to reach the desired seismic demand.
- For the nonlinear dynamic procedure (NDP), nonlinear response history analysis (NRHA) is performed according to Section 7.4 of ASCE/SEI 41-17.
- For the NDP, earthquake ground motions are scaled to reach the desired seismic demand.

The following Figure A1-4 illustrates the general layout of the PERFORM-3D finite element models that have been studied.

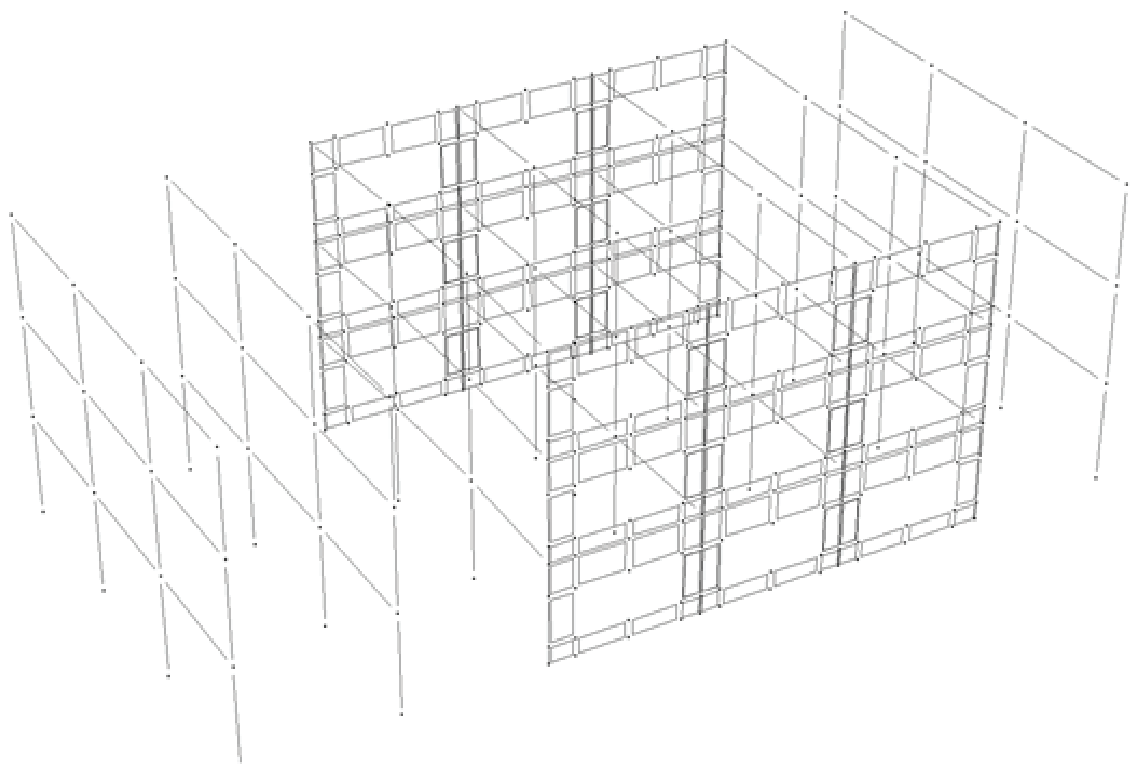

Figure A-4. Three-dimensional view of the PERFORM-3D model.

During the first phase of the study, a typical seismic demand in coastal California outside the near field zone for short-period buildings (S_S = 1.0 at BSE-1N and S_S = 1.5 at BSE-2N) were used for the LSP, NSP, and NDP analyses. This level of seismic shaking was such that the buildings showed insufficient capacity in all three procedures. For this reason, a reduced demand was selected for the new phase of the project. A value of S_S = 0.667 was selected for the BSE-1N level and S_S = 1.0 for the BSE-2N level. Lower demand was expected to help achieve more pronounced results between procedures. The value of S_a = 0.67 was used for the Design Earthquake level demand for ASCE/SEI 7-16. In order to make sure the building is representative of 1960s design, it was also evaluated using the 1961 Uniform Building Code using Z = 1.0 and K = 1.0. For NRHA, the ground motions of the first phase were scaled down by multiplying their accelerations by 2/3.

Since the first phase did not show differences between the LSP, NSP, and NDP procedures, first phase results are not useful in exploring differences, and they are not summarized here. Instead, results for the second phase S_S = 0.67 at BSE-1N and S_S = 1.0 at BSE-2N are summarized. In addition, due to large volume of information, results are only presented in this appendix for shear demands and capacities, rather than moment demands. The main Part 1, Chapter 1 report contains short summaries of key results for the S_S = 1.0 at BSE-1N and S_S = 1.5 at BSE-2N case study, and maximum moment DCRs for both case studies.

A.4 Weak Story Irregularity Studies

A.4.1 ASCE/SEI 41-17 Provisions

Section 7.3.1.1.3 of ASCE/SEI 41-17 defines the “weak story” vertical irregularity based on the average DCR between two stories: “a weak story irregularity shall be considered to exist in any direction of the building if the ratio of the average shear DCR for elements in any story to that of an adjacent story in the same direction exceeds 125%.” Section 7.3.1.1 states that if a component DCR exceeds the lesser of 3.0 and the *m*-factor for the component action and any irregularity is present, then linear procedures shall not be used.

The definition of “weak story” in Section 7.3.1.1.3 of ASCE/SEI 41-17 could be too stringent for some common building types. A very common and simple example would be when a two-story shear wall building with the same wall piers at each story is subjected to an inverted triangular seismic load, the average shear DCR for wall piers in Story 1 is 150% of that in Story 2 and, thus, a “weak story” is identified. By comparison, in ASCE/SEI 7-16, the weak story definition is where the “story lateral strength is less than 80% of the story above,” and the extreme weak story is where “the story lateral strength is less than 65% of the story above.” In this two-story example, with the same story strength at each story, there is no weak story per ASCE/SEI 7-16.

In the presence of the “weak story” irregularity, if the DCR of any wall pier exceeds the lesser of 3.0 the *m*-factor for the component action, linear procedures shall not be used.

Examining the limitation on use of linear procedures in the presence of weak story irregularity will help identify the accuracy of this limitation and possibly broaden the range of buildings that can be evaluated using linear procedures.

A.4.2 Analysis Plan

Comparisons between linear and nonlinear analyses were performed using a linear model and a detailed nonlinear structural model that captures the governing damage/failure mode of the wall piers (shear or flexure). The Acceptance Ratios of wall piers are compared between the linear and nonlinear procedures to check the accuracy of linear procedures.

The analysis plan for examining the weak story irregularity is as follows.

- Three-story reinforced concrete shear wall buildings with different perforated perimeter walls are being studied. For each building, the walls on Gridlines 1 and 4 are identical. The perforated perimeter walls have been designed in the following three patterns:

- Pattern 1: Same wall piers at all three stories, as shown in Figure A-5;

- Pattern 2: Tall first story, as shown in Figure A-6; and

- Pattern 3: Wider wall piers on upper stories, as shown in Figure A-7. Only linear and nonlinear static procedure analyses were explored for Pattern 3. Given the results from Patterns 1 and 2, it was found that there was no need to do nonlinear response history analysis of Pattern 3.

- Create linear and nonlinear building models and conduct linear static, linear dynamic, nonlinear static, and nonlinear response history analyses.

- Compare Acceptance Ratios of wall piers between linear and nonlinear procedures.

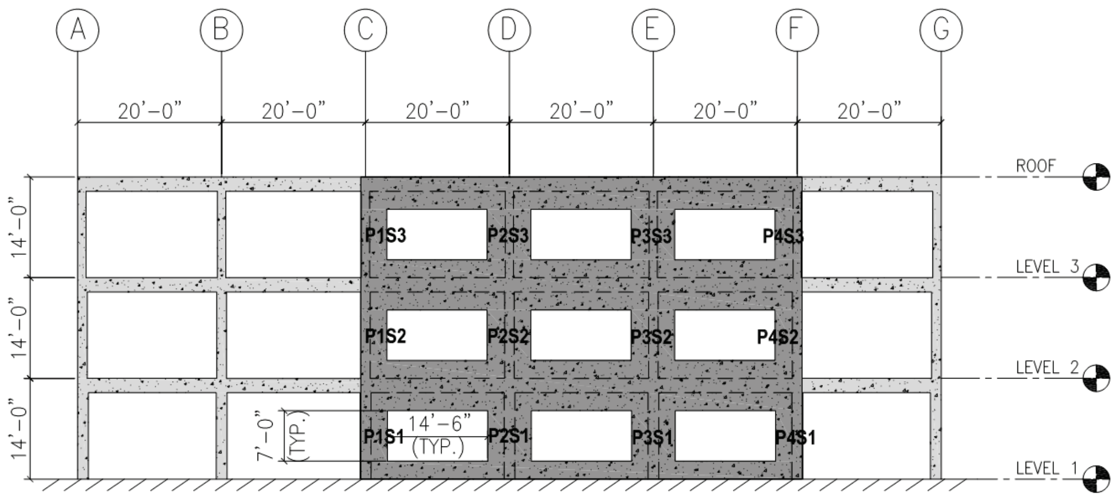

Figure A-5a. Shear Wall Pattern No. 1.

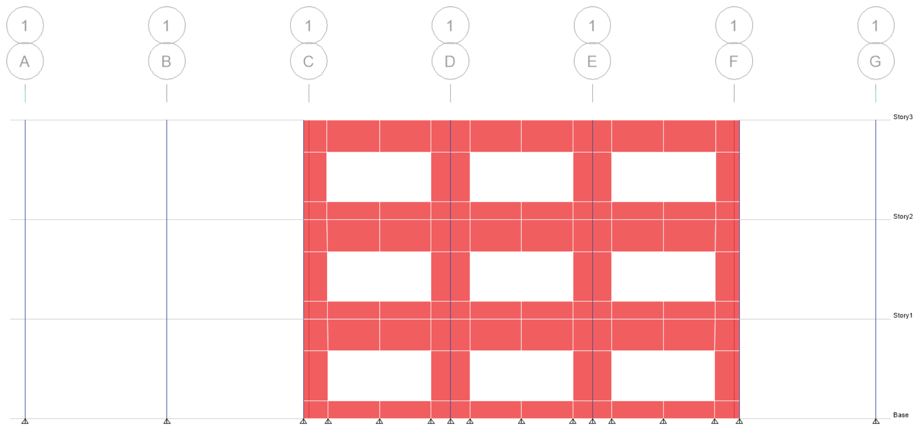

Figure A-5b. ETABS model for Shear Wall Pattern No. 1.

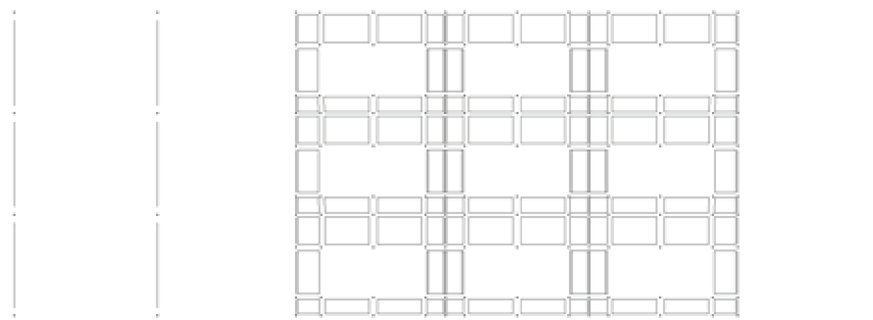

Figure A-5c. PERFORM-3D model for Shear Wall Pattern No. 1.

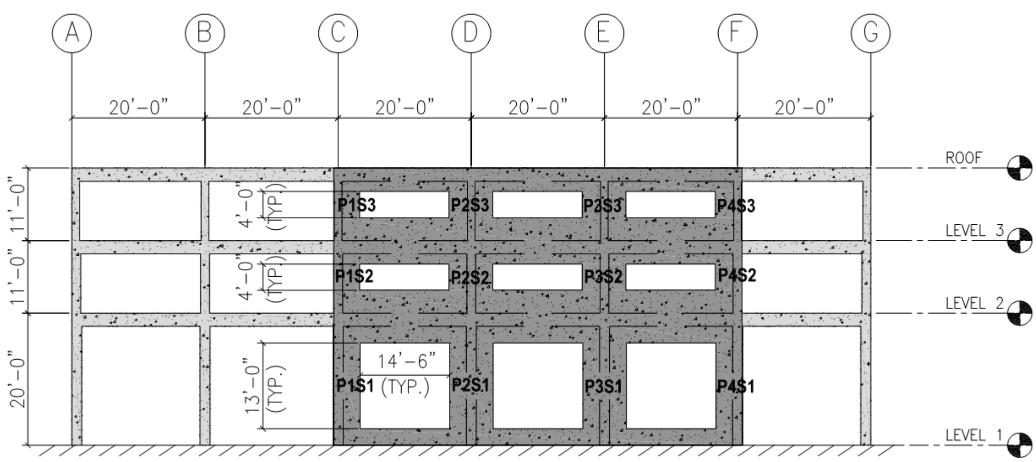

Figure A-6a. Shear Wall Pattern No. 2.

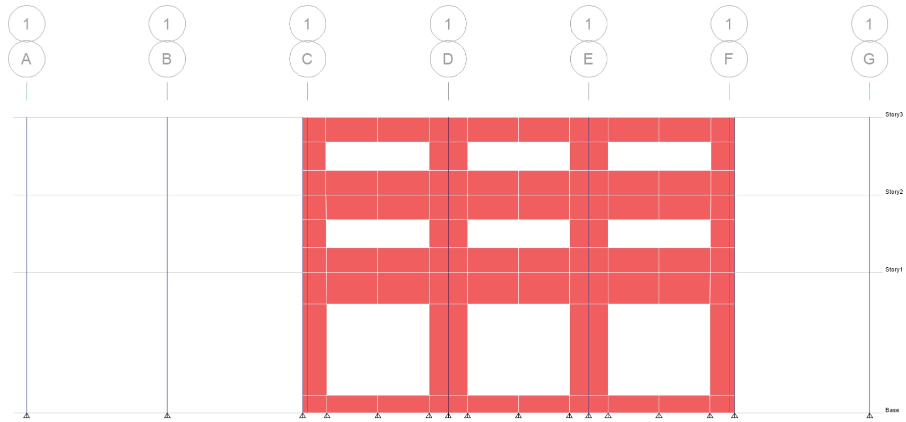

Figure A-6b. ETABS Model for Shear Wall Pattern No. 2.

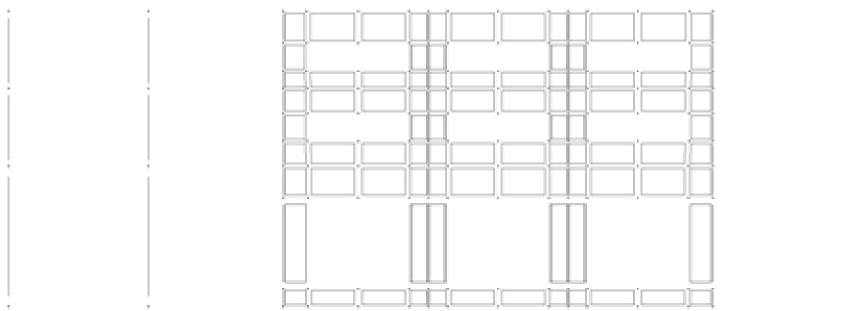

Figure A-6c. PERFORM-3D Model for Shear Wall Pattern No. 2.

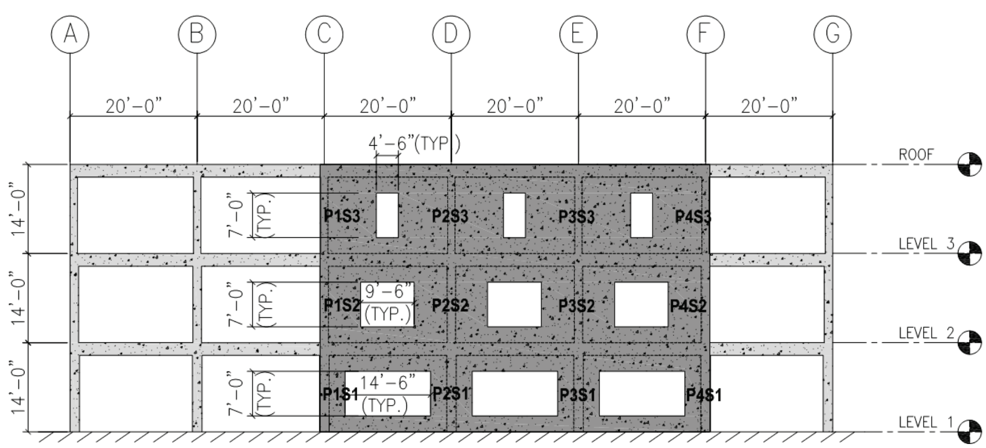

Figure A-7a. Shear Wall Pattern No. 3.

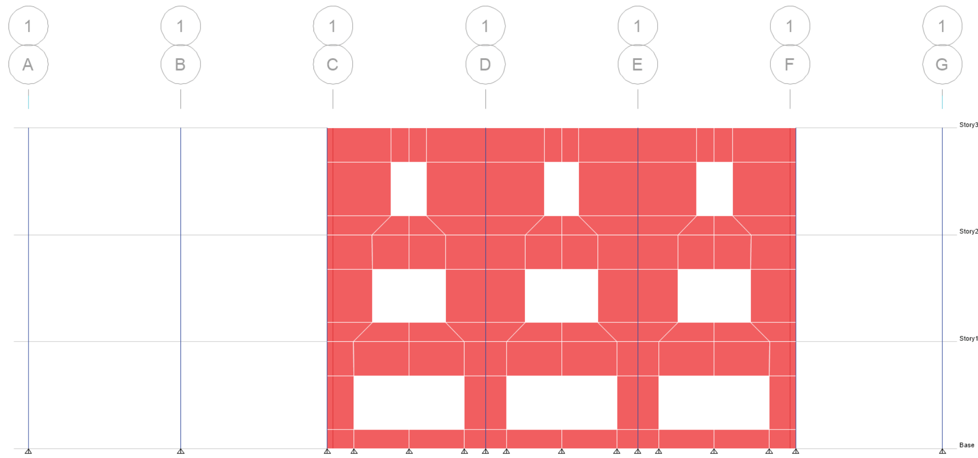

Figure A-7b ETABS Model for Shear Wall Pattern No. 3.

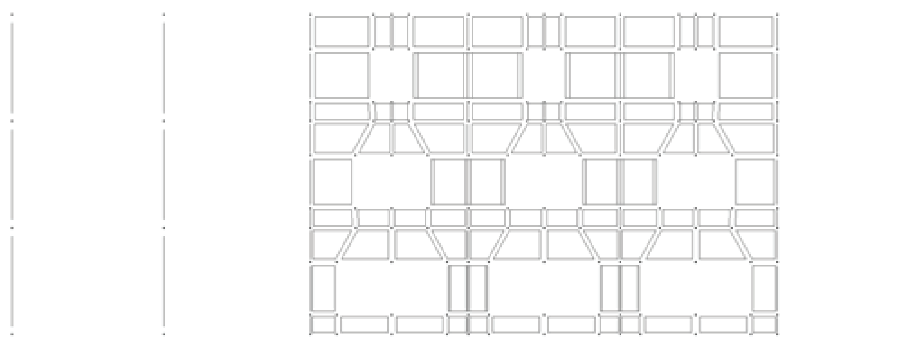

Figure A-7c PERFORM-3D Model for Shear Wall Pattern No. 3.

A.4.3 Details of Models and Analysis

Some key design parameters of shear wall patterns No. 1 to 3 are listed in Table A-1 to Table A-3, respectively. The 1961 UBC requires a minimum horizontal reinforcing ratio of $\rho = 0.0025$. This is larger than the value of $\rho = 0.0015$ in ASCE/SEI 7-16 Section 10.7.2.2 which specifies that walls with lower values shall be considered force-controlled. #4 bars at 9 in. o.c. were used in the 8 in. wall which gives a value of $\rho = 0.0028$.

Table A-1: Design Parameters of Shear Wall Pattern No. 1 (as shown in Figure A-5)

Story	Seismic Story Weight[1]	Story Height	Wall Thickness	Reinforcing Steel[2]	Shear Reinf. Ratio	Pier Width (in.)				Pier Clear Height (in.)			
	(kips)	(ft)	(in.)			P1	P2	P3	P4	P1	P2	P3	P4
3	800	14	8	One layer of #4@9" o.c., each way	0.0028	40	66	66	40	84	84	84	84
2	900	14	8			41	66	66	41	84	84	84	84
1	900	14	8			42	66	66	42	84	84	84	84

1 Seismic story force is shared by two shear walls with the same pattern at Gridlines 1 and 4.
2 Development of reinforcements is assumed to be adequate.

Table A-2: Design Parameters of Shear Wall Pattern No. 2 (as shown in Figure A-6)

Story	Seismic Story Weight[1]	Story Height	Wall Thickness	Reinforcing Steel[2]	Shear Reinf. Ratio	Pier Width (in.)				Pier Clear Height (in.)			
	(kips)	(ft)	(in.)			P1	P2	P3	P4	P1	P2	P3	P4
3	800	14	8	One layer of #4@9" o.c., each way	0.0028	40	66	66	40	48	48	48	48
2	900	14	8			41	66	66	41	48	48	48	48
1	900	14	8			42	66	66	42	156	156	156	156

1 Seismic story force is shared by two shear walls with the same pattern at Gridlines 1 and 4.
2 Development of reinforcements is assumed to be adequate.

Table A-3 Design Parameters of Shear Wall Pattern No. 3 (as shown in Figure A-7)

Story	Seismic Story Weight[1]	Story Height	Wall Thickness	Reinforcing Steel[2]	Shear Reinf. Ratio	Pier Width (in.)				Pier Clear Height (in.)			
	(kips)	(ft)	(in.)			P1	P2	P3	P4	P1	P2	P3	P4
3	800	14	8	One layer of #4@9" o.c., each way	0.0028	100	186	186	100	84	84	84	84
2	900	14	8			71	126	126	71	84	84	84	84
1	900	14	8			42	66	66	42	84	84	84	84

1 Seismic story force is shared by two shear walls with the same pattern at Gridlines 1 and 4.
2 Development of reinforcements is assumed to be adequate.

A.4.4 Shear Wall Pattern 1

A.4.4.1 UBC 1961 ACCEPTANCE RATIOS

Table A1-4 shows the derivation of allowable stress demand (ASD) story forces. Table A1-5 shows key properties for each pier, including the ASD demand-to-capacity ratio. In keeping with the terminology used in FEMA P-2006, the "Acceptance Ratio" will be used to compare results between different methods. For the UBC, this is the traditional demand-to-capacity ratio, which for shear is the allowable stress design ratio V_E / V_{allow}. Per Table A1-5, the maximum Acceptance Ratio is 0.57, and it occurs at the first story.

Table A-4: Calculation of Seismic Story Force per UBC 1961

Story	Seismic weight (k)	Seismic height (ft)	Z	K	H (ft)	D (ft)	T (s)	C	$V = ZKCW$ (k)	H_x (ft)	F_x (k)	F_{story} (k)
3	800	14								42	106	106
2	900	14	1.0	1.0	42	120	0.19	0.09	225	28	80	186
1	900	14								14	40	225
Total	2,600										225	

Table A-5: Check of Wall Pier Capacity per UBC 1961

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	f_{allow} (psi)	V_{allow} (k)	V_E (k)	V_E/V_{allow}
3	P1S3	40	84	8	125	40	8	0.21
	P2S3	66	84	8	125	66	18	0.28
	P3S3	66	84	8	125	66	18	0.28
	P4S3	40	84	8	125	40	8	0.21
2	P1S2	41	84	8	125	41	15	0.36
	P2S2	66	84	8	125	66	31	0.47
	P3S2	66	84	8	125	66	31	0.47
	P4S2	41	84	8	125	41	15	0.36
1	P1S1	42	84	8	125	42	19	0.44
	P2S1	66	84	8	125	66	38	**0.57**
	P3S1	66	84	8	125	66	38	**0.57**
	P4S1	42	84	8	125	42	19	0.44

A.4.4.2 ASCE/SEI 7-16 ACCEPTANCE RATIOS

Table A-6 shows the derivation of factored story forces per ASCE/SEI 7-16. For ASCE/SEI 7-16, the Acceptance Ratio is the traditional demand-to-capacity ratio, which for shear is the factored ratio $V_u / \Phi V_n$. Per Table A-7, the maximum Acceptance Ratio is 1.08, and it occurs at the first story. Acceptance Ratios over 1.0 are shaded in red.

Table A-6 Calculation of Story Shear for Pattern 1 per ASCE/SEI 7-16

Story	Seismic Weight (k)	Seismic Height (ft)	k	S_{DS}	R	I_e	C_s	V (k)	h_x (ft)	C_v	F_x (k)	F_{story} (k)
3	800	14	1	0.667	5	1.0	0.1	347	42	0.47	163	163
2	900	14	1						28	0.35	122	285
1	900	14	1						14	0.18	61	347
Total	2,600										347	

Table A-7 Check of Wall Pier Capacity per ASCE/SEI 7-16 (1.4*D* + 1.0*L* + 1.0*E*)

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	ϕV_n (k)	V_u (k)	$V_u/\phi V_n$
3	P1S3	40	84	8	32	11	0.34
	P2S3	66	84	8	54	35	0.65
	P3S3	66	84	8	54	31	0.59
	P4S3	40	84	8	32	17	0.52
2	P1S2	41	84	8	33	20	0.61
	P2S2	66	84	8	54	53	0.99
	P3S2	66	84	8	54	50	0.94
	P4S2	41	84	8	33	30	0.89
1	P1S1	42	84	8	34	26	0.78
	P2S1	66	84	8	54	58	1.08
	P3S1	66	84	8	54	56	1.05
	P4S1	42	84	8	34	33	0.98

A.4.4.3 ASCE/SEI 41-17 LSP ACCEPTANCE RATIOS AND LIMITATIONS

Table A-8 shows the derivation of story forces per the ASCE/SEI 41-17 LSP. Table A-9 and Table A-11 show that at both the BSE-1N and BSE-2N hazard levels, the limitation restrictions are not triggered since neither the weak story check nor the average DCR exceeds the allowable *m*-factor. However, a soft story is identified, and the limitation is triggered. For the ASCE/SEI 41-17 LSP, the Acceptance Ratio for shear is the ratio $V_{UD} / \kappa m V_{CE}$. Per Table A-10 and Table A-12, the maximum Acceptance Ratios are 0.89 and 1.07 for the BSE-1N and BSE-2N hazard levels, respectively. These maxima occur at the first story.

Table A-8: Calculation of Story Shear for Pattern 1 per ASCE/SEI 41-17

Story	Seismic Weight (k)	Story Height (ft)	k	C_1C_2	C_m	S_a	V (k)	h_x (ft)	C_v	F_x (k)	F_{story} (k)
3	800	14	1					42	0.47	718	**718**
2	900	14	1	1.1	0.8	0.667	1,525	28	0.35	538	**1,256**
1	900	14	1					14	0.18	269	**1,525**
Total	2,600									1,525	

Table A-9: Check of Weak and Soft Story Irregularities for Pattern 1 per ASCE/SEI 41-17 at BSE-1N

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	DCR	*m*-factor (k)	Average Shear DCR	Ratio of Average DCR Between this Story and Story Above	Weak Story?	Limit Use of Linear Procedures?	Story Drift Ratio	Soft story and Limit Use of Linear Static Procedure?
3	P1S3	40	84	8	0.8	2.5	1.0	N/A	N/A	NO	0.09%	NO
	P2S3	66	84	8	1.1	2.5				NO		
	P3S3	66	84	8	1.1	2.5				NO		
	P4S3	40	84	8	0.8	2.5				NO		
2	P1S2	41	84	8	1.5	2.5	1.7	1.70	YES	NO	0.13%	YES
	P2S2	66	84	8	1.9	2.5				NO		
	P3S2	66	84	8	1.9	2.5				NO		
	P4S2	41	84	8	1.5	2.0				NO		

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	DCR	m-factor (k)	Average Shear DCR	Ratio of Average DCR Between this Story and Story Above	Weak Story?	Limit Use of Linear Procedures?	Story Drift Ratio	Soft story and Limit Use of Linear Static Procedure?
1	P1S1	42	84	8	1.8	2.5	2.0	1.16	NO	NO	0.12%	NO
	P2S1	66	84	8	2.1	2.5				NO		
	P3S1	66	84	8	1.8	2.5				NO		
	P4S1	42	84	8	2.1	2.0				NO		

Table A-10: Check of Wall Piers via LSP of ASCE/SEI 41-17 for Pattern 1

BSE-1N, 0.9*D* + 1.0*E*

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	V_{CE} (k)	V_{UD} (k)	m_{LS} (Shear)	$\kappa m_{LS} V_{CE}$	Acceptance Ratio $V_{UD} / \kappa m_{LS} V_{CE}$
3	P1S3	40	84	8	67	53	2.5	167	0.32
	P2S3	66	84	8	110	125	2.5	276	0.45
	P3S3	66	84	8	110	123	2.5	276	0.45
	P4S3	40	84	8	67	56	2.5	167	0.33
2	P1S2	41	84	8	69	100	2.5	171	0.58
	P2S2	66	84	8	110	206	2.5	276	0.75
	P3S2	66	84	8	110	205	2.5	276	0.74
	P4S2	41	84	8	69	103	2.0	137	0.75
1	P1S1	42	84	8	70	123	2.5	176	0.70
	P2S1	66	84	8	110	235	2.5	276	0.85
	P3S1	66	84	8	110	235	2.5	276	0.85
	P4S1	42	84	8	70	126	2.0	141	0.89

Table A-10: Check of Wall Piers via LSP of ASCE/SEI 41-17 for Pattern 1 (continued)

BSE-1N, 1.1*D* + 0.275*L* + 1.0*E*

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	V_{CE} (k)	V_{UD} (k)	m_{LS} (Shear)	$km_{LS}V_{CE}$	Acceptance Ratio $V_{UD}/\ \kappa m_{LS}V_{CE}$
3	P1S3	40	84	8	67	53	2.5	167	0.31
	P2S3	66	84	8	110	126	2.5	276	0.46
	P3S3	66	84	8	110	123	2.5	276	0.45
	P4S3	40	84	8	67	56	2.5	167	0.34
2	P1S2	41	84	8	69	99	2.5	171	0.58
	P2S2	66	84	8	110	207	2.5	276	0.75
	P3S2	66	84	8	110	205	2.5	276	0.74
	P4S2	41	84	8	69	104	2	137	0.76
1	P1S1	42	84	8	70	122	2.5	176	0.70
	P2S1	66	84	8	110	236	2.5	276	0.85
	P3S1	66	84	8	110	235	2.5	276	0.85
	P4S1	42	84	8	70	126	2	141	0.90

Table A-11: Check of Weak and Soft Story Irregularities for Pattern 1 per ASCE/SEI 41-17 at BSE-2N

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	DCR	Average Shear DCR	Ratio of Average DCR Between this Story and Story Above	Weak Story?	Limit Use of Linear Procedures?	Story Drift Ratio	Soft story and Limit Use of Linear Static Procedure?
3	P1S3	40	84	8	1.2	1.5	N/A	N/A	NO	0.13%	NO
	P2S3	66	84	8	1.7				NO		
	P3S3	66	84	8	1.7				NO		
	P4S3	40	84	8	1.2				NO		
2	P1S2	41	84	8	2.2	2.6	1.70	YES	NO	0.20%	YES
	P2S2	66	84	8	2.8				NO		
	P3S2	66	84	8	2.8				NO		
	P4S2	41	84	8	2.2				NO		
1	P1S1	42	84	8	2.7	3.0	1.16	NO	NO	0.18%	NO
	P2S1	66	84	8	3.2				NO		
	P3S1	66	84	8	3.2				NO		
	P4S1	42	84	8	2.7				NO		

Table A-12: Check of Wall Piers via LSP of ASCE/SEI 41-17 for Pattern 1

BSE-2N, 0.9*D* + 1.0*E*

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	$f'_{cE}l_wd_w$ (k)	$P/f'_{cE}l_wd_w$ (k)	m_{CP}	$km_{CP}V_{CE}$	Acceptance Ratio $V_{UD}/\ \kappa m_{LS}V_{CE}$
3	P1S3	40	84	8	1200	0.00	3	201	0.40
	P2S3	66	84	8	1980	0.00	3	331	0.56
	P3S3	66	84	8	1980	0.01	3	331	0.56
	P4S3	40	84	8	1200	0.05	3	201	0.41
2	P1S2	41	84	8	1230	0.00	3	206	0.73
	P2S2	66	84	8	1980	0.00	3	331	0.93
	P3S2	66	84	8	1980	0.02	3	331	0.93
	P4S2	41	84	8	1230	0.10	3	206	0.75
1	P1S1	42	84	8	1260	0.00	3	211	0.88
	P2S1	66	84	8	1980	0.00	3	331	1.07
	P3S1	66	84	8	1980	0.02	3	331	1.06
	P4S1	42	84	8	1260	0.07	3	211	0.89

Table A-12: Check of Wall Piers via LSP of ASCE/SEI 41-17 for Pattern 1 (continued)

BSE-2N, 1.1*D* + 0.275*L* + 1.0*E*

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	$f'_{cE}l_wd_w$ (k)	$P/f'_{cE}l_wd_w$ (k)	m_{CP}	$km_{CP}V_{CE}$	Acceptance Ratio $V_{UD}/km_{LS}V_{CE}$
3	P1S3	40	84	8	1200	0.00	3	201	0.40
	P2S3	66	84	8	1980	0.00	3	331	0.57
	P3S3	66	84	8	1980	0.01	3	331	0.56
	P4S3	40	84	8	1200	0.05	3	201	0.42
2	P1S2	41	84	8	1230	0.00	3	206	0.73
	P2S2	66	84	8	1980	0.00	3	331	0.93
	P3S2	66	84	8	1980	0.03	3	331	0.93
	P4S2	41	84	8	1230	0.10	3	206	0.75
1	P1S1	42	84	8	1260	0.00	3	211	0.88
	P2S1	66	84	8	1980	0.01	3	331	1.07
	P3S1	66	84	8	1980	0.03	3	331	1.06
	P4S1	42	84	8	1260	0.08	3	211	0.89

A.4.4.4 ASCE/SEI 41-17 NSP ACCEPTANCE RATIOS AND LIMITATIONS

Figure A-8 shows the target displacements at the roof for Pattern 1. At the BSE-1N seismic hazard level, the target displacement is 0.74”. At the BSE-2N level, it is 1.50”.

For the ASCE/SEI 41-17 NSP, the Acceptance Ratio for shear is the ratio of the shear strain at the target displacement divided by the target strain per ASCE/SEI 41-17. Per Table A1-13, at the BSE-1N hazard level and the Life Safety Building Performance Level, the highest Acceptance Ratio is at the first story where the strain at the target displacement is 0.32% and the limit is 0.75% (for Pier 4) for an Acceptance Ratio of 0.43. At the BSE-2N level, the Acceptance Ratio is much higher at 2.92.

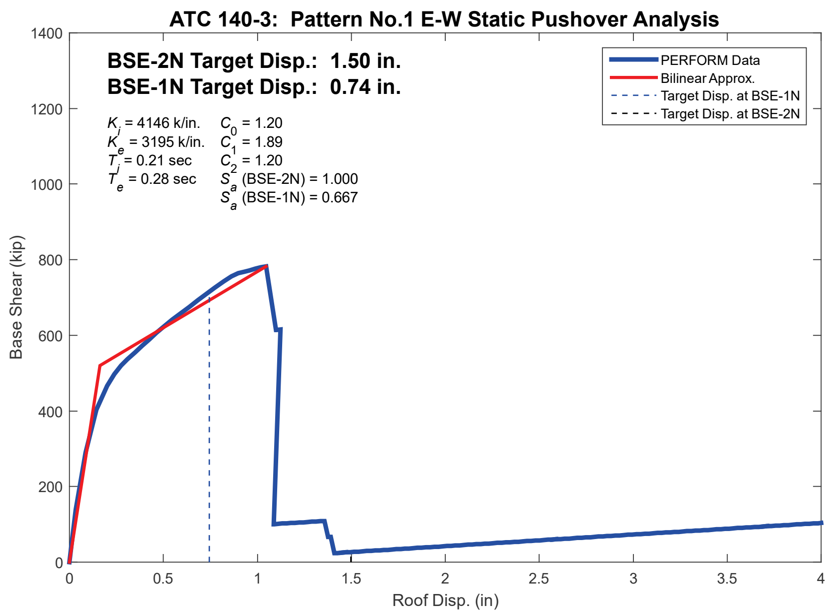

Figure A-8 Target displacements for Shear Wall Pattern No. 1.

Table A-13: Check of Wall Piers via Nonlinear Static Procedure (NSP) of ASCE/SEI 41-17 for Pattern 1

BSE-1N, 1.0*D* + 0.25*L* + 1.0*E*

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	Shear Strain at Target Disp.	Axial at Target Disp. (k)	Target Strain Limit for LS	Acceptance Ratio
3	P1S3	40	84	8	0.00%	-11	1.50%	0.00
	P2S3	66	84	8	0.01%	-8	1.50%	0.00
	P3S3	66	84	8	0.01%	-34	1.50%	0.01
	P4S3	40	84	8	0.00%	-46	1.50%	0.00
2	P1S2	41	84	8	0.09%	1	1.50%	0.06
	P2S2	66	84	8	0.18%	-28	1.50%	0.12
	P3S2	66	84	8	0.21%	-64	1.50%	0.14
	P4S2	41	84	8	0.17%	-139	0.75%	0.23
1	P1S1	42	84	8	0.23%	41	1.50%	0.15
	P2S1	66	84	8	0.33%	-67	1.50%	0.22
	P3S1	66	84	8	0.34%	-81	1.50%	0.23
	P4S1	42	84	8	0.32%	-251	0.75%	0.43

Table A-13: Check of Wall Piers via Nonlinear Static Procedure (NSP) of ASCE/SEI 41-17 for Pattern 1 (continued)

BSE-2N, 1.0*D* + 0.25*L* + 1.0*E*

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	Shear Strain at Target Disp.	Axial at Target Disp. (k)	Target Strain Limit for LS	Acceptance Ratio
3	P1S3	40	84	8	0.00%	-29	2.00%	0.00
	P2S3	66	84	8	0.00%	-15	2.00%	0.00
	P3S3	66	84	8	0.02%	-24	2.00%	0.01
	P4S3	40	84	8	0.00%	-28	2.00%	0.00
2	P1S2	41	84	8	0.15%	-46	2.00%	0.07
	P2S2	66	84	8	0.26%	-63	2.00%	0.13
	P3S2	66	84	8	0.29%	-55	2.00%	0.15
	P4S2	41	84	8	0.25%	-59	2.00%	0.13
1	P1S1	42	84	8	2.53%	-55	2.00%	1.26
	P2S1	66	84	8	2.24%	-125	1.00%	2.24
	P3S1	66	84	8	2.30%	-81	2.00%	1.15
	P4S1	42	84	8	2.92%	-88	1.00%	2.92

A.4.4.5 ASCE/SEI 41-17 NDP ACCEPTANCE RATIOS AND LIMITATIONS

The following tables summarizes the results obtained from the Nonlinear Dynamic Procedure for both hazard levels, BSE-1N and BSE-2N. The maximum Acceptance Ratio within all piers of story 1 are presented here. To facilitate the collection and analysis of the results, the shear strain was measured only at the middle of the pier and the flexural rotation was measured at the top and bottom. Table A-14 and Table A-15 and show the results for Pattern 1 at BSE-1N and BSE-2N, respectively. In these tables, the Acceptance Ratio for shear strain and flexural rotation are indicated for each of the 11 ground motions of the sets. The average values are shown at the last line of the table and the maximum of the Acceptance Ratios per ground motion in Column 6.

Table A-14: Pattern 1 Acceptance Ratio at BSE-1N

EQ	Name	Acceptance Ratio				Comments
		Shear	Rotation Top	Rotation Bottom	Max	
1	Imperial Valley	0.14	0.14	0.14	0.14	Analysis finished
2	Loma Prieta	13.62	0.29	0.34	13.62	All piers at Story 1 reached LS limit state in shear
3	Northridge, Sylmar CSE	42.71	0.26	0.34	42.71	All piers at Story 1 reached LS limit state in shear
4	Northridge, Sylmar OVM	14.92	0.09	0.11	14.92	All piers at Story 1 reached LS limit state in shear
5	Chi Chi TCU079	25.48	0.26	0.36	25.48	All piers at Story 1 reached LS limit state in shear
6	Chi Chi TCU122	0.12	0.13	0.14	0.14	Analysis finished
7	Duzce	1.33	0.29	0.30	1.33	P1 to P3 reached LS limit state in shear
8	Chetsu 65010	0.31	0.14	0.17	0.31	Analysis finished
9	Chetsu 65025	0.18	0.11	0.14	0.18	Analysis finished
10	Mexico, Chihuahua	0.14	0.12	0.13	0.14	Analysis finished
11	Mexico, Ejido Saltillo	0.09	0.12	0.13	0.13	Analysis finished
AVERAGE:		9.01	0.18	0.21	9.01	

Table A-15: Pattern 1 Acceptance Ratio at BSE-2N

EQ	Name	Acceptance Ratio				Comments
		Shear	Rotation Top	Rotation Bottom	Max	
1	Imperial Valley	0.19	0.09	0.09	0.19	Analysis finished
2	Loma Prieta	18.53	0.17	0.17	18.53	All piers at Story 1 reached CP limit state in shear
3	Northridge, Sylmar CSE	36.19	0.18	0.15	36.19	All piers at Story 1 and 2 reached LS limit state in shear
4	Northridge, Sylmar OVM	14.27	0.13	0.14	14.27	All piers at Story 1 reached CP limit state in shear
5	Chi Chi TCU079	15.59	0.17	0.15	15.59	All piers at Story 1 reached CP limit state in shear
6	Chi Chi TCU122	0.15	0.08	0.08	0.15	Analysis finished
7	Duzce	16.42	0.14	0.14	16.42	All piers at Story 1 reached CP limit state in shear
8	Chetsu 65010	0.33	0.13	0.14	0.33	Analysis finished
9	Chetsu 65025	0.17	0.07	0.08	0.17	Analysis finished
10	Mexico, Chihuahua	0.15	0.08	0.08	0.15	Analysis finished
11	Mexico, Ejido Saltillo	0.12	0.08	0.07	0.12	Analysis finished
AVERAGE:		9.28	0.12	0.12	9.28	

The Acceptance Ratios were calculated dividing the maximum deformation experienced in the pier (deformation demand) by the deformation capacity per ASCE/SEI 41-17. The red numbers represent the cases where the Acceptance Ratio is greater than the allowable deformation per ASCE/SEI 41-17. The selected criterion used in PERFORM-3D to stop the analysis was a maximum story drift of 4%. The reason to select that criterion in lieu of a maximum allowable deformation associated with Acceptance Ratio (in shear or flexure) was to allow the building to deform as much as possible to evaluate the level of damage in each pier at the end of the ground motion. After the analysis, no local instability or convergence problem were identified, and all the time steps were applied to the structure under both hazard levels.

As an example, the following figure illustrate the shear deformation time history and the flexural rotation time history at the top and bottom of the pier 1 for Pattern 1 under Loma Prieta ground motion at BSE-1 hazard level. The rotational Acceptance Ratio for the top gage was calculated as follow. Similar calculations were followed to calculate the rest of Acceptance Ratios.

$$AR_{\max}^{C} = \frac{0.00175}{0.006} = 0.29$$

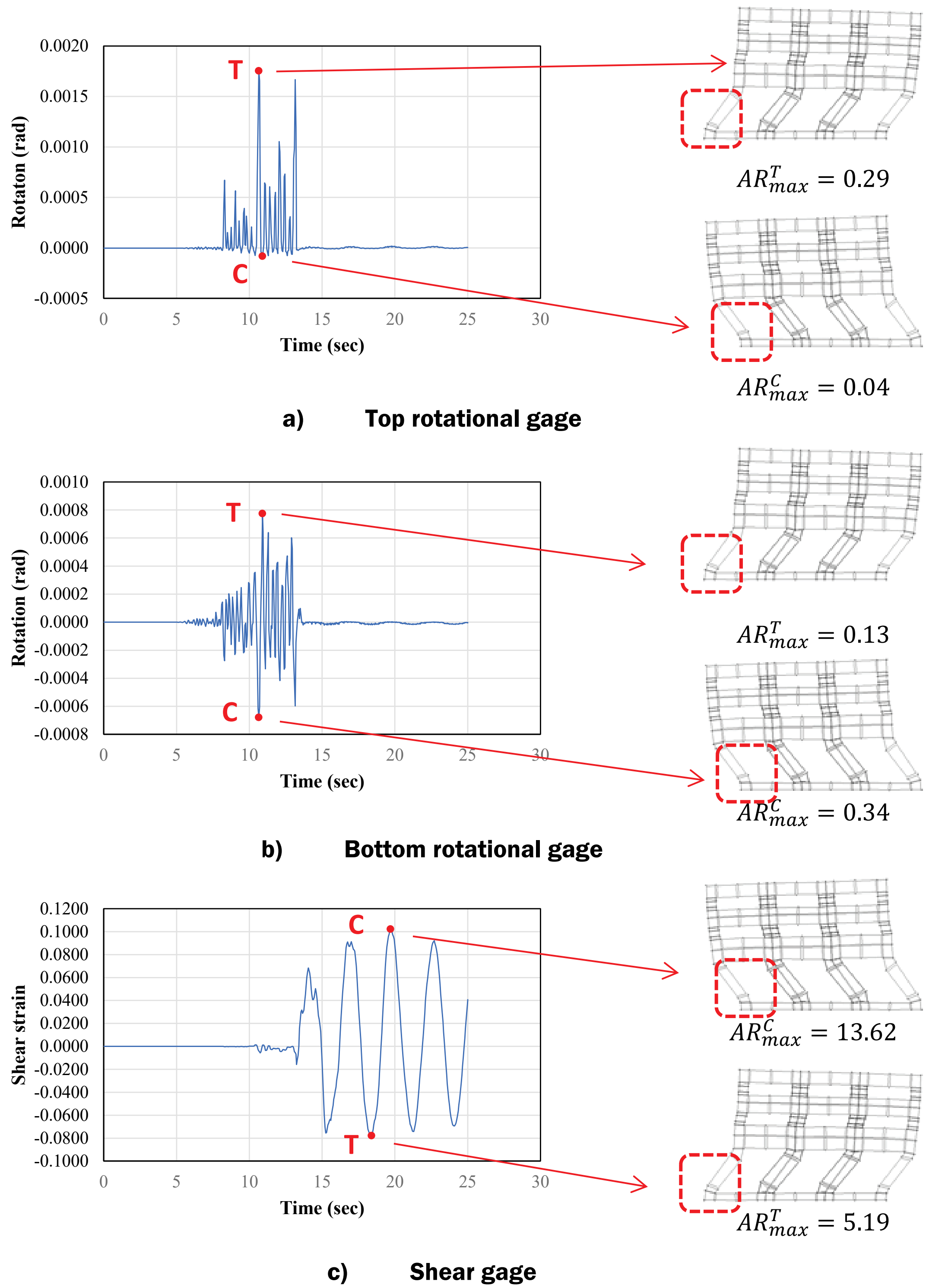

Figure A-9. Nonlinear response history analysis response for Pattern 1 under Loma Prieta earthquake at BSE-1N hazard level.

As can be seen in the tables, Pattern 1 does not have the capacity to withstand the level of shaking imposed by the proposed ground motions. Under both hazard levels, the building piers failed in shear under 5 out of 11 ground motions used. In these cases, all the piers located at Story 1 failed at the same time.

A.4.4.6 FINDINGS

For Pattern 1, the walls are easily adequate for shear demands from the 1961 UBC, with a maximum Acceptance Ratio of 0.57. However, for ASCE/SEI 7-16, they are overstressed at Level 1 with a maximum Acceptance Ratio of 1.08. Thus, the case study building is reasonably representing a 1960s design that would not meet today's demands.

For the ASCE/SEI 41-17 LSP, the maximum Acceptance Ratios are 0.90 and 1.07 at the BSE-1N and BSE-2N hazard levels, respectively. The limitation restrictions are triggered due to the weak story irregularity at the BSE-2N hazard level, and linear procedures are not allowed. For the ASCE/SEI 41-17 NSP, the Acceptance Ratios are 0.43 at the BSE-1N level and 2.92 at the BSE-2N level. Results from the NDP indicate the building does not have enough capacity to withstand the earthquake demand imposed. The building failed in shear under 5 out of 11 of the ground motions used for both hazard levels.

A.4.5 Shear Wall Pattern 2

The following sections summarize the results for the UBC, ASCE/SEI 7-16, and the ASCE/SEI 41-17 LSP, NSP, and NDP for Pattern 2. The same procedures explained in the previous sections were used to calculate the Acceptance Ratios.

A.4.5.1 UBC 1961 ACCEPTANCE RATIOS

Table A-16: Calculation of Seismic Story Force per UBC 1961

Story	Seismic Weight (k)	Seismic Height (ft)	*Z*	*K*	*H* (ft)	*D* (ft)	*T* (s)	*C*	*V* = *ZKCW* (k)	H_x (ft)	F_x (k)	F_{story} (k)
3	800	11								42	95	95
2	900	11	1.0	1.0	42	120	0.19	0.09	**225**	31	79	174
1	900	20								20	51	225
Total	2,600										225	

Table A-17: Check of Wall Pier Capacity per UBC 1961

Story	Pier	l_W (in.)	H_w (in.)	h_w (in.)	f_{allow} (psi)	V_{allow} (k)	V_e (k)	V_e/V_{allow}
3	P1S3	40	48	8	125	40	6	0.16
	P2S3	66	48	8	125	66	18	0.27
	P3S3	66	48	8	125	66	18	0.27
	P4S3	40	48	8	125	40	6	0.16
2	P1S2	41	48	8	125	41	12	0.29
	P2S2	66	48	8	125	66	32	0.48
	P3S2	66	48	8	125	66	32	0.48
	P4S2	41	48	8	125	41	12	0.29
1	P1S1	42	156	8	125	42	21	0.49
	P2S1	66	156	8	125	66	36	0.54
	P3S1	66	156	8	125	66	36	0.54
	P4S1	42	156	8	125	42	21	0.49

A.4.5.2 ASCE/SEI 7-16 ACCEPTANCE RATIOS

Table A-18: Calculation of Story Shear for Pattern 2 per ASCE/SEI 7-16

Story	Seismic Weight (k)	Seismic Height (ft)	k	S_{DS}	R	I_e	C_s	V (k)	h_x (ft)	C_v	F_x (k)	F_{story} (k)	k_{story} (k/in.)
3	800	11	1						42	0.42	147	147	22,690
2	900	11	1	0.67	5	1.0	0.1	**347**	31	0.35	122	268	22,690
1	900	20	1						20	0.23	78	347	8,299
Total	2,600										347		

Table A-19: Check of Wall Pier Capacity per ASCE/SEI 7-16 (1.40*D* + 1.0*L* + 1.0*E*)

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	ϕV_n (k)	V_u (k)	$V_u/\phi V_n$
3	P1S3	40	48	8	32	8	0.24
	P2S3	66	48	8	54	33	0.62
	P3S3	66	48	8	54	29	0.55
	P4S3	40	48	8	32	17	0.52
2	P1S2	41	48	8	33	17	0.52
	P2S2	66	48	8	54	53	0.98
	P3S2	66	48	8	54	49	0.91
	P4S2	41	48	8	33	27	0.82
1	P1S1	42	156	8	34	29	0.85
	P2S1	66	156	8	54	58	1.09
	P3S1	66	156	8	54	57	1.06
	P4S1	42	156	8	34	35	1.02

A.4.5.3 ASCE/SEI 41-17 LSP ACCEPTANCE RATIOS AND LIMITATIONS

Table A-20: Calculation of Story Shear for Pattern 2 per ASCE/SEI 41-17

Story	Seismic Weight (k)	Seismic Height (ft)	k	C_1C_2	C_m	S_a	V (k)	h_x (ft)	C_v	F_x (k)	F_{story} (k)
3	800	11	1	1.1	0.8	0.67	1525	42	0.42	645	645
2	900	11	1					31	0.35	535	1,180
1	900	20	1					20	0.23	345	1,525
Total	2,600									1,525	

Table A-21: Check of Weak and Soft Story Irregularities for Pattern 2 per ASCE/SEI 41-17 at BSE-1N

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	DCR	*m*-factor	Average Shear DCR	Ratio of Average DCR Between this Story and Story Above	Weak Story?	Limit Use of Linear Procedures?	Story Drift Ratio	Soft Story and Limit Use of Linear Static Procedure?
3	P1S3	40	48	8	0.7	2.5	0.9	N/A	N/A	NO	0.06%	NO
	P2S3	66	48	8	1.0	2.5				NO		
	P3S3	66	48	8	1.0	2.5				NO		
	P4S3	40	48	8	0.7	2.5				NO		
2	P1S2	41	48	8	1.3	2.5	1.6	1.77	YES	NO	0.09%	NO
	P2S2	66	48	8	1.8	2.5				NO		
	P3S2	66	48	8	1.8	2.5				NO		
	P4S2	41	48	8	1.3	2.5				NO		
1	P1S1	42	156	8	1.9	2.5	2.1	1.29	NO	NO	0.24%	YES
	P2S1	66	156	8	2.2	2.5				NO		
	P3S1	66	156	8	2.2	2.5				NO		
	P4S1	42	156	8	1.9	2.5				NO		

Table A-22: Check of Wall Piers via LSP of ASCE/SEI 41-17 for Pattern 2

BSE-1N, 0.9*D* + 1.0*E*

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	V_{CE} (k)	V_{UD} (k)	m_{LS}	$\kappa m_{LS} V_{CE}$	Acceptance Ratio $V_{UD}/\kappa m_{LS} V_{CE}$
3	P1S3	40	48	8	67	44	2.5	167	0.26
	P2S3	66	48	8	110	113	2.5	276	0.41
	P3S3	66	48	8	110	111	2.5	276	0.40
	P4S3	40	48	8	67	49	2.5	167	0.29
2	P1S2	41	48	8	69	86	2.5	171	0.50
	P2S2	66	48	8	110	195	2.5	276	0.71
	P3S2	66	48	8	110	194	2.5	276	0.70
	P4S2	41	48	8	69	90	2	137	0.65
1	P1S1	42	156	8	70	132	2.5	176	0.75
	P2S1	66	156	8	110	240	2.5	276	0.87
	P3S1	66	156	8	110	239	2.5	276	0.87
	P4S1	42	156	8	70	134	2	141	0.96

Table A-22: Check of Wall Piers via LSP of ASCE/SEI 41-17 for Pattern 2 (Continued)

BSE-1N, 1.1*D* + 0.275*L* + 1.0*E*

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	V_{CE} (k)	V_{UD} (k)	m_{LS}	$\kappa m_{LS} V_{CE}$	Acceptance Ratio $V_{UD} / \kappa m_{LS} V_{CE}$
3	P1S3	40	48	8	67	43	2.5	167	0.26
	P2S3	66	48	8	110	114	2.5	276	0.41
	P3S3	66	48	8	110	111	2.5	276	0.40
	P4S3	40	48	8	67	50	2.5	167	0.30
2	P1S2	41	48	8	69	85	2.5	171	0.50
	P2S2	66	48	8	110	196	2.5	276	0.71
	P3S2	66	48	8	110	194	2.5	276	0.70
	P4S2	41	48	8	69	91	2	137	0.66
1	P1S1	42	156	8	70	132	2.5	176	0.75
	P2S1	66	156	8	110	240	2.5	276	0.87
	P3S1	66	156	8	110	239	2.5	276	0.87
	P4S1	42	156	8	70	135	2	141	0.96

Table A-23: Check of Weak and Soft Story Irregularities for Pattern 2 per ASCE/SEI 41-17 at BSE-2N

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	DCR	*m*-factor	Average Shear DCR	Ratio of Average DCR Between this Story and Story Above	Weak Story?	Limit Use of linear procedures?	Story Drift Ratio	Soft Story and Limit use of Linear Static Procedure?
3	P1S3	40	48	8	1.0	3.0	1.4	N/A	N/A	NO	0.09%	NO
	P2S3	66	48	8	1.5	3.0				NO		
	P3S3	66	48	8	1.5	3.0				NO		
	P4S3	40	48	8	1.0	3.0				NO		
2	P1S2	41	48	8	1.9	3.0	2.4	1.77	YES	NO	0.14%	NO
	P2S2	66	48	8	2.6	3.0				NO		
	P3S2	66	48	8	2.6	3.0				NO		
	P4S2	41	48	8	1.9	3.0				NO		
1	P1S1	42	156	8	2.8	3.0	3.1	1.29	YES	NO	0.37%	YES
	P2S1	66	156	8	3.2	3.0				YES		
	P3S1	66	156	8	3.2	3.0				YES		
	P4S1	42	156	8	2.8	3.0				NO		

Table A-24: Check of Wall Piers via LSP of ASCE/SEI 41-17 for Pattern 2

BSE-2N, 0.9*D* + 1.0*E*

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	V_{CE} (k)	V_{UD} (k)	m_{CP}	$km_{CP}V_{CE}$	Acceptance Ratio $V_{UD}/km_{CP}V_{CE}$
3	P1S3	40	48	8	67	67	3	201	0.33
	P2S3	66	48	8	110	168	3	331	0.51
	P3S3	66	48	8	110	166	3	331	0.50
	P4S3	40	48	8	67	72	3	201	0.36
2	P1S2	41	48	8	69	130	3	206	0.63
	P2S2	66	48	8	110	292	3	331	0.88
	P3S2	66	156	8	110	290	3	331	0.88
	P4S2	41	156	8	69	134	3	206	0.65
1	P1S1	42	156	8	70	199	3	211	0.94
	P2S1	66	156	8	110	359	3	331	1.08
	P3S1	66	0	8	110	358	3	331	1.08
	P4S1	42	0	8	70	201	3	211	0.95

Table A-24: Check of Wall Piers via LSP of ASCE/SEI 41-17 for Pattern 2 (Continued)

BSE-2N, 1.1*D* + 0.275*L* + 1.0*E*

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	V_{CE} (k)	V_{UD} (k)	m_{CP}	$\kappa m_{CP} V_{CE}$	Acceptance Ratio $V_{UD}/\ \kappa m_{CP} V_{CE}$
3	P1S3	40	48	8	67	67	3	201	0.33
	P2S3	66	48	8	110	169	3	331	0.51
	P3S3	66	48	8	110	166	3	331	0.50
	P4S3	40	48	8	67	73	3	201	0.36
2	P1S2	41	48	8	69	129	3	206	0.63
	P2S2	66	48	8	110	293	3	331	0.88
	P3S2	66	156	8	110	291	3	331	0.88
	P4S2	41	156	8	69	135	3	206	0.65
1	P1S1	42	156	8	70	198	3	211	0.94
	P2S1	66	156	8	110	359	3	331	1.08
	P3S1	66	0	8	110	358	3	331	1.08
	P4S1	42	0	8	70	202	3	211	0.96

A.4.5.4 ASCE/SEI 41-17 NSP ACCEPTANCE RATIOS AND LIMITATIONS

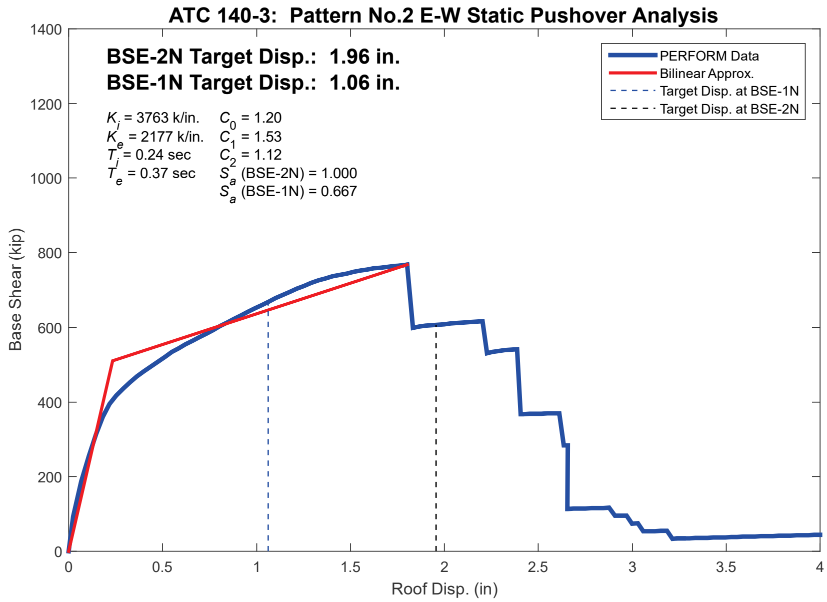

Figure A-10 Target displacements for Shear Wall Pattern No. 2.

Table A-25: Check of Wall Piers via Nonlinear Static Procedure (NSP) of ASCE/SEI 41-17 for Pattern 2

BSE-1N, 1.0*D* + 0.25*L* + 1.0*E*

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	Shear Strain at Target Disp.	Axial at Target Disp. (k)	Target Strain Limit for LS	Acceptance Ratio
3	P1S3	40	48	8	0.01%	-12	1.50%	0.00
	P2S3	66	48	8	0.01%	-14	1.50%	0.00
	P3S3	66	48	8	0.00%	-48	1.50%	0.00
	P4S3	40	48	8	0.00%	-29	1.50%	0.00
2	P1S2	41	48	8	0.10%	-13	1.50%	0.06
	P2S2	66	48	8	0.13%	-43	1.50%	0.09
	P3S2	66	48	8	0.13%	-92	1.50%	0.09
	P4S2	41	48	8	0.01%	-91	0.75%	0.01
1	P1S1	42	156	8	0.17%	23	1.50%	0.11
	P2S1	66	156	8	0.23%	-75	1.50%	0.15
	P3S1	66	156	8	0.27%	-108	0.75%	0.36
	P4S1	42	156	8	0.31%	-214	0.75%	0.42

Table A-25: Check of Wall Piers via Nonlinear Static Procedure (NSP) of ASCE/SEI 41-17 for Pattern 2 (Continued)

BSE-2N, 1.0*D* + 0.25*L* + 1.0*E*

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	Shear Strain at Target Disp.	Axial at Target Disp. (k)	Target Strain Limit for CP	Acceptance Ratio
3	P1S3	40	48	8	0.00%	-20	2.00%	0.00
	P2S3	66	48	8	0.01%	-15	2.00%	0.01
	P3S3	66	48	8	0.01%	-38	2.00%	0.00
	P4S3	40	48	8	0.00%	-34	2.00%	0.00
2	P1S2	41	48	8	0.21%	-22	2.00%	0.10
	P2S2	66	48	8	0.22%	-50	2.00%	0.11
	P3S2	66	48	8	0.25%	-79	2.00%	0.12
	P4S2	41	48	8	0.20%	-96	1.00%	0.20
1	P1S1	42	156	8	0.30%	19	2.00%	0.15
	P2S1	66	156	8	0.56%	-76	2.00%	0.28
	P3S1	66	156	8	0.74%	-170	1.00%	0.74
	P4S1	42	156	8	1.17%	-157	1.00%	1.17

A.4.5.5 ASCE/SEI 41-17 NDP ACCEPTANCE RATIOS AND LIMITATIONS

Table A-26: Pattern 2 Acceptance Ratio at BSE-1N

EQ	Name	Acceptance Ratio				Comments
		Shear	Rotation Top	Rotation Bottom	Max	
1	Imperial Valley	0.23	0.75	0.83	0.83	Analysis finished
2	Loma Prieta	0.99	1.72	2.06	2.06	All piers at story 1 reached LS limit state in flexure
3	Northridge, Sylmar CSE	0.90	1.64	2.02	2.02	Pie 2 to P4 at story 1 reached LS limit state in flexure
4	Northridge, Sylmar OVM	4.07	1.73	1.87	4.07	All piers at story 1 reached LS limit state in shear
5	Chi Chi TCU079	2.38	1.92	2.14	2.38	All piers at story 1 reached LS limit state in flexure.
6	Chi Chi TCU122	0.16	0.68	0.75	0.75	Analysis finished
7	Duzce	0.56	1.58	1.72	1.72	All piers at story 1 reached LS limit state in flexure
8	Chetsu 65010	0.48	1.38	1.56	1.56	Analysis finished
9	Chetsu 65025	0.13	0.64	0.72	0.72	Analysis finished
10	Mexico, Chihuahua	0.14	0.65	0.71	0.71	Analysis finished
11	Mexico, Ejido Saltillo	0.13	0.64	0.72	0.72	Analysis finished
AVERAGE		0.93	1.21	1.37	1.37	

Table A-27: Pattern 2 Acceptance Ratio at BSE-2N

EQ	Name	Acceptance Ratio				Comments
		Shear	Rotation Top	Rotation Bottom	Max	
1	Imperial Valley	0.21	0.49	0.54	0.54	Analysis finished
2	Loma Prieta	3.49	1.06	1.02	3.49	All piers at story 1 reached CP limit state in shear
3	Northridge, Sylmar CSE	11.41	1.05	1.00	11.41	All piers at story 1 reached CP limit state in shear
4	Northridge, Sylmar OVM	6.33	0.64	0.74	6.33	All piers at story 1 reached CP limit state in shear
5	Chi Chi TCU079	6.27	1.07	0.96	6.27	All piers at story 1 reached CP limit state in shear
6	Chi Chi TCU122	0.14	0.43	0.42	0.43	Analysis finished
7	Duzce	0.71	0.97	1.07	1.07	Analysis finished
8	Chetsu 65010	6.78	0.95	1.02	6.78	All piers at story 1 reached CP limit state in shear
9	Chetsu 65025	0.16	0.39	0.45	0.45	Analysis finished
10	Mexico, Chihuahua	0.19	0.50	0.47	0.50	Analysis finished
11	Mexico, Ejido Saltillo	0.12	0.36	0.39	0.39	Analysis finished
	AVERAGE:	3.26	0.72	0.74	3.26	

A.4.5.5 FINDINGS

For Pattern 2, the walls are easily adequate for shear demands from the 1961 UBC, with a maximum Acceptance Ratio of 0.54. However, for ASCE/SEI 7-16, they are overstressed at Levels 1 with maximum Acceptance Ratios of 1.09. As before, the building analyzed represent a 1960s building that will fail under modern building code demand.

For the ASCE/SEI 41-17 LSP, the maximum Acceptance Ratio is 0.96 and 1.08 for BSE-1N and BSE-2N hazard levels, respectively. Once again, the linear procedure limitation provision restrictions are triggered, and linear procedures are not allowed. For the ASCE/SEI 41-17 NSP, the Acceptance Ratios are 0.42 at the BSE-1N level and 1.17 at the BSE-2N level. Results from the NDP indicate the building does not have enough capacity to withstand the earthquake demand imposed. The building failed in shear under 2 and 5 out of 11 of the ground motions used for BSE-1N and BSE-2N, respectively. The building piers failed in moment under 6 and 4 out of 11 of the ground motions used for BSE-1N and BSE-2N, respectively.

A.4.6 Shear Wall Pattern 3

A.4.6.1 UBC 1961 ACCEPTANCE RATIOS

Table A-28: Calculation of Seismic Story Force per UBC 1961

Story	Seismic Weight	Seismic Height	*Z*	*K*	*H*	*D*	*T*	*C*	*I=ZKCW*	h_x	F_x	F_{story}
	(k)	(ft)			(ft)	(ft)	(s)		(k)	(ft)	(k)	(k)
3	800	14							42	106	106	42
2	900	14	1.0	1.0	42	120	0.19	0.09	225	28	80	186
1	900	14							14	40	225	14
Total	2,600									225		

Table A-29: Check of Wall Pier Capacity per UBC 1961

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	f_{allow} (psi)	v_{allow} (k)	V_E (k)	V_E/v_{allow}
3	P1S3	100	84	8	125	100	5	0.05
	P2S3	186	84	8	125	186	22	0.12
	P3S3	186	84	8	125	186	22	0.12
	P4S3	100	84	8	125	100	5	0.05
2	P1S2	71	84	8	125	71	11	0.15
	P2S2	126	84	8	125	126	36	0.28
	P3S2	126	84	8	125	126	36	0.28
	P4S2	71	84	8	125	71	11	0.15
1	P1S1	42	84	8	125	42	19	0.44
	P2S1	66	84	8	125	66	38	0.57
	P3S1	66	84	8	125	66	38	0.57
	P4S1	42	84	8	125	42	19	0.44

A.4.6.2 ASCE/SEI 7-16 ACCEPTANCE RATIOS

Table A-30: Calculation of Story Shear for Pattern 3 per ASCE/SEI 7-16

Story	Seismic Weight (k)	Seismic Height (ft)	k	S_{DS}	R	I_e	C_s	V (k)	h_x (ft)	C_v	F_x (k)	F_{story} (k)	k_{story} (k/in.)
3	800	11	1						42	0.42	147	147	22,690
2	900	11	1	0.67	5	1.0	0.1	**347**	31	0.35	122	268	22,690
1	900	20	1						20	0.23	78	347	8,299
Total	2,600										347		

Table A-31: Check of Wall Pier Capacity per ASCE/SEI 7-16 (1.40 D + 1.0 L + 1.0 E)

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	ϕV_n (k)	V_u (k)	$V_u/\phi V_n$
3	P1S3	100	84	8	81	12	0.14
	P2S3	186	84	8	151	42	0.28
	P3S3	186	84	8	151	35	0.23
	P4S3	100	84	8	81	14	0.17
2	P1S2	71	84	8	58	17	0.30
	P2S2	126	84	8	102	58	0.57
	P3S2	126	84	8	102	53	0.52
	P4S2	71	84	8	58	33	0.57
1	P1S1	42	84	8	34	26	0.76
	P2S1	66	84	8	54	56	1.05
	P3S1	66	84	8	54	55	1.02
	P4S1	42	84	8	34	35	1.04

A.4.6.3 ASCE/SEI 41-17 LSP ACCEPTANCE RATIOS AND LIMITATIONS

Table A-32: Calculation of Story Shear for Pattern 3 per ASCE/SEI 41-17

Story	Seismic Weight	Seismic Height	k	C_1C_2	C_m	S_a	V	h_x	C_v	F_x	F_{story}
	(k)	(ft)					(k)	(k)		(k)	(k)
3	800	11	1					42	0.42	645	645
2	900	11	1	1.1	0.8	0.67	1525	31	0.35	535	1,180
1	900	20	1					20	0.23	345	1,525
Total	2,600									1,525	

Table A-33: Check of Weak and Soft Story Irregularities for Pattern 3 per ASCE/SEI 41-17 for BSE-1N

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	DCR	*m*-factor	Average Shear DCR	Ratio of Average DCR Between this Story and Story Above	Limit Use of Linear Procedures?	Story Drift Ratio	Soft story and Limit use of Linear Static Procedure?
3	P1S3	100	84	8	0.3	2.5	0.4	N/A	NO	0.02%	NO
	P2S3	186	84	8	0.4	2.5			NO		
	P3S3	186	84	8	0.4	2.5			NO		
	P4S3	100	84	8	0.3	2.5			NO		
2	P1S2	71	84	8	0.8	2.5	1.0	2.46	NO	0.06%	YES
	P2S2	126	84	8	1.0	2.5			NO		
	P3S2	126	84	8	1.0	2.5			NO		
	P4S2	71	84	8	0.8	2.5			NO		
1	P1S1	42	84	8	1.8	2.5	2.0	2.09	NO	0.11%	YES
	P2S1	66	84	8	2.1	2.5			NO		
	P3S1	66	84	8	2.1	2.5			NO		
	P4S1	42	84	8	1.8	2.5			NO		

Table A-34: Check of Wall Piers via LSP of ASCE/SEI 41-17 for Pattern 3 for BSE-1N

BSE-1N, 0.9*D* + 1.0*E*

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	V_{CE} (k)	V_{UD} (k)	m_{LS}	$\kappa m_{CP} V_{CE}$	Acceptance Ratio $V_{UD} / \kappa m_{LS} V_{CE}$
3	P1S3	100	84	8	167	46	2.5	418	0.11
	P2S3	186	84	8	311	136	2.5	778	0.17
	P3S3	186	84	8	311	133	2.5	778	0.17
	P4S3	100	84	8	167	47	2.5	418	0.11
2	P1S2	71	84	8	119	96	2.5	297	0.32
	P2S2	126	84	8	211	215	2.5	527	0.41
	P3S2	126	84	8	211	213	2.5	527	0.40
	P4S2	71	84	8	119	102	2.5	297	0.34
1	P1S1	42	84	8	70	125	2.5	176	0.71
	P2S1	66	84	8	110	231	2.5	276	0.84
	P3S1	66	84	8	110	230	2.5	276	0.83
	P4S1	42	84	8	70	129	2	141	0.92

Table A-34 (Continued)

BSE-1N, 1.1*D* + 1.1*L* + 1.0*E*

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	V_{CE} (k)	V_{UD} (k)	m_{LS}	$\kappa m_{CP} V_{CE}$	Acceptance Ratio $V_{UD} / \kappa m_{LS} V_{CE}$
3	P1S3	100	84	8	167	46	2.5	418	0.11
	P2S3	186	84	8	311	137	2.5	778	0.18
	P3S3	186	84	8	311	133	2.5	778	0.17
	P4S3	100	84	8	167	48	2.5	418	0.11
2	P1S2	71	84	8	119	94	2.5	297	0.32
	P2S2	126	84	8	211	216	2.5	527	0.41
	P3S2	126	84	8	211	213	2.5	527	0.40
	P4S2	71	84	8	119	104	2.5	297	0.35
1	P1S1	42	84	8	70	125	2.5	176	0.71
	P2S1	66	84	8	110	231	2.5	276	0.84
	P3S1	66	84	8	110	230	2.5	276	0.83
	P4S1	42	84	8	70	130	2	141	0.93

Table A-35 Check of Weak and Soft Story Irregularities for Pattern 3 per ASCE/SEI 41-17 for BSE-2N

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	DCR	*m*-factor	Average Shear DCR	Ratio of Average DCR Between this Story and Story Above	Limit Use of Linear Procedures?	Story Drift Ratio	Soft story and Limit use of Linear Static Procedure?
3	P1S3	100	84	8	0.4	3.0	0.6	N/A	NO	0.04%	NO
	P2S3	186	84	8	0.6	3.0			NO		
	P3S3	186	84	8	0.6	3.0			NO		
	P4S3	100	84	8	0.4	3.0			NO		
2	P1S2	71	84	8	1.2	3.0	1.4	2.46	NO	0.08%	YES
	P2S2	126	84	8	1.5	3.0			NO		
	P3S2	126	84	8	1.5	3.0			NO		
	P4S2	71	84	8	1.2	3.0			NO		
1	P1S1	42	84	8	2.7	3.0	3.0	2.09	NO	0.17%	YES
	P2S1	66	84	8	3.1	3.0			YES		
	P3S1	66	84	8	3.1	3.0			YES		
	P4S1	42	84	8	2.7	3.0			NO		

Table A-36 Check of Wall Piers via LSP of ASCE/SEI 41-17 for Pattern 3 for BSE-2N

BSE-2N, 0.9*D* + 1.0*E*

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	V_{CE} (k)	V_{UD} (k)	m_{CP}	$\kappa m_{CP} V_{CE}$	Acceptance Ratio $V_{UD}/\kappa m_{CP} V_{CE}$
3	P1S3	100	84	8	167	70	3	502	0.14
	P2S3	186	84	8	311	202	3	933	0.22
	P3S3	186	84	8	311	199	3	933	0.21
	P4S3	100	84	8	167	71	3	502	0.14
2	P1S2	71	84	8	119	145	3	356	0.41
	P2S2	126	84	8	211	321	3	632	0.51
	P3S2	126	84	8	211	319	3	632	0.50
	P4S2	71	84	8	119	151	3	356	0.43
1	P1S1	42	84	8	70	189	3	211	0.90
	P2S1	66	84	8	110	346	3	331	1.04
	P3S1	66	84	8	110	345	3	331	1.04
	P4S1	42	84	8	70	193	3	211	0.92

Table A-36 (Continued)

BSE-2N, 1.1*D* + 1.1*L* + 1.0*E*

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	V_{CE} (k)	V_{UD} (k)	m_{CP}	$\kappa m_{CP} V_{CE}$	Acceptance Ratio $V_{UD} / \kappa m_{CP} V_{CE}$
3	P1S3	100	84	8	167	70	3	502	0.14
	P2S3	186	84	8	311	203	3	933	0.22
	P3S3	186	84	8	311	199	3	933	0.21
	P4S3	100	84	8	167	71	3	502	0.14
2	P1S2	71	84	8	119	144	3	356	0.40
	P2S2	126	84	8	211	322	3	632	0.51
	P3S2	126	84	8	211	319	3	632	0.50
	P4S2	71	84	8	119	153	3	356	0.43
1	P1S1	42	84	8	70	188	3	211	0.89
	P2S1	66	84	8	110	346	3	331	1.05
	P3S1	66	84	8	110	345	3	331	1.04
	P4S1	42	84	8	70	194	3	211	0.92

A.4.6.4 ASCE/SEI41-17 NSP ACCEPTANCE RATIOS AND LIMITATIONS

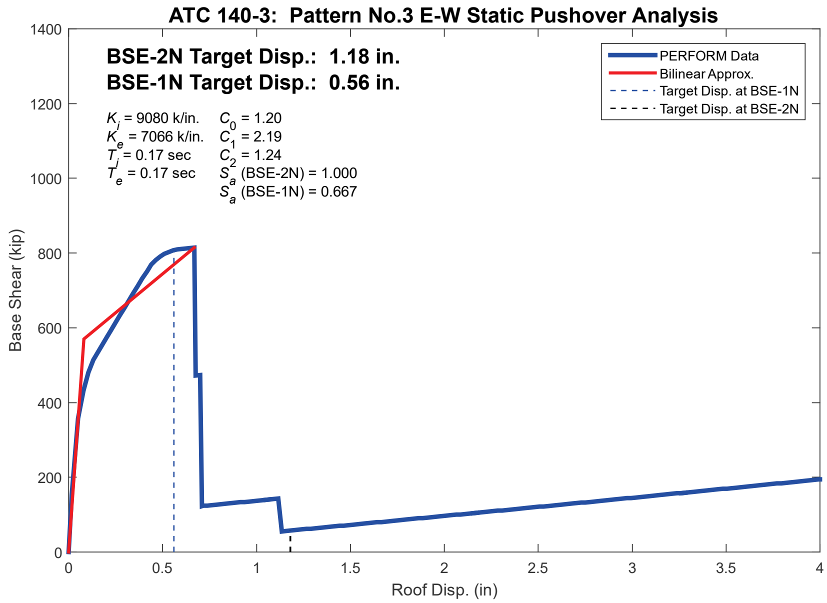

Figure A1-11 Target displacements for Shear Wall Pattern No. 3.

Table A-37 Check of Wall Piers via Nonlinear Static Procedure (NSP) of ASCE/SEI 41-17 for Pattern 3

BSE-1N, 1.0*D* + 0.25*L* + 1.0*E*

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	Shear Strain at Target Disp.	Axial at Target Disp. (k)	Target Strain Limit for LS	Acceptance Ratio
3	P1S3	100	84	8	0.00%	-48	1.50%	0.00
	P2S3	186	84	8	0.00%	-25	1.50%	0.00
	P3S3	186	84	8	0.00%	-11	1.50%	0.00
	P4S3	100	84	8	0.00%	-14	1.50%	0.00
2	P1S2	71	84	8	0.01%	-138	0.75%	0.01
	P2S2	126	84	8	0.01%	-63	1.50%	0.01
	P3S2	126	84	8	0.00%	-16	1.50%	0.00
	P4S2	71	84	8	0.00%	-11	1.50%	0.00
1	P1S1	42	84	8	0.50%	-189	0.75%	0.66
	P2S1	66	84	8	0.62%	-87	1.50%	0.41
	P3S1	66	84	8	0.59%	-41	1.50%	0.39
	P4S1	42	84	8	0.39%	-36	1.50%	0.26

Table A-37 (Continued)

BSE-2N, 1.0*D* + 0.25*L* + 1.0*E*

Story	Pier	l_w (in.)	H_w (in.)	h_w (in.)	Shear Strain at Target Disp.	Axial at Target Disp. (k)	Target Strain Limit for CP	Acceptance Ratio
3	P1S3	100	84	8	0.00%	-22	2.00%	0.00
	P2S3	186	84	8	0.00%	-24	2.00%	0.00
	P3S3	186	84	8	0.00%	-21	2.00%	0.00
	P4S3	100	84	8	0.00%	-16	2.00%	0.00
2	P1S2	71	84	8	0.00%	-46	2.00%	0.00
	P2S2	126	84	8	0.01%	-55	2.00%	0.01
	P3S2	126	84	8	0.00%	-50	2.00%	0.00
	P4S2	71	84	8	0.00%	-29	2.00%	0.00
1	P1S1	42	84	8	2.13%	-53	2.00%	1.07
	P2S1	66	84	8	2.13%	-71	2.00%	1.07
	P3S1	66	84	8	2.12%	-69	2.00%	1.06
	P4S1	42	84	8	0.50%	-43	2.00%	0.25

A.4.6.5 FINDINGS

For Pattern 3, the walls are easily adequate for shear demands from the 1961 UBC, with a maximum Acceptance Ratio of 0.57. However, for ASCE/SEI 7-16, they are overstressed at Level 1 with a maximum Acceptance Ratio of 1.05.

For the ASCE/SEI 41-17 LSP, the maximum Acceptance Ratio is 0.93 and 1.05 for both BSE-1N and BSE-2N hazard levels. The linear procedure limitation provision restrictions are triggered, and linear procedures are not allowed. For the ASCE/SEI41-17 NSP, the Acceptance Ratios are 0.66 at the BSE-1N level and 1.07 at the BSE-2N level.

A.5 Conclusions

A.5.1 Summary of Findings

The findings of the three building patterns that have been studied can be summarized below:

- All three building patterns that have been studied show adequate performance under the UBC 1961 provisions with Acceptance Ratios less than 0.60.
- All three building patterns are overstressed for the ASCE/SEI 7-16 loads with a maximum Acceptance Ratio of 1.08 at Story 1.
- All three building patterns show adequate Acceptance Ratios for the ASCE/SEI 41-17 LSP loads under the BSE-1N seismic hazard level, but they are overstressed at the BSE-2N seismic hazard level, with worst case Acceptance Ratios of 0.96 and 1.08, respectively.
- The linear procedure limitation provision restrictions are triggered for all three patterns, such that linear procedures are not permitted.
- When checked for ASCE/SEI 41-17 NSP, similar results were obtained in all the buildings studied. A maximum Acceptance Ratio of 0.66 was obtained at the lower hazard level. When the seismic demand increase, all three buildings are overstressed with a maximum Acceptance Ratio of 2.92 was calculated.
- For NDP, the Pattern 1 and Pattern 2 buildings both failed to provide enough shear capacity at both hazard levels. Pattern 1 failed in shear at 5 out of 11 ground motions at BSE-1N and BSE-2N. At the BSE-1N hazard level, Pattern 2 failed in shear under 2 ground motions and in flexure under 6 ground motions. At the BSE-2N hazard level, the building failed in shear and flexure under 5 and 4 ground motions, respectively.
- For both Patterns 1 and 2, the LSP results are close to meeting the performance objective, but the NDP results show the building does meet the objective. Thus, the linear procedure limitation provision result of not permitted is appropriate because it prevents a misleading, unconservative conclusion from being drawn based on the LSP.

A.5.2 Issues and Conclusions

The three building patterns produced inconsistent findings and conclusions regarding the limitations that are being examined.

A.5.2.1 BUILDING PATTERN 1

For Building Pattern 1, the following conclusions have been obtained, as shown in Table A1-38.

- It is unclear whether ASCE/SEI 7-16 can predict the Acceptance Ratio for ASCE/SEI 41-17 NSP. At the BSE-1N hazard level, there is a large discrepancy between the results of the two methods with 1.08 from ASCE/SEI 7-16 and 0.43 from ASCE/SEI 41-17, whereas at the BSE-2N hazard level, NSP predict a greater Acceptance Ratio, with values of 1.50 and 2.92, respectively.

- Looking at the Acceptance Ratios produced by the ASCE/SEI 41-17 LSP and NSP procedures, a similar conclusion can be reached. A large discrepancy was found between both methods at both hazard levels. Acceptance Ratios of 0.90 and 0.43 were obtained at the BSE-1N hazard level for LSP and NSP, respectively. When comparing values at the BSE-2N hazard level, the ratios were 1.07 and 2.92. At the lower hazard level, both methods predict acceptable behavior of the building, whereas the contrary was observed at the BSE-2N hazard level.

- No conclusion can be made from the NDP results obtained since the building does not have enough capacity at both the BSE-1N and BSE-2N hazard levels.

- The intent of the limitation provisions is presumably to prevent using less conservative results from the presumably less accurate linear procedures when the results from the nonlinear procedures are more conservative. Answering the question of whether the limitation of LSP is appropriate, it appears that at BSE-1N level the LSP is more conservative than the NSP, with Acceptance Ratios of 0.90 and 0.43 respectively, and thus the limitation is not appropriate regardless of the fact both methods predict acceptable behavior of the building. Contrary to that, at a BSE-2N hazard level, the opposite holds true, since the Acceptance Ratios are 1.07 and 2.92 for the LSP and NSP respectively, and thus the limitation is appropriate, although the Acceptance Ratios at this hazard level are considerably different between themselves.

Table A-38: Building Pattern 1 Summary

Code	Acceptance Ratio	ASCE/SEI 7-16 Bounds NSP?	LSP Bounds NSP?	ASCE/SEI 7-16 Bounds NDP	LSP Bounds NDP	LSP Permitted?	Limitation Provision is Appropriate?
1961 UBC	0.57	--	--	--	--	--	--
ASCE/SEI 7-16	1.08	--	--	--	--	--	--
LSP, BSE-1N	0.90	--	--	--	--	Not Permitted	No
LSP, BSE-2N	1.07	--	--	--	--	Not Permitted	Yes
NSP, BSE-1N	0.43	Yes: 1.08 > 0.43	Yes: 0.90 > 0.43	--	--	--	Too conservative
NSP, BSE-2N	2.92	No: 1.08 < 2.92	No: 1.07 < 2.92	--	--	--	Yes
NDP, BSE-1N	Fail	--	--	No	No	--	Yes
NDP, BSE-2N	Fail	--	--	No	No	--	Yes

A.5.2.2 BUILDING PATTERN 2

For Building Pattern 2, the following conclusions have been obtained as shown in Table A1-39.

- We observe that in this building pattern the provisions of ASCE/SEI 7-16 cannot predict the building performance under ASCE/SEI 41-17 NSP, as the Acceptance Ratio of 1.02 that is obtained from ASCE/SEI 7-16 is far from 0.42 and 1.17 Acceptance Ratios from NSP.

- Similarly, the ASCE/SEI 41-17 LSP does not predict the NSP Acceptance Ratios at the BSE-1N hazard level since there is a significant difference in the results (1.02 vs 0.42). More consistent Acceptance Ratios were obtained at the BSE-2N hazard level (1.02 vs 1.08).

- The limitation provision for the LSP is appropriate in this case for the BSE-1N hazard level contrary to what was observed at the BSE-2N hazard level. The Acceptance Ratios that were obtained from the LSP are much higher compared to the NSP at the lower hazard levels and thus more conservative.

- Similar to Pattern 1, no conclusion can be reached from the NDP results.

Table A-39: Building Pattern 2 Summary

Code	Acceptance Ratio	ASCE/SEI 7-16 Bounds NSP?	LSP Bounds NSP?	ASCE/SEI 7-16 Bounds NDP	LSP Bounds NDP	LSP Permitted?	Limitation Provision is Appropriate?
1961 UBC	0.57	--	--	--	--	--	--
ASCE/SEI 7-16	1.02	--	--	--	--	--	--
LSP, BSE-1N	0.96	--	--	--	--	Not Permitted	No
LSP, BSE-2N	1.08	--	--	--	--	Not Permitted	Yes
NSP, BSE-1N	0.42	Yes: 1.02 > 0.43	Yes: 0.96 > 0.42	--	--	--	Too conservative
NSP, BSE-2N	1.17	No: 1.02 < 1.17	No: 1.08 < 1.17	--	--	--	Yes
NDP, BSE-1N	Fail	--	--	No	No	--	Yes
NDP, BSE-2N	Fail	--	--	No	No	--	Yes

A.5.2.3 BUILDING PATTERN 3

For Building Pattern 3, the following conclusions have been obtained, as shown in Table A1-40.

- Similar to Pattern 1, it is observed that it is unclear whether ASCE/SEI 7-16 can predict the Acceptance Ratio for ASCE/SEI 41 NSP. At the BSE-1N hazard level, there a large discrepancy between the results of the two methods, whereas at the BSE-2N hazard level they are reasonably close.

- Looking at the Acceptance Ratios produced by the ASCE/SEI 41 LSP and NSP procedures, the LSP can reasonably predict the Acceptance Ratio for NSP at the BSE-2N hazard level; however, a discrepancy when comparing values at the BSE-1N hazard level is observed.

- Answering the question of whether the linear procedure limitation of LSP is appropriate, it appears that at the BSE-1N hazard level the LSP is more conservative than the NSP, and thus the limitation is not appropriate. Contrary to that, at a BSE-2N hazard level, the opposite holds true, and thus the limitation appears appropriate.

- NDP analyses were not conducted for Building Pattern 3.

Table A-40: Building Pattern 3 Summary

Code	Acceptance Ratio	ASCE/SEI 7-16 Bounds NSP?	LSP Bounds NSP?	LSP Permitted?	Limitation Provision is Appropriate?
1961 UBC	0.57	--	--	--	--
ASCE/SEI 7-16	1.04	--	--	--	--
LSP, BSE-1N	0.93	--	--	Not Permitted	--
LSP, BSE-2N	1.04	--	--	Not Permitted	--
NSP, BSE-1N	0.66	Yes: 1.04 > 0.66	Yes: 0.93 > 0.66	--	Too conservative
NSP, BSE-2N	1.07	No: 1.04 < 1.07	No: 1.04 < 1.07	--	Yes

A.5.2.4 CLOSING

The WG1 case studies summarized above formed a portion of the analysis used to evaluate the ASCE/SEI 41-17 linear limitation provisions. See Part 1, Chapter 1 for a broader perspective incorporating other case studies and research, as well as the resulting code change proposal and rationale.

A.7 References

ACI, 2014, *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-14, American Concrete Institute.

ASCE, 2017, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE/SEI 7-16 Report, American Society of Civil Engineers Structural Engineering Institute, Reston, Virginia.

FEMA, 2018, *Example Application Guide for ASCE/SEI 41-13 Seismic Evaluation and Retrofit of Existing Buildings with Additional Commentary for ASCE/SEI 41-17*, FEMA P-2006 Report, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

ICBO, 1961, 1961 Uniform Building Code, Volume 1, International Conference of Building Officials, Los Angeles, California.

Part 3, Appendix A: Derivation of Equations for Case 3

A.1 Dimensions *X* and *Y* when the soil pressure distribution forms a rectangle and a triangle

When the soil pressure distribution under the footing forms a combination of a rectangle and a triangle as shown in the Figure A-1, let *X* represent the length of the rectangular portion and *Y* the length of the triangular portion.

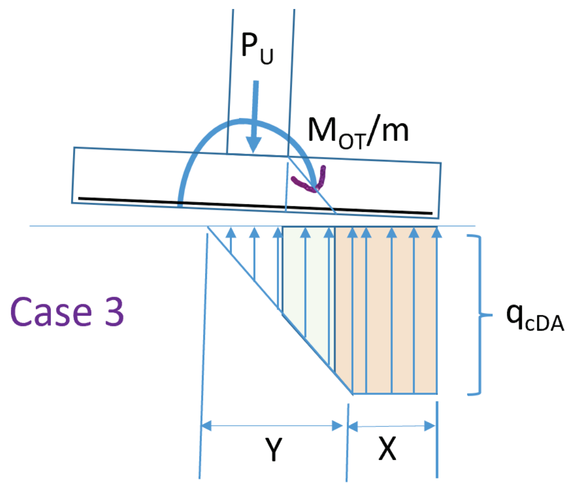

Figure A-1 Soil pressure distribution under the footing forms a rectangle and a triangle.

Taking moments about the center line of the footing:

$$M_{UD} = \ q_{cDA} B_f X \left(\frac{L_f}{2} - \frac{X}{2}\right) + \frac{q_{cDA} B_f Y}{2}\left(\frac{L_f}{2} - X - \frac{1}{3}Y\right);$$

or

$$\frac{2M_{UD}}{q_{cDA} B_f} = \ L_f X - X^2 + \frac{L_f Y}{2} - XY - \frac{Y^2}{3};$$

or

$$\frac{2M_{UD}}{q_{cDA} B_f} = \ L_f\left(X + \frac{Y}{2}\right) - X^2 - XY - \frac{Y^2}{3} \qquad Eq. A - 1$$

Summing the axial forces on the footing

$$\left(X+\frac{Y}{2}\right)=\frac{P_U}{q_{cDA}B_f} \qquad Eq.A-2$$

Substituting equation A-2 in equation A-1.

$$\frac{2M_{UD}}{q_{cDA}B_f}=\frac{P_U L_f}{q_{cDA}B_f}-X^2-XY-\frac{Y^2}{3}; \qquad Eq.A-3$$

From equation A-2

$$X=\frac{P_U}{q_{cDA}B_f}-\frac{Y}{2};$$

Therefore,

$$X^2=\left(\frac{P_U}{q_{cDA}B_f}\right)^2-\frac{P_U}{q_{cDA}B_f}Y+\frac{Y^2}{4};$$

And

$$XY=\frac{P_U Y}{q_{cDA}B_f}-\frac{1}{2}Y^2;$$

Substituting for X² and XY in Eq, A3 and expanding we get:

$$\frac{2M_{UD}}{q_{cDA}B_f}-\frac{P_U L_f}{q_{cDA}B_f}=-\left\{\left(\frac{P_U}{q_{cDA}B_f}\right)^2+\frac{Y^2}{12}\right\}$$

or

$$\frac{Y^2}{12}=\frac{P_U L_f}{q_{cDA}B_f}-\frac{2M_{UD}}{q_{cDA}B_f}-\left(\frac{P_U}{q_{cDA}B_f}\right)^2$$

or

$$Y=\sqrt{12\left\{\frac{P_U L_f}{q_{cDA}B_f}-\frac{2M_{UD}}{q_{cDA}B_f}-\left(\frac{P_U}{q_{cDA}B_f}\right)^2\right\}}$$

Let

$$P'=\frac{P_U}{q_{cDA}B_f}$$

$$M'=\frac{M_{UD}}{q_{cDA}B_f}$$

Then

$$Y = \sqrt{12\{P'L_f - 2M' \ - P'^2\}}$$

and

$$X = P' - \frac{1}{2}Y$$

Appendix B: Archetype Building 1 Concrete Moment Frame with Concrete Shear Walls

B.1 Motivation

Previous seismic rehabilitation guidelines such as FEMA 356 *Prestandard and Commentary for the Seismic Rehabilitation of Buildings,* and later the ASCE 41/SEI -06 *Seismic Rehabilitation of Existing Building* have both provided guidelines for foundation analysis and retrofit design. ASCE/SEI 41-13 and -17 sought to improve these guidelines and provide more accurate results. However, there are still a number of areas for improvement within these provisions. The WG-2 Foundation Working Group was tasked with evaluating the shallow foundation provisions in the ASCE/SEI 41-17 standard for clarity, usability, and technical content and providing recommendations and code change proposals as input for deliberation by the ASCE/SEI 41-23 committee to be incorporated in the next ASCE/SEI standard update.

To provide historical context, ASCE/SEI 41-06 foundation provisions utilized linear procedures to incorporate soil-structure interaction which included kinematic and foundation damping. However, for flexible base modeling, both FEMA 356 and ASCE/SEI 41-06 allowed for infinite ductility if a spring was added in modeling; the soil strength was not required to be evaluated. Research has shown that this soil bearing with infinite ductility assumption can be correct when the axial forces on the foundation (both gravity and earthquake) are low. However, it is not always the case, and can cause an underestimation of deformations (transient, during the earthquake, and permanent) in the superstructure when axial forces are higher. In addition, given the infinite ductility assumption, acceptance criteria was not provided for the flexible base modeling case. Lastly, the Method 1 soil stiffness assumed that the footing was rigid, and the soil remained elastic and in contact over the entire bottom of footing surface, which can overestimate the soil stiffness by a significant amount.

ASCE 41/SEI -06 also decoupled the rocking and yielding mechanisms and had separate checks for them, despite that they do not occur independently. In ASCE/SEI 41-13, the decoupled rocking issue was addressed with the addition of *m*-factor tables and nonlinear acceptance criteria for these actions that are a function of the soil stiffness and gravity loads on the foundations. In addition, ASCE/SEI 41-13 revised the soil-foundation-structure interaction provisions and added limitations. The fundamental concept of both the ASCE/SEI 41-13 revision and the ASCE/SEI 41-17 update is that if the acceptance criteria of the foundation chapter are satisfied, regardless of the methodology used (subject to limitations of each method), then the foundation deformations are accurate enough, and the analysis is suitable for determining the component level acceptance criteria of the superstructure. This philosophy is retained in the changes ATC is proposing for ASCE 41-23.

However, several issues with the ASCE/SEI 41-17 foundation chapter have been identified.

- There are large gaps in the ASCE/SEI 41-17 linear procedure process including:
  - A lack of clarity on when a fixed-base assumption is permitted leads to confusion about what analysis provisions to follow.
  - The rigidity of the footing relative to the soil must be determined, in order to establish the applicable analysis method. However, the method provided for the relative rigidity determination is in the commentary section and does not take into account that soil separates from the footing during rocking action.
  - Linear provisions for footings that are flexible relative to soil are not provided (Method 3 did not provide provisions for linear procedures). Therefore, the user has no guidance on evaluating strip (combined) footing or mat foundation conditions with a lack of structural footing stiffness or strength that would classify the structural component of the foundation as flexible relative to the soil.

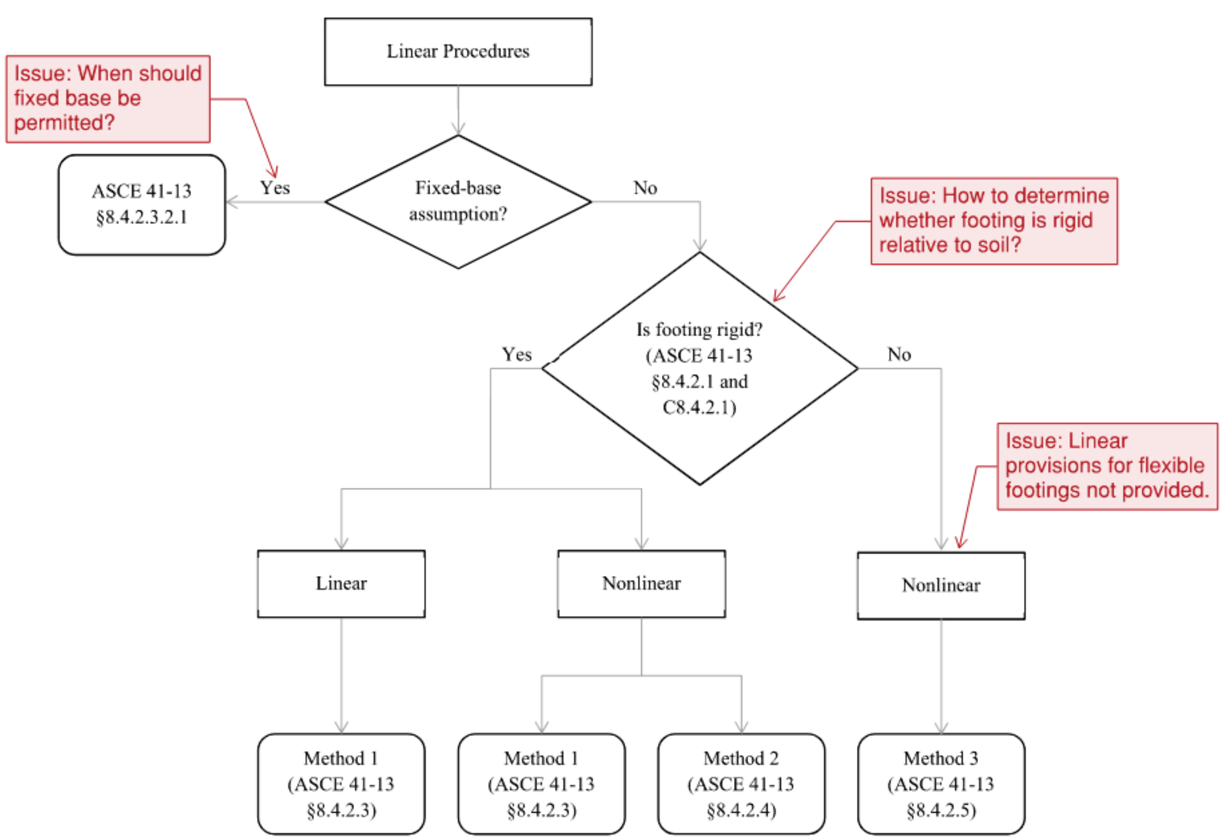

Figure B-1 ASCE/SEI 41-13 linear procedures flow chart with gaps identified.

- Some of the provisions require a flexible-base analysis to be done in addition to a fixed-base analysis to determine if the results from the fixed-base procedures could be used.
- The prescriptive soil properties permitted to be used when soils information is not available are so low that it would most certainly require soil exploration to be conducted for evaluation of the foundations for almost all buildings regardless of their foundation capacity or level of seismicity. Further, the prescriptive bearing capacities are inconsistent between the two methods provided; expected bearing capacities based on the calculated gravity loads to existing footings per ASCE/SEI 41-17 Equation 8-3 are conservative when compared to expected bearing capacity based on Equations 8-1.
- Method 2 is a complicated process and it is unclear if the complexity provides more accurate results.
- Acceptance criteria for compression due to overturning in the absence of moment on the footing (for example, axial overturning action at ends of a brace frame supported by two independent footings) is not explicitly addressed.
- Soil bearing acceptance is expressed only in terms of ultimate bearing capacity for an isolated rectangular footing resisting axial load and uniaxial moment. This left out a lot of cases and necessitated the use of engineering judgement which potentially resulted in inconsistencies and misapplication in the use of the standard. The *m*-values provided in ASCE/SEI 41-17 are derived based on axial and overturning actions but were incorrectly applied to soil bearing. The intent is that these *m*-values should not be applied to soil bearing.
- The use of the fixed-base method can results in unusually large footings for a typical superstructure. The results can also be unusually large compared to footings using the other methods or ASCE/SEI 7-16 for similar-sized superstructures.
- Foundation acceptance is only provided for soil bearing with little guidance provided for evaluation of the foundation structural component. In fact, the provisions in the material chapters pertaining to foundations are inconsistent with Chapter 8. The material chapters for concrete and masonry require force-controlled foundation designs while the wood and steel chapter requires foundation design to be deformation controlled, see Table B-1. Further, it is unclear from Chapter 8 if certain items for nonconforming concrete beams can be treated as deformation-controlled actions.

Table B-1 Material Specific Structural Foundation Requirements from FEMA P-2006

Foundation Material	ASCE/SEI 41-13 Section	Action Type
Steel	§ 9.9.4	Deformation-controlled for steel pile; Force-controlled for connection from pile to pile cap
Concrete	§ 10.12.3	Force-controlled; the required capacity is limited by 125% of the capacity of the supported vertical component
Masonry	§ 11.6.2	Force-controlled and modeled as elastic with no inelastic deformation capacity unless demonstrated through ASCE/SEI 41-13 § 7.6
Wood	§ 12.6.2	Flexure and axial loads are considered deformation-controlled with *m*-factors per ASCE/SEI 41-13 Table 12-3. Acceptability of soil below wood footings determined per ASCE/SEI 41-13 Chapter 8.

- Bounding for stiffness and bearing capacities has been required because soil is inherently less homogeneous and has greater variations in material properties than other materials such as steel or concrete. However, the bounding in ASCE/SEI 41-17 is problematic for two main reasons. First, the high and low bounding requires extra analysis effort and thus should yield significantly different results. However, because the bounding and calibration is determined from ASCE/SEI 41-17 Figure 8-2 (Gazetas, 1991), the bounding does not result in significant changes in the superstructure response. Second, the ASCE/SEI 41-17 Figure 8-2 (Gazetas, 1991) equations are based on a rigid structure and elastic soil response where the soil remains in contact with the footings, this overestimates the stiffness. Before applying bounding requirements to the more realistic lower stiffnesses, case studies need to be completed to ensure that the bounding yields significant results as well as not overreach and cause undue conservatism.
- Usability and clarity issues include:
  - Navigation through the foundations chapter in ASCE/SEI 41-17 is complicated as requirements for linear and nonlinear procedures were intermixed within the standard.
  - In some cases, acceptance criteria and direction for items such as bounding and stiffness are provided in narrative form. This led to confusion in applying the provisions and reduced the useability and clarity of the chapter. Tabulated acceptance criteria would be easier to follow.
- Further nuanced technical concerns:

- Definitions of select key terms, such as uplift, are not clear, leading to confusion and misuse of provisions. Uplift in the context of these provisions is the pure axial force causing the entire footing to separate from the soil as opposed to some soil separation on part of the footing as the footing rotates due to rocking action.

- Determination of the effective footing width (B_f) for a mat foundation is missing,

- Where footing overturning action cannot be idealized as a rectangular or I-shaped footing, such as combined footings and mat foundations, the analysis method requires engineering judgement.

- Although based simply on statics, ASCE/SEI 41-17 Equation 8-10 for determining the upper bound moment capacity of a rigid shallow rectangular footing may be confusing to users without its derivation and therefore could lead to implementation issues.

$$M_{CE} = 0.5(L_f\, P_{UD})(1\text{-}q/q_c) \qquad \text{(ASCE/SEI 41-17 Eq. 8-10)}$$

- The overturning action is very dependent upon the transient axial load level, so when evaluating with pseudo seismic forces, determining a realistic seismic axial load is very difficult. This issue is not limited to foundations; it applies to linear procedures of other chapters as well.

- The intended definition of Q_g utilized in Chapter 8 is the expected dead load excluding live loads and snow loads and load factors. This is the Q_D definition in Chapter 7 (the action cause by dead loads).

- The FEMA P-2006 *Example Application Guide for ASCE/SEI 41*-13 examples used the load combinations where the intent was just to use expected dead load, indicating that this misinterpretation is common.

These numerous issues with ASCE/SEI 41-17 were a catalyst for goals for the ATC WG-2 Foundation Working Group study to provide proposed provision and commentary changes for consideration by the ASCE/SEI 41-23 committee and subcommittee. The priorities were to derive a shallow foundation provision structure that was user friendly and to address all the above gaps as well as new items as discovered during the case study work. This ATC work utilized a case study to investigate hypotheses based on these highlighted issues with the provisions.

B.2 Case Study Overview – Archetype 1

Two case study buildings were investigated as part of this project. Archetype 1, an existing concrete two-way slab and column moment frame building, was investigated to evaluate the use of ASCE/SEI 41-17 Chapter 8 for clarity, usability, and technical content as part of ATC 140 –Working Group 2 objectives. The structure in its original configuration and with concrete shear wall retrofit ASCE/SEI 41-17 were investigated during the study.

Archetype 2, a concrete moment frame building, and its analysis are described in a separate appendix.

B.2.1 Building Description

The 1920s existing building is a five-story, 55-foot-tall reinforced concrete structure that measures approximately 104 by 84 feet (5 by 4 bays) in plan. Concrete columns occur on an approximate 20-foot square grid throughout the building, and the structure is supported at its base on shallow isolated footings. Floor and roof slabs are reinforced concrete; the core for the existing elevator and stair are non-structural infill walls.

The gravity system consists of 5 ½-inch reinforced concrete flat slabs at floors 2 to 5 and 6-inch flat slabs at the roof. Drop caps at the columns are typically 7'-0" square and approximately 9-inches additional thickness. At the second floor, the interior columns are typically 34-inches in diameter and exterior columns are typically 24-inches by 43.5-inches. The interior columns decrease in diameter at the upper floors.

The existing lateral force-resisting system is slab-column moment resisting frame (Concrete Moment Frame, C1), and with new shear walls added, Concrete Shear Walls with Rigid Diaphragms (C2). The retrofit consists of adding shear walls at strategic locations; one in the center of the building for north-south loading, two in the orthogonal direction at the ends of the building to provide shear and plan torsion stability (See Figure B-1 and Figure B-2 below). The studies apply unidirectional loading in the north-south direction which generates essentially no seismic axial load at the shear wall and in the east-west direction to study the effects from significant seismic axial loads (both up and downward).

The single shear wall added in the center of the building for north-south loading has limited seismic axial load and allows for examination of a singular rectangular footing, and then further examination as it is expanded to reach the adjacent columns as shown in Figure B-2. This reduced the number of parameters being studied, and in particular it removed the sensitive seismic axial load component which can have a profound effect on the foundation's behavior. In contrast, the exterior shear walls see significant seismic axial force, both uplift and downforce, making them an ideal case to study the seismic axial effects without noise from other parameters.

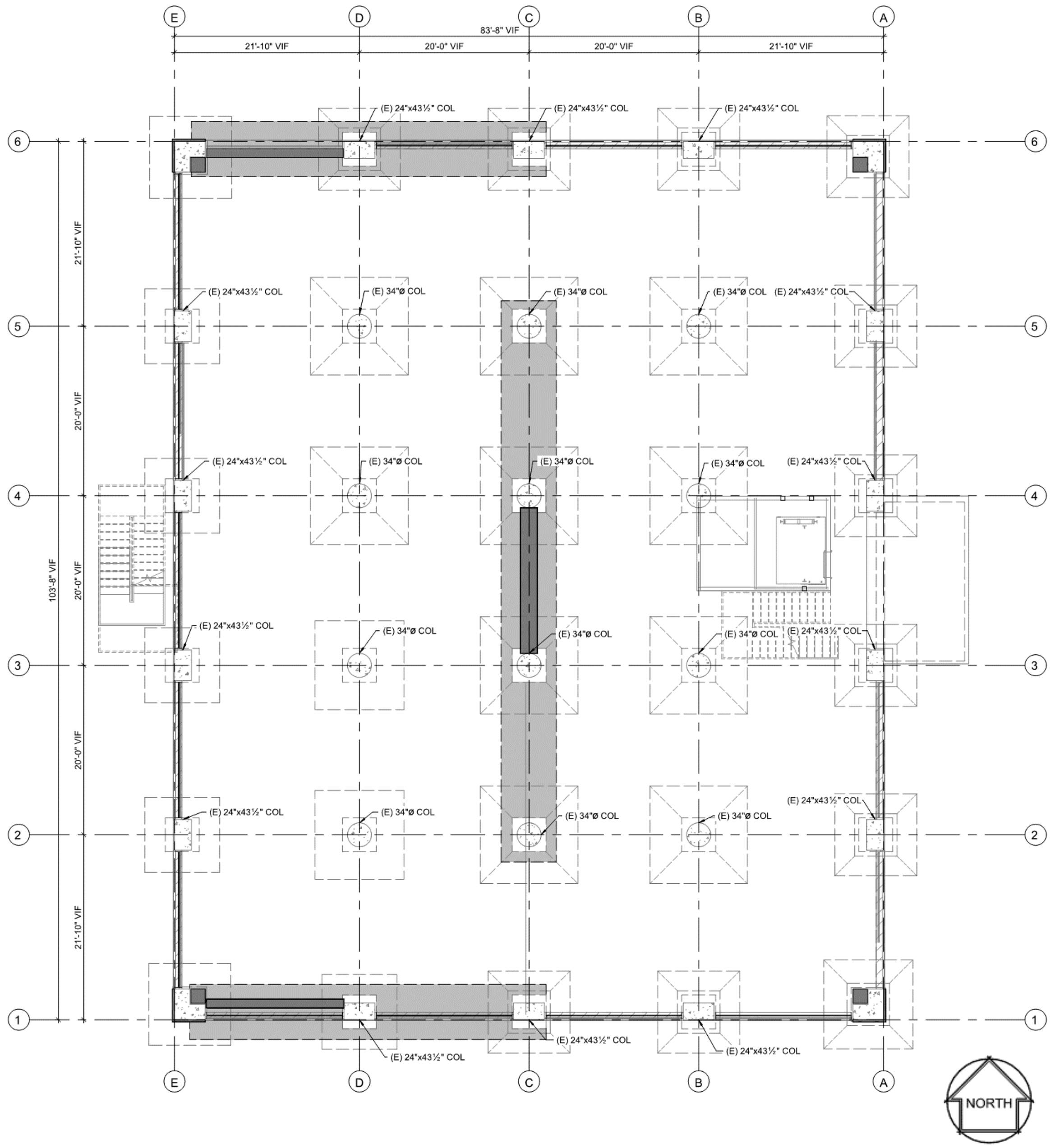

Figure B-2 Foundation plan.

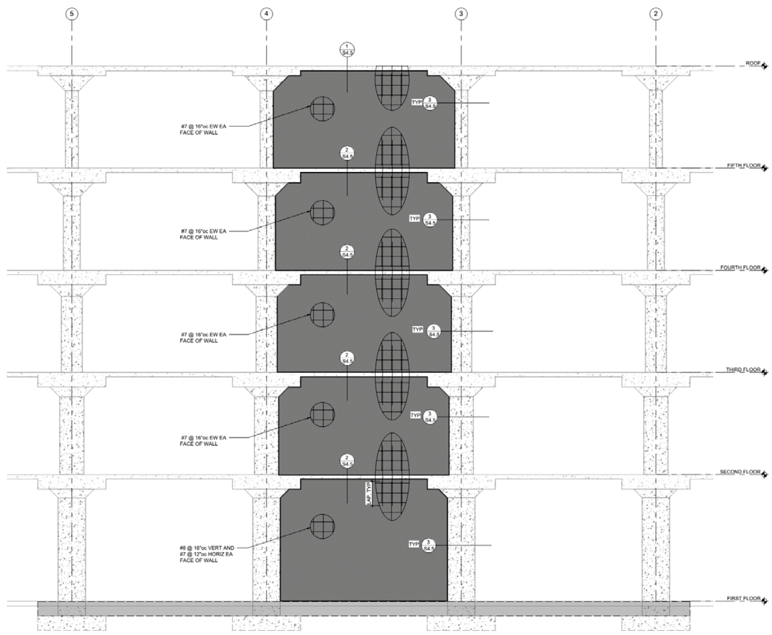

Figure B-3 Retrofit wall and existing structure elevation (3 of 5 bays shown). The grey grade beam and shear wall were added in the retrofit.

The building is located in a high seismic region and would be classified as Risk Category 2 per the 2019 IBC.

B.2.2 Soil Conditions

A geotechnical investigation was performed on the site. The soil consists of medium stiff clayey fill underlain by stiff to very stiff clay and claystone bedrock. New and existing footings are founded on the claystone bedrock with a N_{60} (penetration blow count corrected to an equivalent hammer energy efficiency of 60%) equal to 25 per the geotechnical engineer. The initial shear modulus is calculated per ASCE/SEI 41-17 § 8.4.2.2, and the effective shear modulus is determined based on the ratio in ASCE/SEI 41-17 Table 8-2.

B.2.3 General Modeling Assumptions

B.2.3.1 ANALYSIS MODEL

The finite element analysis program, ETABS by CSI, is widely used by the engineering community and is the analysis platform in this case study. The analysis model is three-dimensional for all cases and consists of analysis objects including joints, frames, and areas. ETABS automatically converts the object-based model into an element-based model in the analysis. The element-based model consists of finite elements and joints with lumped joint mass.

Where nonlinear characteristics are included in the analysis, lumped plasticity, user-defined hinge properties are input in ETABS and assigned to frame elements. Fiber modeling is not utilized in this analysis as the nonlinear aspects of the elements are adequately captured by the nonlinear hinges applied to frame elements, though it is recognized that modelling to the frame elements at the center of the wall, as opposed to the neutral axis, does somewhat underestimate superstructure demands. Walls and slabs that are typically defined as shell elements in linear models are defined as frame elements in the nonlinear models for assignment of frame hinges.

When analyzed with a fixed-base assumptions, the base of each column (including each end of shear walls) are restrained against translation and rotation which is consistent with the foundation details. This assumption was compared to a model with base of the columns pinned (base of columns restrained against translation but not rotation). The fundamental period of the two models was within 5% of each other, indicating that, for this building, the column base fixity does not have a large effect on overall building response. However, the pinned-base analysis underestimates the strength and stiffness of the column frames, so the fixed base analysis is utilized for this case study.

In the analysis models with foundation components explicitly modeled (flexible base), overturning action on the soil is modeled as either a single rotational spring or coupled axial springs as discussed later.

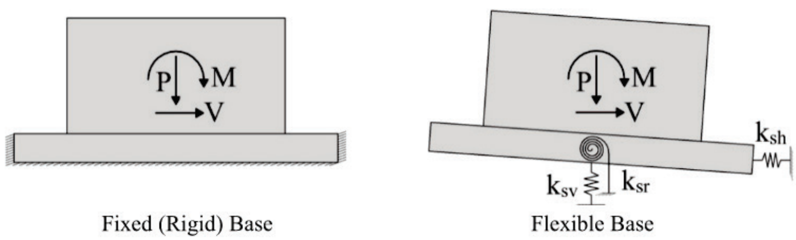

Figure B-4 Structure boundary conditions from FEMA P-2008.

Multiple analysis models of the same building were used to analyze the different ASCE/SEI 41-17 foundation modeling provisions. Analysis models for all hypotheses are outlined in Table B-2. Each hypothesis (described in Section B.3) will reference the models utilized. These models were developed to cover multiple foundation modeling options in ASCE 41: linear and nonlinear static procedures, and fixed base and flexible base, Method 1 springs. Load cases studied vary by hypothesis.

Every analysis model is a 3-dimensional ETABS model with the following attributes:

- Unidirectional loading in the North-South direction (except for the design of the East-West retrofit foundation as described in Section B.6)
- Expected material properties used in each model are f'_{ce} = 4.8 ksi and f_{ye} = 52.0 ksi.
- The concrete effective stiffnesses are cracked properties per ASCE/SEI 41-17 Table 10.5

Rigid diaphragms, though slab and column frame actions are included. Accidental torsion is not investigated within these analyses except where bi-directional cases are noted. The building plan is symmetric in the north-south direction with the retrofit wall at the plan center of the building and thus torsional response is ignored for these investigations. The retrofit in the east-west direction provides walls on the west side on the north and south ends of the structure as shown in Figure B-2 and for the bi-directional analysis at these footings, the models include a 5% accidental torsion (ASCE 41-17 §7.2.3.2.1 & ASCE/SEI 7-10 §12.8.4.2).

Table B-2 Analysis Models Utilized in the Case Study

	Model A	**Model B**	**Model B.1**	**Model C**	**Model D**
Analysis Procedure	LSP (§ 7.4.1.3)	LSP (§ 7.4.1.3)	LSP (§ 7.4.1.3)	NSP (fundamental mode load application)	NSP (fundamental mode load application)
Soil Springs	N/A (Method 1 Fixed Base)	Method 1 linear springs with upper and lower bound stiffness values	Method 3, type varies, see Section B.3.6: Hypothesis 6	N/A (Fixed Base)	Method 1 nonlinear moment rotation springs compression-only vertical springs with expected values and no uplift capacity
Foundation Retrofit	Retrofit and No Retrofit cases modeled	Modeled	Modeled	Modeled	Modeled
Columns	Elastic Frame Elements	Elastic Frame Elements	Elastic Frame Elements	Elastic Frame Elements with Nonlinear Hinges Top and Bottom	Elastic Frame Elements with Nonlinear Hinges Top and Bottom
Structural Slab	Elastic Frame Elements	Elastic Frame Elements	Elastic Frame Elements	Elastic Frame Elements with Nonlinear Hinges Each End	Elastic Frame Elements with Nonlinear Hinges Each End
Shear Wall	Elastic Frame Elements	Elastic Frame Elements	Elastic Frame Elements	Elastic Frame Elements with Nonlinear Flexural Hinges Top and Bottom and Shear Hinge at Center	Elastic Frame Elements with Nonlinear Flexural Hinges Top and Bottom and Shear Hinge at Center

A significant amount of time was invested ensuring accurate modeling and that the code interpretations used in this study were in keeping with the industry standard of practice. A peer review was completed of the modeling to ensure that the case study began with a highly reliable model. In creating the models, additional background foundation concepts in ASCE/SEI 41-17 were investigated including the expected bearing capacities, bearing capacity bounding as well as the expected restoring dead load, discussed in Sections B.4 and B.5 of this report. Understanding the interpretations of these concepts and their effects on results is necessary so that these factors do not convolute results aimed at examining other topics.

Note that when completing ASCE/SEI 7-10 analyses for comparison, the results can vary significantly due to the use of different redundancy factor ρ and torsional effects. For this study, the redundancy factor, ρ, is taken as 1.0 since the existing column and slab frames provide redundancy; however, ρ = 1.3 could also be considered technically accurate. For this case, the ASCE/SEI 7-10 results would have been more similar to the ASCE/SEI 41-17 linear results had ρ = 1.3 been used. Simplifications and assumptions such as these redundancy factors and torsional effects can change results significantly. Therefore, it is important to use engineering judgement when comparing and drawing conclusions about these analysis results.

To represent a realistic force level in a high seismicity area, these studies utilized a site with accelerations: for ASCE/SEI 7-10: S_{DS} = 1.0 g and S_{D1} = 0.6, and for ASCE/SEI 41-17: S_{xs} =1.0g and S_{x1} =0.6g (BSE-1E), and S_{xs} =1.5g and S_{x1} =1.0g (BSE-2E). The vertical distribution of forces is derived from the base shear calculations using ASCE/SEI 7 and the pseudo seismic force demands using ASCE/SEI 41.

B.2.3.2 ANALYSIS PROCEDURES

Linear and nonlinear analysis procedures from ASCE/SEI 41-17 Chapter 7 were utilized in this study. Many of the hypotheses compare linear results from the linear static procedure (LSP) to nonlinear results from the nonlinear static procedure (NSP). In these comparisons, the nonlinear results are utilized as the benchmark for calibration with linear procedures. This study assumes that the results of nonlinear analyses are reasonable for comparison with the results of the hypotheses related to linear analyses. Additionally, though fundamentally difficult to compare, a parallel assessment using ASCE/SEI 7-10 provisions was also performed. Note: Some aspects of building may not conform to the requirements of current code but are used for illustrative purposes to highlight use of the foundation provisions in ASCE/SEI 41-17 and compare outcomes with the provisions for new buildings using ASCE/SEI 7. As with this type of parametric study, engineering judgment is required when generating, reviewing, and drawing recommendations from the results.

B.2.3.3 GENERAL OVERVIEW OF ASCE/SEI 41-17 CHAPTER 8 SOIL MODELING METHODOLOGIES

There are three “methods” for foundation modeling in ASCE/SEI 41-17. These different methods were utilized in the case study. There are two methodologies (fixed-base and flexible-base) included within Method 1. Method 1 fixed base models do not have soil springs and are restrained against translation and global rotation at the soil-structure interface. Acceptance criteria for the fixed base models are per ASCE/SEI 41-17 §8.4.2.3.2.1 which includes provisions for soil bearing and overturning stability of individual foundation elements. Method 1 flexible base models use uncoupled moment, shear, and axial springs to model rigid foundations such that the moment and shear behaviors are independent of the axial load. Method 1 soil springs can be utilized for both linear and non-linear analysis methods but is only applicable to footings assumed rigid compared to the soil. Method 2 is also for shallow footings considered rigid compared to the soil but can only be utilized with nonlinear analysis methods. Method 2 provides an alternative approach for rigid footings that uses a bed of nonlinear springs that accounts for coupling between vertical loads and moment. Method 2 is the preferred approach when there is significant variation in axial load. The moment-rotation and vertical load-deformation characteristics are modeled as a beam on a nonlinear Winkler foundation with stiffer vertical springs at the end regions of the foundation to allow for tuning of the springs to approximately match the elastic vertical and rotational stiffness provided in Method 1. This Method 2, if applied in the NDP, may also be used to account for settlement and permanent deformations, though determination of those requires complicated combinations of plastic and gap elements in parallel and in series (Harden et al., 2005). Method 3 is the only method allowed for shallow foundations where the structural component (footing) is flexible (not rigid) relative to soil, and it is only applicable to nonlinear analysis procedures. Method 3 uses a similar methodology to Method 2 with Winkler springs beneath the foundations, except that a uniform distribution of soil stiffness and strength is applied. The differences between the different methods can be significant since Method 2 is meant to be calibrated with Method 1, which is based on low-strain, elastic soil response and assumes the soil remains in contact with the footing, whereas Method 3 can include geometric nonlinearity where the soil separates from the footing.

Deep foundation provisions are not investigated as part of this case study.

B.2.3.4 SUPERSTRUCTURE PROPERTIES

Concrete modeling and analysis procedures from ASCE/SEI 41-17 Chapter 10 are followed for the ETABS model superstructure.

- Material Properties (ASCE/SEI 41-17 § 10.2) – Materials properties utilized in the analysis model are expected strengths based on usual data collection. Existing drawings of the building were available for review, although they did not specify design strengths of materials. Materials testing was performed to determine expected strengths.

- Modeling and Design (ASCE/SEI 41-17 § 10.3.1) – Elastic component effective stiffnesses are determined per ASCE/SEI 41-17 Table 10-5. When nonlinear models are utilized, hinge properties are defined per ASCE/SEI 41-17 Figure 10-1. Nonlinear hinges are assigned at appropriate locations on frame elements within the ETABS model where nonlinear behavior is expected.

    - For concrete column frame elements, a nonlinear P-M hinge is defined at the top of the column at the base of the capital, and at the base of the column at the connection to the floor slab. For each hinge, the slope from A to B in the load-deformation relation is the same as defined for the linear models. The slope from B to C is taken as 10% of the initial slope. The deformation or rotation that defines point C is defined by other tables in ASCE/SEI 41-17 Chapter 10. An example column P-M hinge is included in Figure B-5.

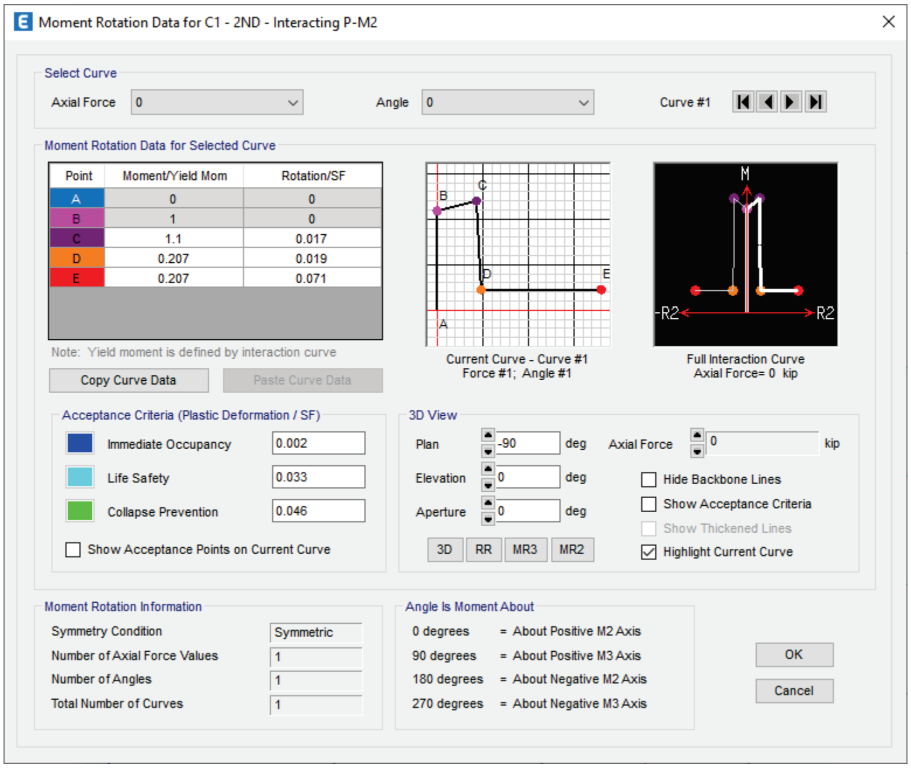

Figure B-5 Sample column P-M hinge property.

- Concrete shear walls are modeled as frame elements in all analysis models with the appropriate effective elastic stiffness values (ASCE/SEI 41-17 Table 10-5). For nonlinear analysis, the concrete shear wall frame elements have flexural hinges at top and bottom and a shear hinge at the center per ASCE/SEI 41-17 Tables 10-19 and 10-20. Note that the shear wall flexural hinges are moment only, not P-M hinges. See example shear wall hinge properties in Figure B-6 and Figure B-7.

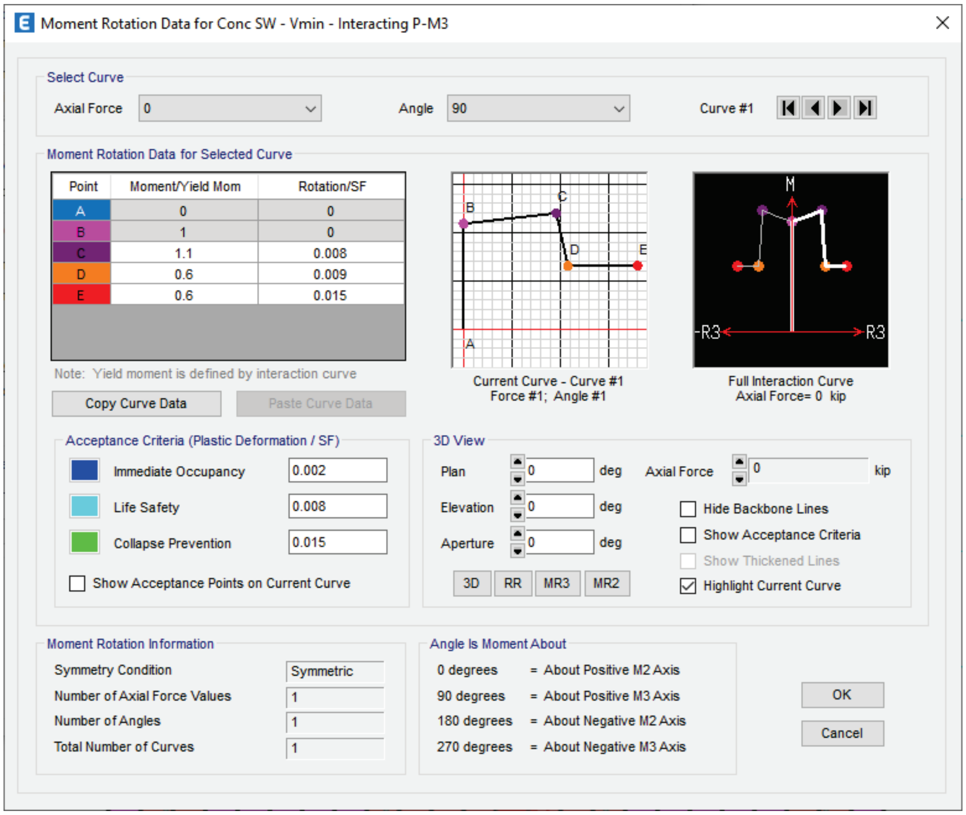

Figure B-6 Sample concrete shear wall flexural hinge property.

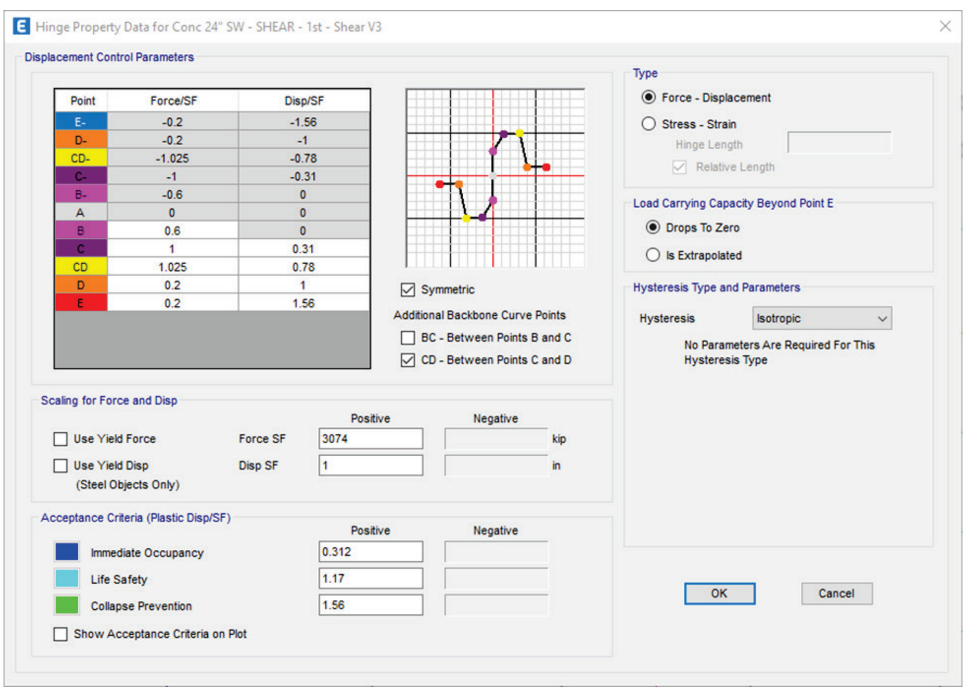

Figure B-7 Sample concrete shear wall shear hinge property.

- The concrete floor slab at each level is modeled with frame elements to capture frame action with the columns, and in-plane diaphragm action is modelled by slaving coordinates at each floor and roof together to form a rigid diaphragm. The ground floor slab-on-grade is omitted from all analysis models as it does not contribute to the behavior of the structure. In the nonlinear models, the effective beam width model per ASCE/SEI 41-17 § 10.4.4.1 is utilized to model the slab and the drop panels, both as frame elements that contribute to the moment resisting action of the frame. Hinges per ASCE/SEI 41-17 Table 10-15 are assigned to the slab frame elements at the edge of the drop panels in the nonlinear models. Sample hinge properties are included in Figure B-8 and Figure B-9. Rigid diaphragm constraints are applied for both linear and nonlinear analysis models.

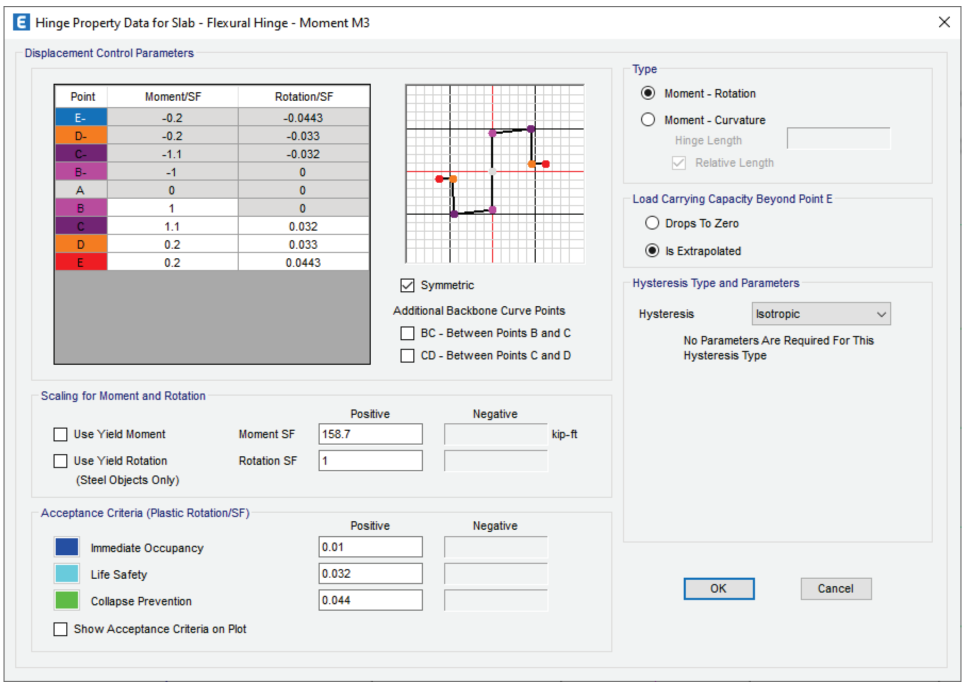

Figure B-8 Sample flexural slab hinge property.

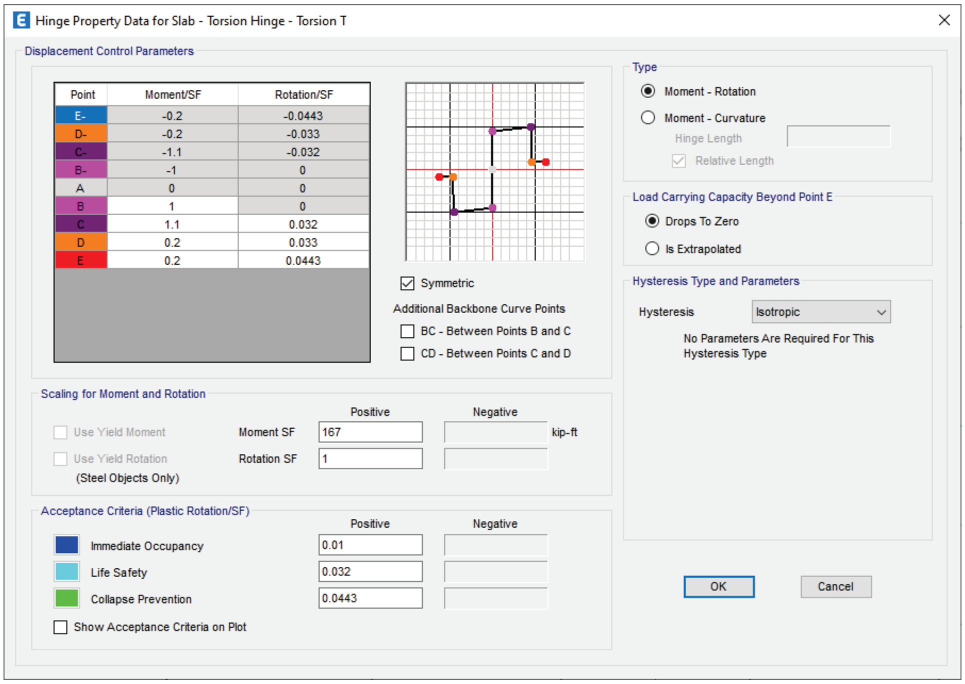

Figure B-9 Sample torsional slab hinge property.

Exterior and interior staircases are not included in the analysis model. The elevator shaft is modeled as an opening in the slab at each floor level. The stair and shaft walls are not included in the model as they are nonstructural hollow clay tile walls that are removed as part of the retrofit.

B.2.4 Building Retrofit

The investigations completed used ASCE/SEI 41-17 to examine the existing footings, but also to examine possible foundation retrofits in conjunction with the new proposed shear walls.

B.2.4.1 PROPOSED FOUNDATION RETROFIT GEOMETRY (NORTH – SOUTH DIRECTION)

The existing foundations were evaluated using ASCE/SEI 41-17 linear static procedures with a fixed-base assumption, as well as ASCE/SEI 7-10, equivalent lateral force methodology with a fixed-base assumption for their capacity to support overturning forces due to lateral loading on the new concrete shear wall at the center of the building. These studies used an acceleration level S_a of 1g to represent a realistic force level in a high seismicity area as well as site specific accelerations at a high seismicity site. These accelerations were also scaled to identify at what accelerations the foundation acceptance criteria and allowable bearing pressure are met. While further discussion of these studies is included in subsequent chapters, they all indicated that the existing foundation was not adequate to support overturning forces due to lateral loading on the new concrete shear wall. Therefore, new concrete foundations were proposed as shown in Figure B-2.

The retrofit footing at the central wall connects the existing pad footings at adjacent columns together to create one continuous footing. This engages additional dead load that reduces the uplift at the foundation due to the lateral loading on the new shear wall. The retrofit requires continuous reinforcement through the existing footings for flexure. The proposed retrofit plan layout is shown in Figure B-10 with geometric properties in Table B-3. To simplify the analysis, the retrofit footing is approximated as a rectangular footing with an average footing width to account for the variations in footing width along its length.

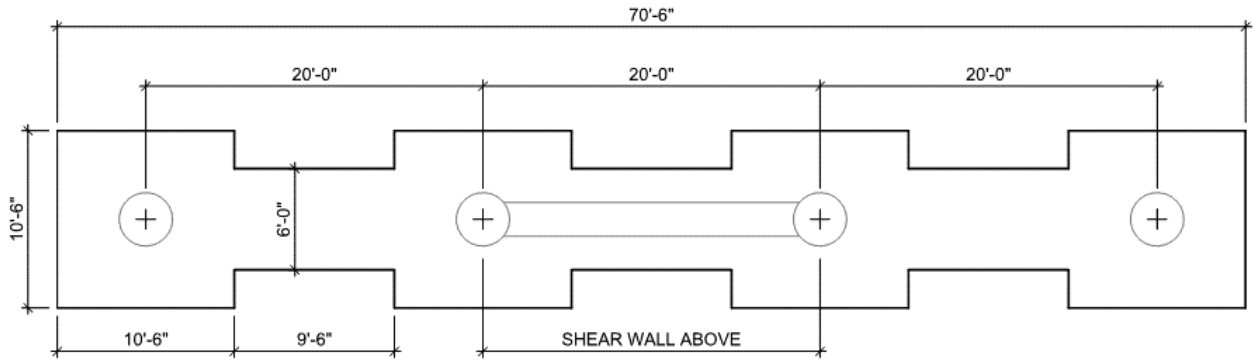

Figure B-10 Retrofit footing plan layout with dimensions.

Table B-3 Retrofit Footing Geometric Properties

Retrofit Footing Geometric Properties	
Footing Area (A_f)	612 ft^2
Average Footing Width (B)	8.7 ft

The footing retrofit was designed utilizing ASCE/SEI 7-10 provisions assuming the new footing is rigid compared to the soil and an elastic, triangular soil bearing pressure distribution. The retrofit footing was designed to meet bearing pressure requirements and for structural footing strength. It was determined that a 6-foot-wide by 4-foot-deep footing 6-foot-wide by 4-foot-deep footing with (30) #11 bars top and bottom is adequate for the design loads. This footing was then used for comparisons with ASCE/SEI 41-17 foundation designs. ASCE/SEI 41-17. It is noted that a Method 3 approach would be more appropriate for this footing configuration. However, as practice may treat this as rigid, we are exploring Methods 1 and 2 for comparison purposes.

B.2.4.2 PROPOSED FOUNDATION RETROFIT GEOMETRY (EAST – WEST DIRECTION)

Based on calculations performed as part of Section B.6, the existing foundation is not adequate to support overturning forces due to lateral loading on the new concrete shear walls.

In the east-west direction, the proposed retrofit footing extends one bay beyond the shear wall towards the middle of the building. It connects three existing pad footings together to create one continuous footing. This engages additional dead load that reduces the uplift at the foundation. The retrofit requires continuous reinforcement through the existing footings for flexure. The proposed retrofit plan layout is shown in Figure B-11. The retrofit footing was idealized as rectangular rather than three 10'-6" square footings connected by a narrower continuous footing.

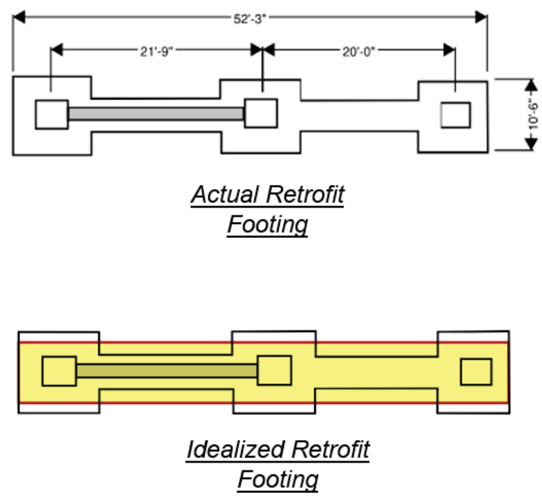

Figure B-11 Rectangular retrofit footing plan layout in east-west direction.

However, analysis indicated that this retrofit was not acceptable. Therefore, an alternative retrofit footing was also investigated in which an L-shaped footing extends one bay perpendicular to the retrofit shear wall as shown in Figure B-12. This engages additional dead load. Further discussion of these footing designs can be found in Section B.6.

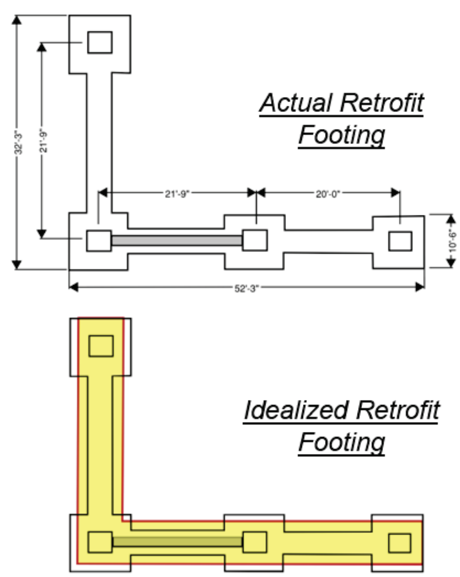

Figure B-12 L-shaped retrofit footing plan layout in east-west direction.

B.3 Investigation Hypotheses

Hypotheses were developed that pose answers to questions arising from the highlighted issues with the ASCE/SEI 41-17 foundation guidelines, and served to guide the case study investigations. Each hypothesis is set to examine a technical point within the chapter that the working group sees as requiring clarification. Each hypothesis attempts to isolate one aspect to be quantitatively investigated. In some hypotheses, it is difficult to study one provision without understanding the implications of other assumptions. These are investigated within each hypothesis as required. This section summarizes the hypotheses and the resulting conclusions and recommendations at a high level. Subsequent sections (B.4 through B.12) provide more in-depth results and discussions of topics of interest that arose throughout the hypothesis work.

While the general results of each hypothesis are discussed within this section, the detailed results of the technical studies are organized in later sections within the framework of ASCE/SEI 41-17 Chapter 8.

B.3.1 Hypothesis 1: Acceptance Criteria for Fixed Base Condition

Hypothesis 1 states that the use of the Method 1 linear, fixed base foundation approach with overturning action *m*-factors in accordance with ASCE/SEI 41-17 § 8.4.2.3.2.1 provides reasonable assurance that the overturning stability and forces are accurate and slightly conservative and the fixed base model may be used to evaluate superstructure components.

B.3.1.1 HYPOTHESIS 1 PROCESS

LSP (linear static procedure) was performed and compared to NSP (nonlinear static procedure) analysis. The acceptance ratios of the linear and nonlinear analyses were compared to test the hypothesis. For both the LSP and the NSP, the stiffnesses used were the best estimate expected values; the upper and lower bound stiffnesses were not modeled. The following analysis were done using Model A as described in Table B-2.

A pseudo lateral force was applied at every floor for the following base shear force levels and scenarios (linear load cases). For each case, the foundation soil acceptance ratios and foundation structure acceptance ratios were recorded (for retrofit footing cases).

1. Using ASCE/SEI 41-17 with an acceleration level Sa of 1g to represent a realistic force level in a high seismicity area without a foundation retrofit.
2. Using ASCE/SEI 41-17, scale the base shear until the overturning compression action demand balances with the capacity when an *m*-factor for CP of 4.0 is applied without a foundation retrofit using compression acceptance criteria per ASCE/SEI 41-17 § 8.4.2.3.2 (S_a = 0.62g).
3. Using ASCE/SEI 7-10 with an acceleration level Sa of 1g, with R = 6 (special concrete shear wall, though one could have used R = 5), to represent a site-specific force level in a high seismicity area without a foundation retrofit.

4. Using ASCE/SEI 7-10, scale the base shear until the overturning compression action demand balances with the capacity without a foundation retrofit (S_a = 0.18g)
5. Using ASCE/SEI 41-17 with a site-specific acceleration level Sa for the same location as Linear Case 3 without a foundation retrofit.

All of the above cases indicated that a shear wall retrofit without a foundation retrofit is unacceptable for this building. The final three cases include a foundation retrofit.

6. Design of a retrofit foundation for site-specific loading of Linear Case 3 using ASCE/SEI 7-10.
7. Evaluation of foundation designed in Linear Case 6 using ASCE/SEI 41-17 and site-specific loading of Linear Case 5.
8. Evaluation of retrofit footing from Linear Case 6 with LSP Method 1 foundation springs.

Following completion of the LSP analysis cases, NSP was utilized for comparison and benchmark with Model D. Model C was also investigated to determine the superstructure behavior without displacement at the foundation/soil interface.

These results from Model C and Model D were recorded for comparison with Model A results:

1. Column demands (all actions) as well as the associated capacities
2. Slab demands (flexure action) as well as the associated capacities
3. Total base shear
4. Shear force action in shear wall
5. Soil Acceptance Criteria

Primarily, acceptance ratios are used to compare the LSP and NSP results:

- Q_{UD} / $k m Q_{CE}$ (ASCE/SEI 41-17 LSP Results)
- Target Displacement Rotation / Allowable Rotation (ASCE/SEI 41-17 NSP Results)
- Demand / Capacity (ASCE/SEI 7-10 Results)

B.3.1.2 HYPOTHESIS 1 RESULTS

Table B-4 summarizes the findings from the first four linear analysis cases investigated. Based on the analysis results, the existing foundations are not adequate with a typical high seismicity site design acceleration of 1g for both ASCE/SEI 41-17 and ASCE/SEI 7-10 analyses.

Table B-4 Summary of Acceptance Ratios and DCRs for Linear Cases 1, 2, 3, and 4

	ASCE/SEI 41-17			ASCE/SEI 7-10	
	Case 1	Case 2		Case 3	Case 4
	Realistic Force Level Sa = 1g	Acceptable w/o footing retrofit Sa = 0.30g	CP m-factor & ASCE/SEI 41-17 Section	Realistic Force Level Sa = 1g	Acceptable w/o footing retrofit Sa = 0.18g
Uplift acceptance ratio or DCR (Conventional Tributary Area Restoring Dead Load)	2.1	0.6	8 (8.4.2.3.2.1)	2.4	0.4
Uplift acceptance ratio or DCR (Capacity Based Design Restoring Dead Load)	1.6	0.5	8 (8.4.2.3.2.1)	0.9	0.1
Bearing Pressure acceptance ratio or DCR	3.3	1.0	4 (8.4.2.3.2.1)	2.1	1.0
Overall Overturning Stability DCR	1.7	0.5	10 (7.2.8.1)	3.2	0.5
Outcome	NG	OK		NG	OK

It was only when the seismic acceleration with ASCE/SEI 41-17 methodology was reduced to 0.30g that the existing footing was sufficient for the seismic loading. Similarly, an acceleration of 0.18g was required with ASCE/SEI 7-10. These findings indicate that in high seismic regions, a shear wall retrofit without a foundation retrofit is unlikely to be acceptable. While this is obvious for most readers, there has been a repeated theme discussed by some that "foundation retrofit is uneconomical to perform and that buildings don't tip over". This study numerically proves that foundation retrofit is required in order to ensure the shear wall performs as intended and does protect the existing components; therefore, the building is able to meet the targeted performance level.

If a foundation retrofit was not provided, the building would not meet the target performance objective. In that case, the retrofit would fall in the category of a Partial Retrofit in accordance with ASCE/SEI 41-17 Section 2.2.5. These limitations should be relayed to the stakeholders if this partial retrofit approach is taken.

These results were confirmed with an additional study at a specific high seismicity site (Linear Case 5). The ASCE/SEI 41-17 analysis was performed using the BSE-2E Seismic Hazard Level with

Collapse Prevention acceptance criteria, which is consistent with the ASCE/SEI 41-17 Basic Performance Objective for Existing Buildings (BPOE). The site-specific seismic parameters were S_{XS} = 1.5g (ASCE/SEI 41-17) and S_{DS} = 1.0 g (ASCE/SEI 7-1). The results for the ASCE/SEI 41-17 are described in Table B-5 and Table B-6.

Table B-5 Summary of ASCE/SEI 41-17 Acceptance Ratios for Linear Case 5 and ASCE/SEI 7-10 DCRs for Linear Case 5 and 3

	ASCE/SEI 41-17		ASCE/SEI 7-10
	Acceptance Ratio		DCR
Uplift (Conventional Tributary Area Restoring Dead Load)	3.0		2.4
Uplift (Capacity Based Design Restoring Dead Load)	2.1		0.9
Bearing Pressure	4.4		2.1
Overall Overturning Stability	2.2		3.2
Outcome	NG		NG

As expected, for this specific high-seismic site, the shear wall retrofit without a foundation retrofit is unacceptable and would not meet the target performance level. Therefore, further cases were analyzed to design and evaluate a retrofit footing. Alternatively, in practice, one could proceed on a partial retrofit basis.

The footing retrofit was designed utilizing ASCE/SEI 7-10 assuming the new footing is rigid compared to the soil (Linear Case 6). The resulting footing is described in Section B.2.4.1. For Linear Case 7, this footing was then used for evaluation with ASCE/SEI 41-17 using the same site-specific seismic hazard and performance level as Linear Case 5. This footing does not quite meet the acceptance criteria of ASCE/SEI 41-17, see Table B-6 below.

Table B-6 Linear Case 7 Results: Summary of Footing Retrofit Acceptance Ratios and DCRs

	ASCE/SEI 41-17				**ASCE/SEI 7-10**	
	Section	CP m-factor	Acceptance Ratio		Section	DCR
LSP, Bearing Pressure	8.4.2.3.2.1	4	1.32		12.13.4	0.95
LSP, Uplift	8.4.2.3.2.1	8	0.71		12.13.4	0.56
LSP, Overall Overturning	7.2.8.1	10	0.51		12.8.5	0.75
Outcome			NG			OK

However, note that the bearing pressure acceptance ratio for ASCE/SEI 41-17 is 1.32. As the ASCE/SEI 7-10 results are very dependent on the system Response Modification Coefficient, R, and the redundancy factor ρ, these ASCE/SEI 7-10 results could align with the ASCE/SEI 41-17 acceptance ratio of 1.32 had for example ρ = 1.3 been used. Further, the footing strength is not adequate based on an ASCE/SEI 41-17 force-controlled analysis, with the acceptance ratio of 4.9, see Table B-41. It does meet ASCE/SEI 41-17 if the footing flexural action is treated as deformation controlled, though that is not permitted in accordance with ASCE/SEI 41-17 Section 10.12.3.

Following these analyses, the retrofit footing based on the ASCE/SEI 7-10 design was evaluated for site specific loading using the LSP with Method 1 foundation springs per ASCE/SEI 41-17 Figure 8-2. Spring derivation and methodology discussion can be found in Section B.8.3.1. In short, the retrofit footing is treated as a rigid body for the Method 1 spring stiffness derivations. Method 1 uses uncoupled moment and axial springs to model rigid foundations such that moment and shear behaviors are independent of axial load. Shear (sliding) springs may also be used. In this case, and for all methods in this investigation, lateral moment is restrained within the analysis model. The results of this flexible base analysis (Linear Case 8) are compared to the results of the fixed base study (Linear Case 7) in Table B-7.

Table B-7 Comparison of Fixed Base (Linear Case 7) and Flexible Base Method 1 (Linear-Case 8) Acceptance Ratios

	Fixed Base	Flexible Base
LSP, Bearing Pressure	1.32	1.16
LSP, Uplift	0.71	0.68
LSP, Overall Overturning Stability	0.51	0.50
Outcome	NG	NG

These analyses show similar results, with the flexible base ratio slightly less than that of the fixed base model. Following completion of the LSP analysis, NSP was utilized as comparison and benchmark for these studies. Multiple soil spring methodologies were also examined as a part of this NSP analyses, see Section B.10.3 for these derivations. The nonlinear superstructure hinge behavior was modeled in accordance with ASCE/SEI 41-17 Chapter 10. Following are the NSP cases evaluated and brief findings associated with each. Complete analysis findings can be found in Section B.10. Note that the acceptance ratios discussed in these findings are the rotations at the base of the shear wall at the target displacement compared to the allowable footing rotation from ASCE/SEI 41-17 Table 8-4.

1. Fixed base NSP as a baseline for comparison
    - The calculated target displacement is equal to 5.3 inches. The fundamental period of the structure is 0.45 seconds, which matches the LSP analysis.

2. Method 1 soil springs NSP – force-controlled foundation design

    - The flexural demand at the target displacement was used to assess the footing as force-controlled with lower-bound strength as specified in ASCE/SEI 41-17 § 10.12.3. The footing flexural action has an acceptance ratio of 1.37, so it is not acceptable and would require additional strength with this analysis approach.

    - ASCE/SEI 41-17 provides guidance in the commentary (§ C8.4.2.1) for determining when a foundation is rigid compared to soil by comparing the foundation stiffness to the soil stiffness in Equation C8-1. Based on this definition, the footing is not rigid compared to the soil; therefore, Method 1 is not applicable. Therefore, alternative Method soil springs are explored.

3. Method 2 (non-tuned) soil springs NSP – force-controlled foundation design

    - ASCE/SEI 41-17 §8.4.2.4.1 and C8.4.2.4.1 state that Method 2 springs should be tuned to match the stiffness of Method 1 and provide a reference to Gajan et al (2010). In order to assess the affect of tuning or not tuning Method 2 springs, this model assumes no tuning and is compared to the subsequent model which does tune the springs.

    - The footing rotation at the target displacement meets the acceptance criteria.

    - The flexural action in the footing is also assessed at the target displacement to verify the footing strength. The footing is undersized for the force-controlled flexural demands, with an acceptance ratio of 2.29.

    - During this investigation, it was found that the acceptance criteria in ASCE/SEI 41-17 Table 8-4 is highly dependent on the A_c/A_f factor and the b/L_c of the footing. The allowable rotation is highly sensitive to the footing area, and in this case the footing width, since the length is constrained. When the footing width doubles, the allowable rotation increases by a factor of 5.7, which demonstrates that the calculated rotation is highly sensitive. See further discussion in Section B.10.5.4.

4. Method 2 (tuned) soil springs NSP – force-controlled foundation design

    - This method utilized tuned springs per Gajan et. Al. instead of ASCE/SEI 41-17 Figure 8-2 (Gazetas, 1991), see Section B.1.3.3. These revised springs negligibly change the response of the structure from Method 1 to tuned Method 2 as indicated in Section B.1.5.10, However, as noted in Section 5.8.3.4, using the K_{50} stiffness had a significant difference, and led to a more flexible system.

    - The acceptance ratio for the footing rotation is similar to the initial Method 2 results. The flexural foundation acceptance ratio is 1.69.

5. Method 3 soil springs NSP – force-controlled foundation design

- The footing that was designed using ASCE/SEI 7-10 is then evaluated for force-controlled flexure in accordance with ASCE/SEI 41-17. and the structural footing design is not acceptable with an acceptance ratio of 2.39.

6. Method 3 soil springs NSP – deformation-controlled foundation design

   - Although foundations are typically required to be evaluated as force-controlled in accordance with ASCE/SEI 41-17 § 10.12.3, the nonlinear modeling and acceptance criteria provisions for concrete beams within ASCE/SEI 41-17 Chapter 10 are applied to the foundation structure for this case, which is commonly done in practice. The ASCE/SEI 7-10 designed retrofit footing has flexural hinges assigned to each end of the footing beams between the existing footings. The hinges are assessed in accordance with the provisions of ASCE/SEI 41-17 Chapter 10 to the Collapse Prevention acceptance criteria.

   - The fundamental period of this model is 0.63 seconds, which is more than the LSP Method 1 (0.54 seconds) and LSP fixed based procedures (0.45 seconds). The target displacement is 10.7 inches.

   - The acceptance criteria per ASCE/SEI 41-17 Table 8-4 is dependent on the L_c (defined as the length of the contact area and equal to A_c/b). In this case, that is taken as the full length of the footing. Judgement may be required in other foundation configurations.

   - The acceptance ratio for the footing at the target displacement is 0.43. Therefore, the retrofit footing design is acceptable based on a deformation-controlled foundation design.

7. Method 3 soil springs NSP – deformation-controlled, acceptance criteria at inflection points

   - As discussed in FEMA P-2006 § 5.7.6.1, a flexible footing could be assessed by evaluating individual sections separated at inflection points. For this case, the acceptance criteria is recalculated for a similar condition to NSP Case 6 but with the soil acceptance criteria evaluated with L_c defined for three individual segments based on flexural inflection point locations.

   - All of the segments meet their acceptance criteria (acceptance ratios are 0.49, 0.34, and 0.20). The highest loaded segment also has the lowest rotation as the beam hinge adjacent to it is yielding which reduces the rotation demand.

A summary of all of the foundation acceptance criteria cases (excluding those without the retrofit footing) are included in Table B-41 in Section B.10.3. The only ASCE/SEI 41-17 analysis cases where the structural footing is acceptable are the cases where the footing is evaluated as deformation-controlled. The force-controlled design of footing is overly conservative when compared to the ASCE/SEI 7-10 results. In general, these NSP results were less conservative than the LSP results (except the fixed base case), which is expected.

B.3.1.2 HYPOTHESIS 1 CONCLUSIONS

In conclusion, these analyses found:

- Retrofit of foundation is required with superstructure retrofit at high seismic sites regardless of the ASCE/SEI 7-10 or ASCE/SEI 41-17 approach, unless a partial retrofit goal is selected.
- It is difficult to compare ASCE/SEI 7-10 to ASCE/SEI 41-17 results due to fundamentally different approaches. The ASCE/SEI 7-10 approach is sensitive to the Response Modification Factor and the Redundancy Factor such that ASCE/SEI 7-10 solutions can vary significantly as a result.
- Further guidance is recommended to determine when the foundation is rigid compared to the soil, or if the superstructure is sensitive to foundation deformations. The notion of relative rigidity of the footing to the soil is only applicable to small strains, whereas large strains will likely lead to soil separation from footing as well as localized yielding of the soil and footing,
- The force-controlled design of the footing is overly conservative when compared to the ASCE/SEI 7-10 results, as well as based on judgement. See Section B.9.2.2.
- Guidance on the definition of L_c in ASCE/SEI 41-17 Table 8-4 is recommended. It is unclear if it should always be the full length of the footing or taken between inflection points at flexible footings as discussed in FEMA P-2006.
- Clarity is recommended in terms of stiffness derivation, capacity calculation and acceptance criteria definition for each of the Methods. Currently all the acceptance criteria are in ASCE/SEI 41-17 Table 8-4, which also includes the modeling parameters for Method 1 and 2. See Section B.8 for further discussion.
- The acceptance criteria in ASCE/SEI 41-17 Table 8-4 are sensitive to the axial load on the footing and the footing geometry. Slight changes to load or the footing dimensions significantly change the acceptance criteria. Investigation into this is recommended for future development of the acceptance criteria tables.
- Per ASCE/SEI 41-17 Section 8.4.2.3.2, there is an exception for fixed base foundations that states “Where a shallow foundation is subject to a seismic compression force that exceeds three times the gravity load or Ac/A exceeds 0.6, the foundation overturning demand shall be treated as force controlled…”. However, it was discovered that users were not treating the foundation overturning demand as force-controlled when it fell under this category. Therefore, in providing m-factors for overturning moment actions, the revisions recommended by this committee include that if Ac/Af > 0.4, the m-factors are to be calculated from the Table 8-3.

Ultimately, based on the case study results, the use of the linear, fixed base foundation approach with overturning action m-factors in accordance with ASCE/SEI 41-17 § 8.4.2.3.2.1 provides reasonable assurance that the overturning stability and forces are accurate and the fixed base model may be used to evaluate superstructure components.

However, these results indicate that by using a linear fixed base foundation approach, the footing design may be overly conservative resulting in a massive footing, and as happened with this case, a new foundation that meets ASCE/SEI 7-10 may not meet the ASCE/SEI 41-17 acceptance criteria for the target performance objective.

B.3.2 Hypothesis 2: Bounding Requirements for LSP (Method 1)

Hypothesis 2 states that the use of overturning action m-factors in accordance with Table 8-3 provides reasonable assurance that the overturning stability and forces are accurate and, therefore, structural components have reasonable demands from the flexible base model. Also, that this statement is independent of lower and upper bound conditions; i.e. that lower bound stiffness or stiffness derivations that account for separation between the footing and soil, will provide sufficient accuracy and that the upper bound need not be analyzed to obtain reasonable analysis results.

B.3.2.1 HYPOTHESIS 2 PROCESS

LSP (linear static procedure) was performed and compared to NSP (nonlinear static procedure) analysis. The ASCE/SEI 41-17 design approach was used and compared against the ASCE/SEI 7-10 design. The acceptance ratios of the linear and nonlinear analyses are compared to test the hypothesis. For the LSP, both upper and lower bound stiffness values are evaluated.

The following analyses were completed using Models A and B per Table B-2.

A pseudo lateral force was applied in accordance with ASCE/SEI 41-17 for a specific site with an ASCE/SEI 7-10 S_{DS} = 1.0g to represent a realistic force level in a high seismicity area for the following scenarios (Linear Cases):

1. Fixed Base
2. ASCE/SEI 41-17 Method 1 Linear Soil Springs – upper and lower bound
3. K_{50} Linear Soil Springs – upper and lower bound. K_{50} springs are further explained in Section B.8.3.1.

Following completion of the LSP analysis, NSP was utilized for comparison and as a benchmark with Model D per Table B-2. The NSP Model D use Method 1 nonlinear moment rotation springs and compression-only vertical springs with expected values and no uplift capacity. These results from Model D were recorded for comparison with the above Model A and B results:

1. Total base shear
2. Foundation Soil Acceptance Ratios
3. Foundation Structure Acceptance Ratios
4. Maximum vertical deflection in retrofit footing
5. Column demands (all actions) as well as the associated capacities

6. Shear wall demands (all actions)
7. Slab demands (flexure action) as well as the associated capacities
8. Story Drift

As with Hypothesis 1, acceptance ratios are used to compare the LSP and NSP results:

- Q_{UD} / k$m$$Q_{CE}$ (ASCE/SEI 41-17 LSP Results)
- Target Displacement Rotation / Allowable Rotation (ASCE/SEI 41-17 NSP Results)
- Demand / Capacity (ASCE/SEI 7-10 Results)

B.3.2.2 HYPOTHESIS 2 RESULTS

Brief findings associated with each linear analysis case are included below. Complete analysis findings can be found in Section B.8.

1. Linear Fixed Base: Hypothesis 2's Linear Case 1, is the same site -specific loading examined as Hypothesis 1's Linear Case 7. It utilizes ASCE/SEI 41-17 and a fixed base model that includes the retrofitted foundation. See Section B.8.3.3.1 for more detailed results.
   - The retrofit footing was then evaluated for bearing pressure due to overturning using ASCE/SEI 41-17 Equation 8-10 and the overturning moment capacity is calculated with the upper-bound soil bearing capacity in accordance with ASCE/SEI 41-17 § 8.4.2.3.2. The acceptance ratio is 1.32. The overturning moment capacity is dependent on the expected vertical load P_{UD}. Further discussion on the calculation of P_{UD} is provided in Section B.5. For this and subsequent calculations, P_{UD} is equal to the unfactored, expected vertical load including the self-weight of the footing.
   - These fixed base results are compared against the ASCE/SEI 7-10 allowable bearing pressure calculation. As with Hypothesis 1, for the purposes of this evaluation, the site-specific seismic acceleration of 1g is used. The redundancy factor, ρ, is taken as 1.0. The base shear is calculated including the R-factor for a special concrete shear wall (R=6) and ASD load cases are utilized to evaluate the allowable bearing capacity for comparison. The footing is adequate for the ASCE/SEI 7-10 analysis, with a bearing pressure acceptance ratio of 0.98.
2. ASCE/SEI 41-17 Method 1 Linear Soil Springs – upper and lower bound (see Section B.8.3.3.2 for more detailed results)
   - Method 1 foundation springs are in accordance with ASCE/SEI 41-17 Figure 8-2. Spring derivation and methodology can be found in Section B.8. In accordance with ASCE/SEI 41-17 § 8.4.2, the lower bound stiffness is calculated as half of the expected value and the upper bound stiffness is calculated as twice the expected value.

- The moment demand at the base of the footing is the determined from the reaction of the rotational soil spring. Resulting footing acceptance ratio for the lower bound case is 0.61 and 0.75 for upper bound stiffness.

3. K_{50} Linear Soil Springs – upper and lower bound (see Section B.8.3.3.3 for more detailed results)
   - K_{50} boundary rotation stiffness assumes that 50% of the moment capacity is mobilized and accounts for non-service level actions and displacements (EQ actions) and includes gapping between soil and footing. Spring derivation and methodology can be found in Section B.8.3.1. The $300M_{c,foot}$ value is the expected rotational stiffness for a rectangular footing and $550Mc_{,foot}$ is applicable to an "I" shaped footing and is provided for comparison.
   - Acceptance ratios for the lower bound case are 0.38 and for the upper bound case are 0.50.

The results indicate that the ASCE/SEI 41-17 fixed base analysis provides reasonable correlation to the ASCE/SEI 7-10 foundation design. The flexible-base analysis procedures have lower acceptance ratios which is consistent with the reduced force attracted to the shear wall because of flexibility in the supporting foundation as well as higher m-factors permitted for the flexible-base analysis. The difference between acceptance ratios for lower and upper-bound analyses is relatively negligible for this case study. Following completion of the LSP analysis, NSP was utilized as comparison and benchmark for these studies. The nonlinear superstructure hinge behavior was modeled in accordance with ASCE/SEI 41-17 Chapter 10. Complete analysis findings can be found in Section B.8.3.3.4. Note that the acceptance ratios discussed in these findings are the rotations at the base of the shear wall at the target displacement compared to the allowable footing rotation from ASCE/SEI 41-17 Table 8-4.

4. Flexible Base NSP Method 3 soil springs
   - The effective fundamental period of this model is 0.70 seconds and the target displacement is 12.8 inches.
   - The acceptance ratio for the footing at the target displacement is 0.78. Therefore, the retrofit footing design is acceptable. See Section B.8.3 and Table B-32.

B.3.2.3 HYPOTHESIS 2 CONCLUSIONS

In conclusion, these analyses found:

- Stiffness bounding conclusions confirm the hypothesis:
  - For linear analyses, because the ASCE/SEI 41-17 Figure 8-2 used for spring stiffness assumes that the soil remains in contact with the footing, the results are reasonable only if the soil remains in contact with the footing Based on the soil bearing and superstructure results, if ASCE/SEI 41-17 Figure 8-2 must be used for stiffness derivation, then the lower bound Method 1 stiffness provides a fairly reasonable approach to modeling flexibility for linear procedures and is recommended to be used for the LSP. The lower-bound stiffness

provided reasonable results relative to the ASCE 7 foundation design while still including the effect of some foundation displacement in the super structure.

- Upper-bound stiffness does not yield sufficiently different results (superstructure component actions and foundation overturning acceptance ratios) to warrant the additional effort to include in the analysis procedures, and therefore need not be evaluated.

- K_{50} effective stiffness (with gapping) correlates better with nonlinear analysis methods (Method 3) and is considered more realistic and more accurate than the Method 1 Lower Bound solution (see Section B.8.3.1.2 for more information on K_{50} effective stiffness).

- There are discrepancies between the stiffness values of ASCE/SEI 41-17 Figure 8-2 (Gazetas, 1991) equations which are based on a rigid footing and elastic soil response where the soil remains in contact with the footings, and those stiffness values derived using the modulus of subgrade reaction and methods that embrace and incorporate soil separation from the footing as well as flexible and yielding structural footings. There is potential to bound and calibrate springs on the wrong (too stiff) solution. This leads to Hypotheses 3 and 5.

- See Figure B-13 for a comparison of the stiffness modeling parameters based on the different methods. Note that the ASCE/SEI 41-17 Figure 8-2 (Gazetas, 1991) parameters, even the lower-bound, are significantly higher than that of the K_{50} method. The ASCE/SEI 41-17 Figure 8-2 (Gazetas, 1991) method overestimates the stiffness. See Section B.8.3.1.4 for further information.

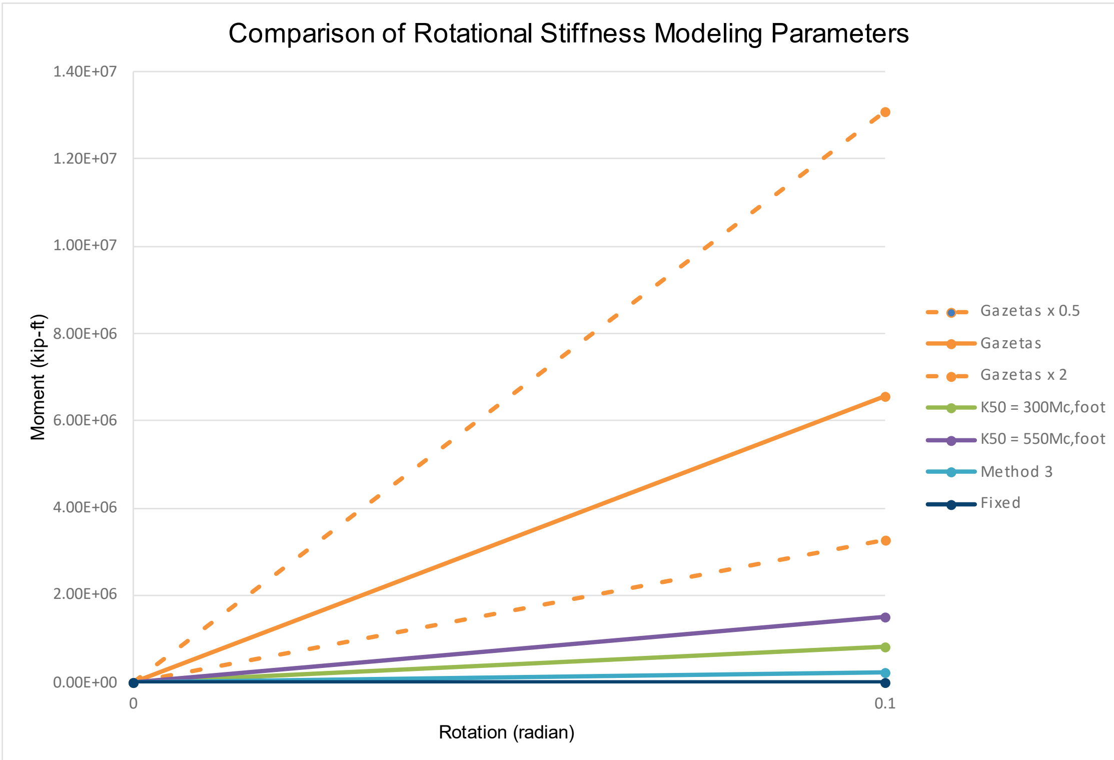

Figure B-13 Comparison of stiffness modeling parameters based on different methods.

While investigating Hypothesis 2, additional issues within the foundation provisions of ASCE/SEI 41-17 came to light. The investigations into these issues include:

- For this archetype building, the superstructure failure mechanism changes from fixed base analysis to the flexible base analysis. For the fixed base analysis, the columns remained elastic whereas in the flexible base case, the columns were failing in flexure. See section B.8.3.4 for more information.

- For this archetype building, when flexibility is introduced in a nonlinear system, when flexibility is introduced in a nonlinear system, the effective fundamental period of the model shifted significantly from 0.45 seconds for fixed base to 0.7 seconds for the flexible base NSP Method 3 soil springs proving that the fixed base model is very approximate.

- In addition to bounding of soil stiffness, the Hypothesis 2 models were used in investigating bearing strength bounding's effect on overturning moment capacity acceptance ratios. Further details of these calculations are included in Section B.4.1.2. The conclusions of that analysis include:

    - The use of upper-bound soil bearing strength for fixed-base analysis provides reasonable results compared to ASCE/SEI 7-10. For soil assessment, it is recommended that the terminology be revised to specify the use of the expected soil bearing strength with a factor of 2 to account for transient, seismic loading effects in lieu of referring to "upper-bound" strength. Using the expected soil bearing without a factor of 2 to design the structural footing provides a more reasonable structural footing size/configuration. The use of lower bound stiffness combined with upper bound soil bearing is difficult for users to follow.

  - The use of lower-bound soil bearing strength does not provide acceptable results for flexible-base analyses relative to ASCE 7 foundation design.

- Further guidance and clarity are recommended for defining P_{UF}, the expected vertical load on soil at the footing interface caused by gravity and seismic loads (formerly P_{UD}). See Section B.5 for detailed calculations.
  - Provide user further guidance on the calculation of P_{UD} (factored, unfactored, include footing weight, etc.).
  - Recommend clarification that P_{UD} be expected (unfactored) load with footing weight included as was the original intention of this calculation.
- There are multiple approaches to determining allowable rotations for an atypical foundation configuration. See Section B.6 for more information.
  - I-shaped vs. rectangular footings provide numerically different allowable rotations.
  - Guidance should be provided to the user for cases where I-shaped footings when b/L_c is not between 1 and 10.
  - Rotation demand can be determined as rotation between end points of wall or between points of contraflexure.

As a part of Hypothesis 1 and 2, the scope of Hypotheses 4 (Force versus Deformation Control) and 5 (Calibration of Springs for Method 2) were also completed, see the corresponding sections for those results.

B.3.3 Hypothesis 3: Stiffness and strength relativity between structural footing and soil

The ASCE/SEI 41-17 standard provides three methods of modeling and evaluation of shallow, flexible base foundations. The selection of the appropriate method is dependent on whether the footing is modeled using a rigid base or flexible base (building's boundary condition) assumption which is based on the relative flexibility and strength of the structural footing and the soil foundation. The footing flexibility assessment should take into account the soil bearing pressure distribution, for instance whether uplift occurs, as well as the strength of the foundation element. Where the structural footing is flexible relative to the soil foundation or yielding of the structural footing or slab occurs, the footing is classified as flexible. Methods 1 and 2 are intended for footings that are stiff and Method 3 is used for footings that are flexible relative to the soil. Figure B-14 provides a flowchart to assist the user with understanding the existing ASCE/SEI 41-17 process.

Contrary to the current ASCE/SEI 41-17 approach, the interaction between the structural system and its foundation should be considered at a high level before the nuance of structural footing to soil comparison is made, if at all. At this higher-level perspective, the decision-making process should determine if the foundation flexibility should be included or not (fixed base), and if so, should just the soil or both the structural footing and the soil be included. The hypothesis below was developed with this in mind.

As indicated in the flowchart, ASCE/SEI 41-17 § 8.4.2.1 requires that Method 3 be used where either stiffer soil relative to structural footing or a yielding foundation occurs. Equations C8-2 and C8-3 provided in ASCE/SEI 41-17 commentary should not be used to determine relative stiffness between soil and footing where uplift occurs or where the footing yields, and when used will provide incorrect results. Hypothesis 3 states that the determination of when Method 1 or 2 is acceptable to be used as opposed to Method 3 should be based on the engineer's judgement, which should include an assessment of whether the integrated curvature of the structural footing (rotations and associated vertical deformations) is significant as compared to the nonlinear soil action. It is not practical to provide an all-encompassing numerical determination of when flexible foundation modelling is required.

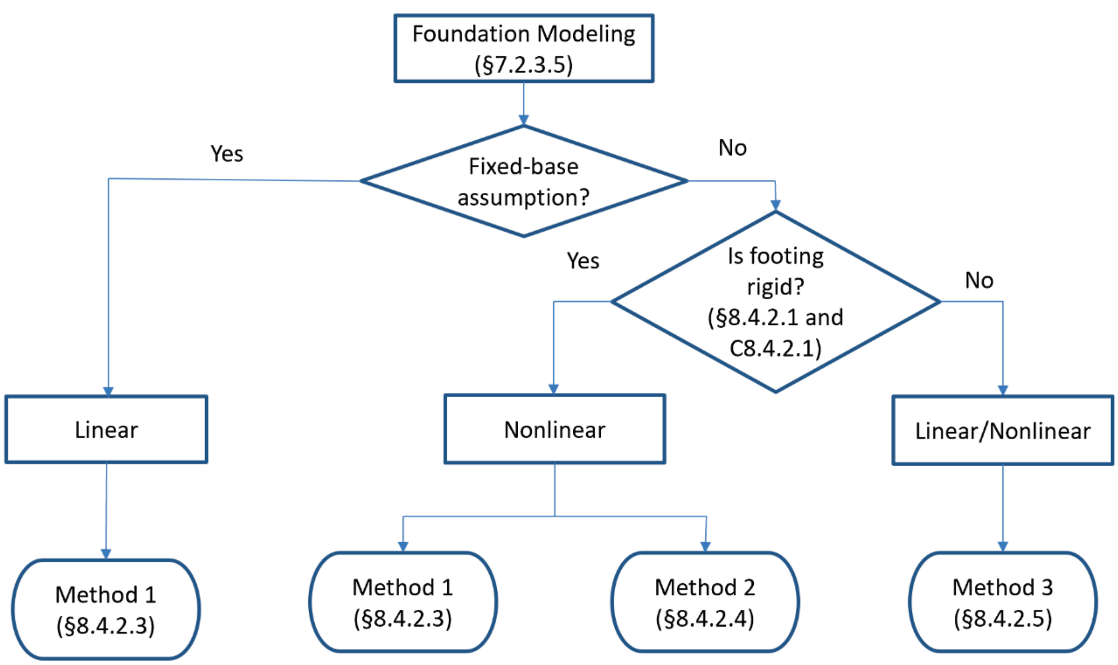

Figure B-14 Various foundation modeling approaches in ASCE/SEI 41-17.

This hypothesis was investigated through the analysis performed in other hypotheses. Based on those results, the following was determined:

- Fixed-base and flexible-base assumptions are not trivial to the performance of the superstructure. Depending on the building type and configuration, an "incorrect" base fixity assumption can incorrectly demonstrate that displacements in the superstructure are acceptable. Therefore, an assessment of foundation flexibility should be included in the provisions.

- The equations provided in ASCE/SEI 41-17 commentary for determining relative flexibility between soil and foundations are not particularly useful to the practicing engineer as they are typically oversimplified for foundation applications and they do not account for any soil gapping or foundation yielding, which is common in rocking foundations under seismic loading.

- General guidance in narrative format regarding how to assess when fixed foundations are permitted should be provided to assist the user.

B.3.4 Hypothesis 4: Force- versus deformation-controlled footing assessment

As seen in Hypotheses 1 and 2, foundations designed using force-controlled provisions were overly conservative relative to the ASCE/SEI 7-10 designed results. Therefore, further examination of the foundation design being force- versus deformation-controlled was warranted. In ASCE/SEI 41-17, concrete foundations are typically required to be evaluated as force-controlled actions in accordance with ASCE/SEI 41-17 § 10.12.3, which requires the structural component of the foundation to remain essentially elastic. Hypothesis 4 posits that foundation component yielding that meets the deformation-controlled acceptance criteria defined for actions of that component does not preclude the structure meeting the target performance objective. In some cases, controlled yielding (defined as meeting the deformation-controlled acceptance criteria) will enable the target performance objective to be met with less retrofit scope. The goal is to investigate two scenarios: one where the retrofitted footing is allowed to yield and one where it remains essentially elastic utilizing LSP and NSP.

B.3.4.1 HYPOTHESIS 4 PROCESS

The models created for Hypotheses 1 and 2 were used in this study, see below. The structural foundations were modeled as elastic concrete beams on either elastic foundation (Method 1 and K_{50}) springs or nonlinear compression-only foundation springs (Method 3).

- ASCE/SEI 7-10 (for comparison)
- Models with footing designs based on elastic beam methodology with lower-bound soil springs:
    - LSP - Fixed Base

  - LSP - Method 1 Lower Bound (Rigid Footing)
  - LSP - Method 1 Upper Bound (Rigid Footing)
  - LSP - K_{50} 300$M_{c,foot}$ (Rigid Footing)
  - LSP - K_{50} 550$M_{c,foot}$ (Rigid Footing)
- NSP - Method 3

Using these models, the structural foundation components are evaluated for each model and compared to the ASCE/SEI 7-10 calculations. Foundation strength is evaluated as both force-controlled and deformation-controlled for comparison.

B.3.4.2 HYPOTHESIS 4 RESULTS

For linear analyses, the case study shows reasonable correlation with ASCE/SEI 7 foundation design where the foundation is modelled as an elastic structural foundation on elastic soil springs (tension/compression) with unreduced elastic loads applied and the structural foundation is assessed as deformation-controlled.

Footing design based on deformation-controlled actions using acceptance criteria from the appropriate material chapters should be permitted. The use of an elastic beam modeling approach utilizing elastic springs with expected stiffness provides a reasonable approach to foundation design and evaluation. As an alternative, reduction of pseudo-elastic forces by an *m*-factor or *DCR* may provide reasonable results depending on stability of the compression-only soil spring analysis model.

There are inconsistencies between the various material chapters that will need to be resolved as some require actions to be treated as force-controlled, and others as deformation-controlled. These will need to be reexamined with these foundation provisions for consistency.

See Section B.7 and for further investigation into the modeling and assessment of flexible foundations.

B.3.5 Hypothesis 5: Calibration of Springs for Method 2

During the Hypothesis 1 and 2 studies, calibrating Method 2 to Method 1 was difficult to implement. Hypothesis 5 states that further guidance is necessary for calibration of Method 2 springs to Method 1 spring stiffness.

Method 2 provides an alternative approach for rigid foundations that uses a bed of nonlinear springs that accounts for coupling between vertical loads and moment; therefore, Method 2 is preferred over Method 1 when there is significant variation in vertical load. Based on the commentary and provisions provided in ASCE/SEI 41-17, Method 2 is recommended for nonlinear analysis procedures where yielding and gapping springs are used to represent the soil-footing interaction.

Method 2 is not intended for use with linear procedures and is expected to be too complicated for linear procedures due to the unreduced axial forces that are the basis of the linear procedures.

The moment-rotations and vertical load-deformation characteristics are modeled as a beam on a nonlinear Winkler foundation with stiffer vertical springs at the end regions of the foundation to allow for tuning of the springs to approximately match the elastic vertical and rotational stiffness (see Figure B-56). While this distribution of spring stiffness does allow accurate modeling of rotational and vertical stiffness and moment capacity, it does not ensure that the settlements are accurately predicted with Method 2 nonlinear dynamic analyses. Method 2 allows for soil acceptance criteria to be based on those of the superstructure or those of ASCE/SEI 41-17 Table 8-4 for foundation rotations for NDP.

The stiffness equations for sliding and rocking foundation stiffness are accurate for lightly loaded foundations, loaded in the elastic range. But if the foundations produce significant nonlinear actions, these stiffnesses tend to overestimate the effective foundation stiffness. This stiffness issue also affects the stiffness, strength and capacity distribution of vertical springs. The existing procedure in ASCE/SEI 41-17 attempts to select distributed springs in a way that approximates the vertical and rocking stiffnesses from elastic solutions that do not apply when nonlinearity occurs.

ASCE/SEI 41-17 implies the user should calibrate the Method 2 Winkler spring equations (Figure 8-5) to match the vertical and rotational stiffnesses from the elastic solutions (Method 1) in Figure 8-2. However, the stiffness equations noted in Figure 8-5 do not have variables to calibrate. The commentary C8.4.2.4.1 refers to Gajan et al. (2010) for reference on calibration, which provides a method of calibrating the equations. There is a possible order of magnitude difference between the two calculations. The most reasonably accurate calibration procedure is contained in the reference as it is dependent on the footing dimensions and can more closely be tuned to Method 1 findings. However, as noted in previous sections, Method 1 assumes that the soil remains in contact with the footing which would lead Method 2 to overestimate the stiffness if calibrated to Method 1. As soon as there is some yielding and footing uplift, there is a reduction in effective rotational stiffness. Therefore, calibrating Method 2 to Method 1 is not rational. Instead, Method 2 could be calibrated to the K_{50} stiffness. K_{50} boundary rotation stiffness assumes that 50% of the moment capacity is mobilized and accounts for non-service level actions and displacements (EQ actions) and includes gapping between soil and footing. Studies have shown that there is a direct correlation between moment capacity and K_{50} and therefore that it can be an accurate method for determining the secant stiffness of a rocking foundation for the point at which half of the moment capacity is mobilized. See Section B.8.3.1.2 for further information on the K_{50} stiffness derivation.

B.3.5.2 HYPOTHESIS 5 RESULTS

If Method 2 is kept, it is recommended that Method 2 be calibrated to the K_{50} stiffness. A procedure described in Gavras et al., 2015 could also be explored as a calibration method to improve the spring parameter selection for rocking and axial loading in Method 2. However, it is recommended that Method 2 is removed until a consensus is reached and provisions updated to be consistent between Method 1 and Method 3 or K_{50}. The goal of this calibration should be to more realistically model the capacity, stiffness and energy dissipation in moment-rotation hysteresis loops while sacrificing some of the accuracy of the vertical stiffness.

B.3.6 Hypothesis 6: Method 3 not intended to be used with LSP and LDP

ASCE/SEI 41-17 § 8.4.2.1 requires Method 3, which includes geometric nonlinearity at the soil-structure interface, to be used where structural foundation flexibility or yielding is significant. However, ASCE/SEI 41-17 lacks clear provisions on how to use Method 3 with LSP and LDP. Nonsensical instability can occur at the foundation interface when linear, pseudo seismic (unreduced) forces are applied to a linear model. Therefore, Hypotheses 6 states that LSP and LDP should not be used with Method 3 in the current form. However, linear Method 3 provisions are proposed in the 2023 edition to reflect industry practice, and are developed to permit a linear approach for flexible foundations relative to soil.

B.3.6.1 HYPOTHESIS 6 PROCESS – NORTH SOUTH DIRECTION

Multiple models were used for comparison in this evaluation, and the structure was examined in both the North-South direction, and the East-West direction. While all superstructures were linear elastic (Model B.1 from Table B-2), modeling varied as described below:

1. North-South LSP Method 3 model with linear elastic soil springs (ASCE/SEI 41-17 Eq. 8-11)
    - Applied Loads: unreduced pseudo elastic loads
    - Soil springs: linear-elastic (resists tension/compression)
    - Soil spring stiffness:
        - Modulus of subgrade reaction, k_{sv} (Method 3, ASCE/SEI 41-17 Eq. 8-11)
        - Expected and lower bound stiffness evaluated

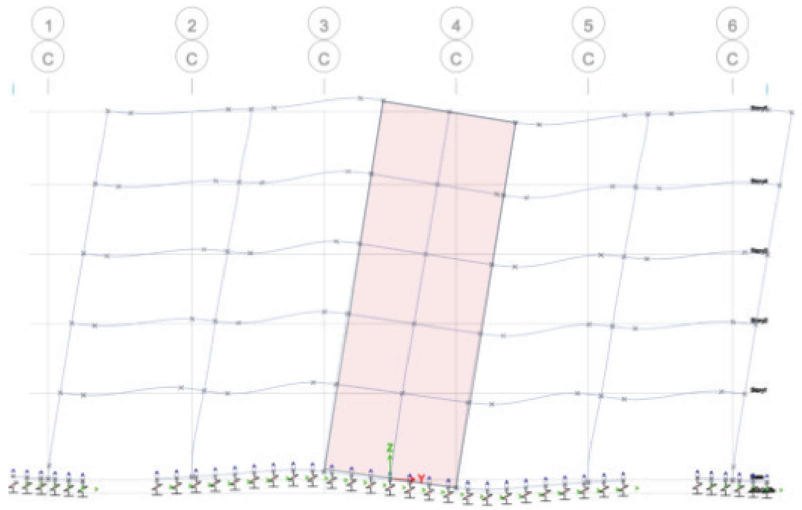

Figure B-15: North-South LSP model with Method 3 linear elastic soil springs

2. North-South LSP Method 3 model with nonlinear soil springs (compression-only) capped at expected bearing capacity
   - Pseudo elastic loads reduced by m = 4
   - Stiffness:
     - Compression stiffness = modulus of subgrade reaction, k_{sv} (Method 3, Eq. 8-11)
     - Expected stiffness evaluated

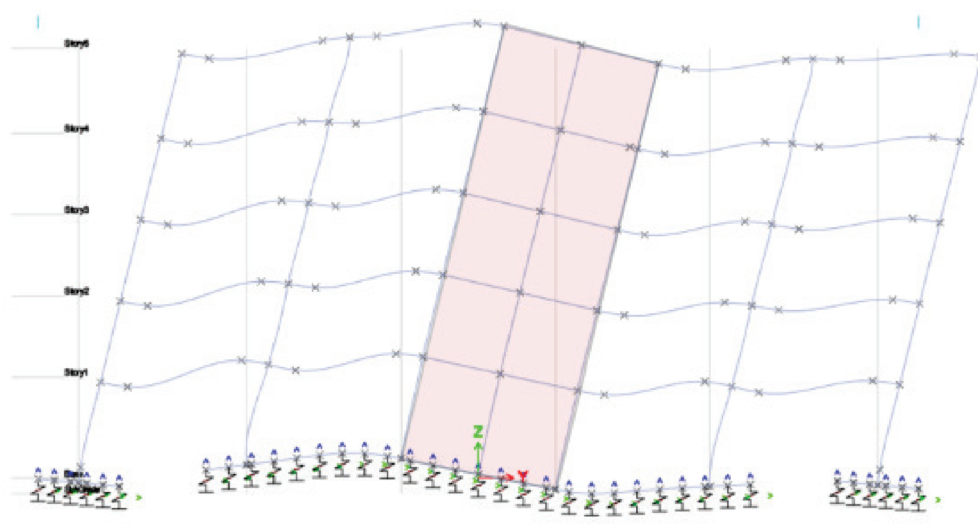

Figure B-16: North-South LSP Method 3 model nonlinear soil springs (compression-only) capped at expected bearing capacity.

These analyses require calculation of the Effective Length and Total Axial Load P_{UD}. The effective length as shown in Figure B-17 is the distance between the inflection points.

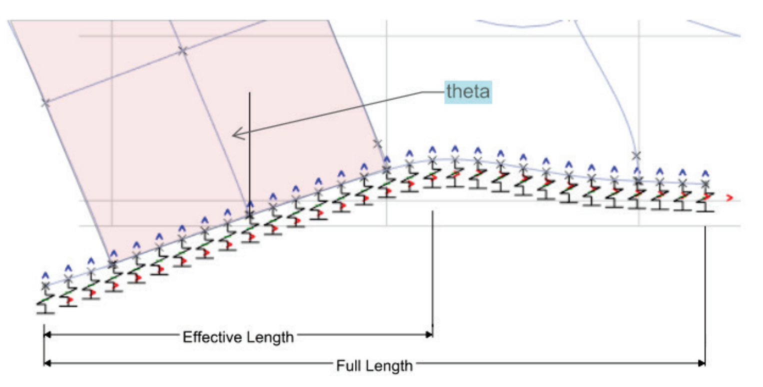

Figure B-17 Effective Length.

The total P_{UD} can be determined using statics to sum the forces, the spring reactions and the internal footing shears, as shown in Figure B-18. The total P_{UD} is based on the equation provided in ASCE/SEI 41-17 Section 8.4.2.3.1:

$$P_{UD} = P_G \text{ +- } P_E/\mathrm{DCR}$$

In this case, the spring reactions and internal footing shears are used to determined the gravity and seismic axial forces. Per ASCE/SEI 41-17 Section 8.4.2.3.1, the DCR need not be taken as less than C_1C_2 or greater than $2C_1C_2$. In this case, the maximum is used so the DCR is 2(1.1) = 2.2.

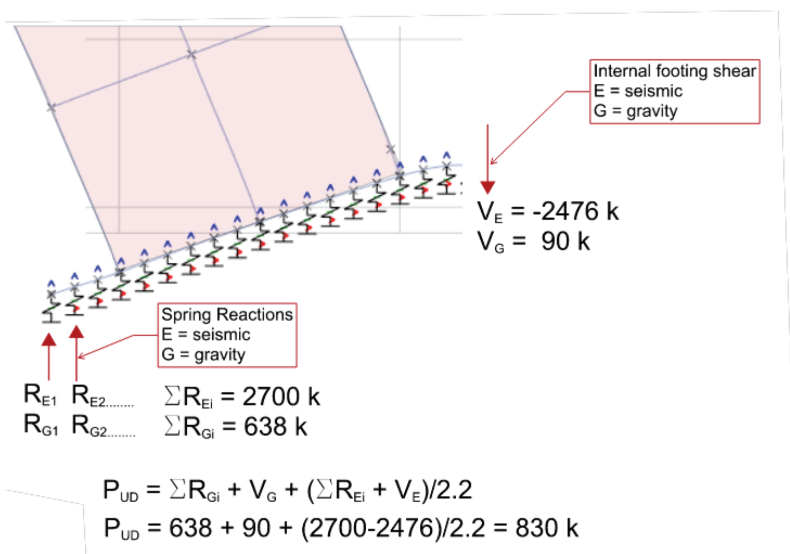

Figure B-18 P_{UD} determination.

Using the Effective Length and Total P_{UD}, the allowable rotation can be determined. An example calculation is included below in Figure B-19.

b=	9.5	ft
L =	32.3	ft
A_f=	307	ft^2
P_{UD}=	830	kips
q_C=	10.5	ksf
A_C=	79	ft^2
A_f=	307	ft^2
A_{rect} =	307	ft^2
A_{rect} -A_f/A_{rect} =	0.00	
A_C/Af=	0.26	
L_c =A_c/b =	8.31	ft
b/Lc =	1.14	
allow. rotation =	$\boxed{0.0353}$	radians, 41-17 Table 8-4
rotation =	0.017	
acceptance ratio =	0.62	

Figure B-19 Allowable rotation determination.

B.3.6.4 HYPOTHESIS 6 – NORTH SOUTH RESULTS AND CONCLUSIONS

The results of each North-South analysis are included in Table B-9 below. For the linear elastic approach, the rotation demand is determined at the base of the shear wall. It is compared to the allowable rotation determined using ASCE/SEI 41-17 Table 8-4 (as for nonlinear procedures), but modified by a factor of 0.75 to convert to a linear analysis per ASCE/SEI 41-17 Section 7.6.3.7. This factor may not be applicable for this application but was used in an endeavor to remain consistent with Chapter 7. For the nonlinear spring approach, the bearing demand is determined from analysis and compared to an expected bearing capacity with no additional m-factors applied.

Table B-8 Summary of Foundation Soil Results (N-S) Direction from Hypotheses 2 and 6

		Foundation Soil Acceptance Ratio
Hypothesis 2 LSP (See Section B.8.3.3 for calculations)	LSP – ASCE/SEI 7-10 (Section B.8.3.3.1)	0.98
	LSP - Fixed Base (Section B.8.3.3.1)	1.32
	LSP - Method 1 Lower Bound (Rigid Footing) (Section B.8.3.3.2)	0.61
	LSP - Method 1 Upper Bound (Rigid Footing) (Section B.8.3.3.2)	0.75
	LSP - K_{50} 300$M_{c,foot}$ (Rigid Footing) (Section B.8.3.3.3)	0.38
	LSP - K_{50} 550$M_{c,foot}$ (Rigid Footing) (Section B.8.3.3.2)	0.50
Hypothesis 6 (North-South)	LSP - Method 3 Expected Stiffness (Full length, Total P_{UD})	0.58
	LSP - Method 3 Expected Stiffness (Effective length, Effective P_{UD})	0.45
	LSP - Method 3 Lower Bound Stiffness[1] (Full length, Total P_{UD})	0.84
	LSP - Method 3 Lower Bound Stiffness[(1)] (Effective length, Effective P_{UD})	0.64
	*LSP - Nonlinear Method 3 Springs Expected Stiffness (*m* = 4)	0.79
Hypothesis 2 NSP	NSP - Method 3 Expected Stiffness (Full length, Total P_{UD}) (Section B.8.3.3.4)	0.78
	NSP - Method 3 Expected Stiffness (Effective length, Effective P_{UD})	0.47

(1) Lower Bound stiffness is k_{sv} (ASCE/SEI 41-17 Eq. 8-11) multiplied by 0.5.

As can be seen in the table, the proposed linear elastic approach, using lower-bound stiffness and effective footing length and axial loads, provides reasonable acceptance ratios when compared to Method 1 and NSP Method 3 analyses. However, the application of $m = 4$ may be conservative or unconservative depending on the strength capacity of the structural system of any given building and the distribution of forces therein. Therefore, this nonlinear springs approach could require further guidance.

For linear procedures, the recommended methodology from what is currently developed is the LSP Method 3 Lower Bound Stiffness with effective length and effective P_{UD} highlighted in Table B-9 above. This is more liberal than ASCE/SEI 7-10 and more conservative than ASCE/SEI 41-17 NSP, which deems it acceptable for this case.

B.3.6.3 HYPOTHESIS 6 PROCESS – EAST WEST DIRECTION

A similar process was completed in the East-West direction where the shear wall and retrofit rectangular footing are located at the corner edge of the structure. Multiple models were used for comparison in this evaluation and while all superstructures were linear elastic, modeling varied as described below:

1. East-West LSP Fixed Base
    - Applied Loads: unreduced pseudo elastic loads
    - Soil springs: N/A – fixed base

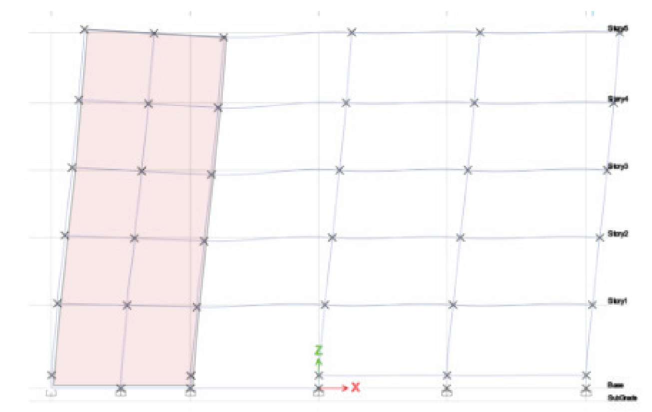

Figure B-20 East-West LSP analysis model with fixed base.

2. East-West LSP Method 3 model with linear elastic soil springs (ASCE/SEI 41-17 Eq. 8-11)
    - Applied Loads: unreduced pseudo elastic loads
    - Soil springs: linear-elastic (resists tension/compression)
    - Soil spring stiffness:
        - Modulus of subgrade reaction, k_{sv} (Method 3, ASCE/SEI 41-17 Eq. 8-11)
        - Expected and lower bound stiffness evaluated

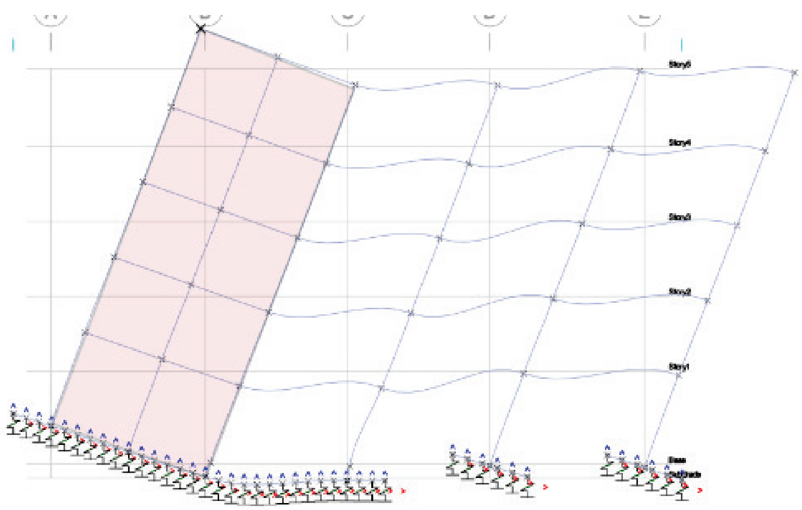

Figure B-21 East-West LSP model with Method 3 linear elastic soil springs.

3. East-West LSP Method 3 model with linear elastic soil springs (ASCE/SEI 41-17 Fig. 8-2)
   - Applied Loads: unreduced pseudo elastic loads
   - Soil springs: linear-elastic (resists tension/compression)
   - Soil spring stiffness:
     - Vertical translation stiffness from ASCE/SEI 41-17 Fig. 8-2
     - Expected stiffness evaluated

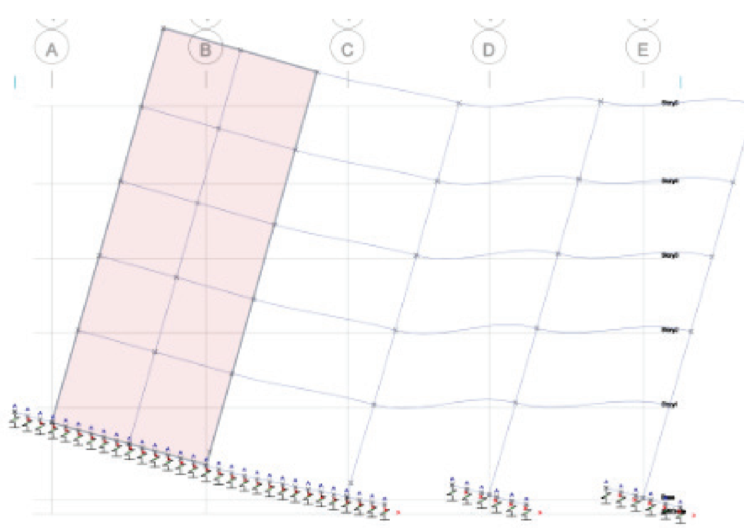

Figure B-22 East-West LSP model Method 3 linear elastic soil springs.

4. East-West LSP Method 3 model with nonlinear soil springs (compression-only) capped at expected bearing capacity
   - Pseudo elastic loads reduced by m = 4
   - Stiffness:
     - Compression stiffness = modulus of subgrade reaction, k_{sv} (Method 3, Eq. 8-11)
     - Expected stiffness evaluated

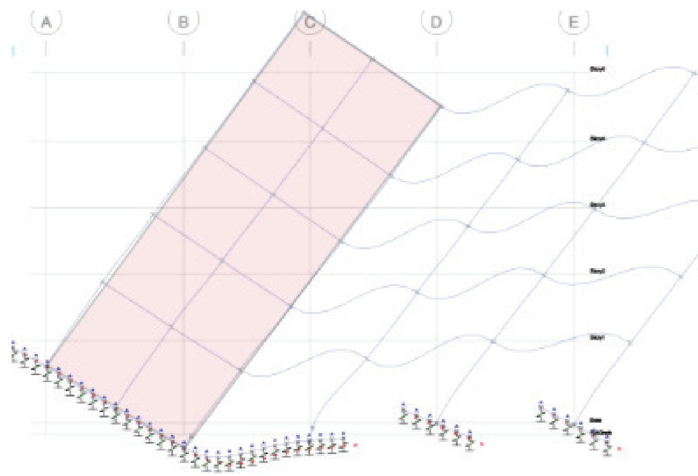

Figure B-23 East-West LSP Method 3 model nonlinear soil springs (compression-only) capped at expected bearing capacity.

5. East-West LSP Method 3 model with nonlinear soil springs (compression-only) NOT capped at expected bearing capacity
   - Pseudo elastic loads reduced by m = 4
   - Stiffness:
     - Compression stiffness = modulus of subgrade reaction, k_{sv} (Method 3, Eq. 8-11)
     - Expected stiffness evaluated

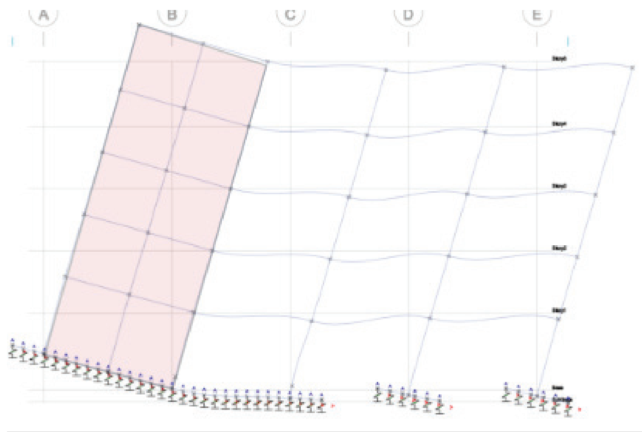

Figure B-24 East-west LSP Method 3 model nonlinear soil springs (compression-only) *not* capped at expected bearing capacity.

These results were also compared with an NSP push-over analysis in which the compression-only, nonlinear soil springs were capped at the expected bearing capacity.

B.3.6.4 HYPOTHESIS 6 - EAST WEST RESULTS AND CONCLUSIONS

The results of each East-West analysis are included in Table B-9 below. As with the North-South cases, for the linear elastic approach, the rotation demand is determined at the base of the shear wall. It is compared to the allowable rotation determined using ASCE/SEI 41-17 Table 8-4 (as for nonlinear procedures), but modified by a factor of 0.75 to convert to a linear analysis per ASCE/SEI 41-17 Section 7.6.3.7. This factor may not be applicable for this application but was used in an endeavor to remain consistent with Chapter 7. For the nonlinear spring approach, the bearing demand is determined from analysis and compared to an expected bearing capacity with no additional m-factors applied.

Table B-9: Summary of Foundation Soil Results (E-W) Direction from Hypotheses 2 and 6

			Foundation Soil Acceptance Ratio
LSP - ASCE 7-10			1.34
LSP - Fixed Base[(1)]			2.03
LSP - Method 3 ASCE/SEI 41-17 Figure 8-2 (Gazetas, 1991) Stiffness Expected			0.76
LSP - Method 3 Expected Stiffness (Full length, Total P_{UD})			0.44
LSP - Method 3 Expected Stiffness (Effective length, Effective P_{UD})			0.42

LSP - Method 3 Lower Bound Stiffness (Full length, Total P_{UD})	0.66
LSP - Method 3 Lower Bound Stiffness (Effective length, Effective P_{UD})	0.62
LSP - Nonlinear Method 3 Springs Capped/(m = 4)	N/A(2)
LSP - Nonlinear Method 3 Springs Uncapped/(m = 4)	1.27

(1) Overturning capacity includes restraint from slabs framing into structure above footing.
(2) N/A indicates soil is yielding and the soil acceptance is determined from the numerical stability of the subject model. Foundation springs are capped at expected bearing capacity of soil.

As can be seen in the table, the proposed linear elastic approach, using lower-bound stiffness and effective footing length and axial loads, provides reasonable acceptance ratios. It is unclear how to do a comparison between the uncapped and capped conditions.

Again, the application of $m = 4$ may be conservative or unconservative depending on the strength capacity of the structural system of any given building and the distribution of forces therein. Therefore, this nonlinear springs approach could require further guidance.

For linear procedures, the recommended methodology from what is currently developed is the LSP Method 3 Lower Bound Stiffness with effective length and effective P_{UD} highlighted in Table B-9 above. This is more liberal than ASCE/SEI 7-10 and more conservative than ASCE/SEI 41-17 NSP, which deems it acceptable for this case.

A summary of the superstructure results for the E-W cases, LSP Fixed Base, Method 3 Lower Bound, and NL Springs uncapped, is in included in the tables below. The fixed base analysis significantly underestimates the superstructure demand compared to flexible base procedures. Depending on structural system, acceptance ratios using a uniform m-factor could be conservative, unconservative or accurate. Therefore, again it is not recommended that a uniform value be used for all structural systems.

Table B-10 East-West Acceptance Ratios for Interior Columns for different LSP Procedures

Existing Interior Columns - Moment Acceptance Ratios (CP Limit State)							
Analysis Model	**1st Story**	**2nd Story**	**3rd Story**	**4th Story**	**5th Story**	**Max.**	**Outcome**
LSP - Fixed Base	0.39	0.15	0.20	0.19	0.20	0.39	OK
LSP - Method 3 Lower Bound	0.63	0.62	0.74	1.00	1.08	1.08	NG
*LSP - NL Springs Uncapped Demand/(m = 4)	0.85	0.58	0.67	0.95	1.85	1.85	NG
NSP - Method 3	0.00	0.00	0.00	0.00	0.05	0.05	OK

Existing Interior Columns - Shear Acceptance Ratios (CP Limit State)							
Analysis Model	**1st Story**	**2nd Story**	**3rd Story**	**4th Story**	**5th Story**	**Max.**	**Outcome**
LSP - Fixed Base	0.19	0.12	0.17	0.17	0.15	0.19	OK
LSP - Method 3 Lower Bound	0.36	0.36	0.58	0.86	0.87	0.87	OK
*LSP - NL Springs Uncapped Demand/(m = 4)	0.44	0.32	0.56	0.82	1.39	1.39	NG
NSP - Method 3							

Note: No additional m-factor applied to component capacity. Actual m-factor for structural component ranges from 4.2-2.7

Table B-11 East-West Acceptance Ratios for Exterior Columns for different LSP Procedures

Existing Exterior Columns - Moment Acceptance Ratios (CP Limit State)							
Analysis Model	**1st Story**	**2nd Story**	**3rd Story**	**4th Story**	**5th Story**	**Max.**	**Outcome**
LSP - Fixed Base	1.04	0.27	0.27	0.53	0.14	1.04	NG
LSP - Method 3 Lower Bound	5.15	0.71	1.11	1.14	0.41	5.15	NG
*LSP - NL Springs Uncapped Demand/(m = 4)	1.41	0.20	0.43	0.87	0.55	1.41	NG
NSP - Method 3	0.33	0.04	0.04	0.00	0.00	0.33	OK
Existing Interior Columns - Shear Acceptance Ratios (CP Limit State)							
Analysis Model	**1st Story**	**2nd Story**	**3rd Story**	**4th Story**	**5th Story**	**Max.**	**Outcome**
LSP - Fixed Base	1.16	0.29	0.33	0.36	0.09	1.16	NG
LSP - Method 3 Lower Bound	5.33	1.11	1.18	1.28	0.31	5.33	NG
*LSP - NL Springs Uncapped Demand/(m = 4)	1.41	0.32	0.32	0.76	0.44	1.41	NG
NSP - Method 3							

NOTE: No additional m-factor applied to component capacity. Actual m-factor for structural component ranges from 4.2-2.7

Table B-12 East-West Acceptance Ratios for Retrofit Shear Walls for different LSP Procedures

Retrofit Shear Wall - Shear Acceptance Ratios (CP Limit State)							
Analysis Model	**1st Story**	**2nd Story**	**3rd Story**	**4th Story**	**5th Story**	**Max.**	**Outcome**
LSP - Fixed Base	0.49	1.03	0.83	0.60	0.25	1.03	NG

LSP - Method 3 Lower Bound	0.30	0.80	0.58	0.01	0.10	0.80	OK
*LSP - NL Springs Uncapped Demand/(m = 4)	0.22	0.80	0.57	0.34	0.14	0.80	OK
NSP - Method 3	0.01	0.01	0.11	0.05	0.01	0.11	OK
Retrofit Shear Wall - Moment Acceptance Ratios (CP Limit State)							
Analysis Model	**1st Story**	**2nd Story**	**3rd Story**	**4th Story**	**5th Story**	**Max.**	**Outcome**
LSP - Fixed Base	0.56	0.53	0.46	0.42	0.24	0.56	OK
LSP - Method 3 Lower Bound	0.82	1.24	0.77	0.32	0.17	1.24	NG
*LSP - NL Springs Uncapped Demand/(*m* = 4)	0.56	1.04	0.66	0.38	0.21	1.04	NG
NSP - Method 3	0.00	0.00	0.00	0.00	0.00	0.00	OK

Note: No additional m-factor applied to component capacity. Actual m-factor for structural component is 4.0

Table B-13 East-West Acceptance Ratios for the Concrete Slab for different LSP Procedures

Existing Slab - Flexure Acceptance Rations (CP Limit State)							
Analysis Model	**2nd Story**	**3rd Story**	**4th Story**	**5th Story**	**Roof**	**Max.**	**Outcome**
LSP - Fixed Base	0.37	0.37	0.35	0.29	0.15	0.37	OK
LSP - Method 3 Lower Bound	1.32	1.32	1.19	1.00	0.60	1.32	NG
LSP - NL Springs Uncapped Demand/(*m* = 4)	1.52	1.49	1.38	1.27	0.77	1.52	NG
NSP - Method 3	0.25	0.22	0.20	0.16	0.00	0.25	OK

Note: No additional m-factor applied to component capacity. Actual m-factor for structural component ranges from 3.3-3.4

Table B-14 East-West Story Drift Results for different LSP Procedures

Story Drift - drift per story (in)						
Analysis Model	**2nd Story**	**3rd Story**	**4th Story**	**5th Story**	**Roof**	**Max.**
LSP - Fixed Base	0.59	0.69	0.68	0.62	0.54	0.69
LSP - Method 3 Lower Bound	2.86	2.47	2.46	2.41	2.46	2.86
LSP - NL Springs Uncapped Demand/(*m* = 4)	0.98	0.85	0.85	0.83	0.86	0.98
NSP - Method 3	1.95	1.72	1.68	1.61	1.64	1.95

Story Drift - drift ratio per story						
Analysis Model	**2nd Story**	**3rd Story**	**4th Story**	**5th Story**	**Roof**	**Max.**
LSP - Fixed Base	0.004	0.005	0.005	0.005	0.004	0.005
LSP - Method 3 Lower Bound	0.018	0.020	0.020	0.019	0.019	0.020
LSP - NL Springs Uncapped Demand/(m = 4)	0.006	0.007	0.007	0.007	0.007	0.007
NSP - Method 3	0.013	0.014	0.013	0.013	0.012	0.014

B.4 Prescriptive Expected Bearing Capacities (Proposed ASCE/SEI 41-23 Section 8.4.2.1): Bearing Capacity Determination and Bounding

During the Hypothesis 1 analysis and in setting up the case study models, multiple approaches to defining bearing capacity and its upper and lower bounds were investigated.

B.4.1 Bearing Capacity Investigation

B.4.1.1 METHODOLOGY

The expected soil bearing capacity was evaluated with the following different approaches and then compared.

- Based on allowable bearing pressure specified on the original construction drawings (ASCE/SEI 41-17 Equation 8-1).
- Based on the calculated gravity loads to the existing footing (ASCE/SEI 41-17 Equation 8-3).
- Based on the maximum force that can be transferred to the soil through the structure as limited by the deformation-controlled structural footing capacity (expected strength with m = 1).
- Based on the maximum force that can be transferred to the soil through the structure based on the force-controlled structural footing capacity as specified in ASCE/SEI 41-17 § 10.12.3.

B.4.1.2 EXISTING FOOTING DESCRIPTION

The existing building was designed and built in the early 1920s for storage. Design loading noted on the existing drawings is 300 psf live load at each floor level and 40 psf live load at the roof. Due to the high loading, the existing footing below each column is robust as shown in Figure B-25. It is important to note that footings of other buildings with more typical design live loads would likely be

18” to 24” deep. The expected rebar yield strength and concrete compressive strength are shown in Table B-15 and are based on usual testing; therefore, the knowledge factor is equal to 1.0.

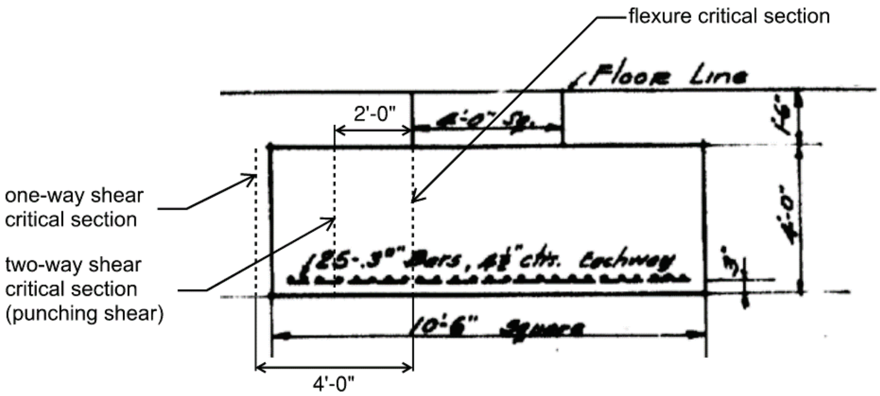

Figure B-25 Existing footing with critical sections identified.

Table B-15 Existing Footing Expected Strength

	Expected Strength	Knowledge Factor
Rebar Expected Yield Strength, f_{ye}	38.6 ksi	1.0
Concrete Expected Strength, f'_{ce}	3.2 ksi	1.0

B.4.1.3 EXPECTED SOIL BEARING CAPACITY BASED ON ALLOWABLE BEARING PRESSURE

A geotechnical report was developed for the building retrofit. The allowable bearing pressure provided in the report for dead plus live loads is 3.5 kips-per-square-foot (ksf). This is converted to a prescriptive expected bearing capacity in accordance with ASCE/SEI 41-17 Equation 8-1 and an upper-bound bearing capacity in accordance with ASCE/SEI 41-17 Figure 8-1a as shown below.

The prescriptive expected allowable bearing capacity for this spread footing is:

$$q_c = 3q_{allow} \qquad \text{(ASCE/SEI 41-17 Eq. 8-1)}$$

where:

$q_{allow,D+L}$ = 3.5 ksf (allowable bearing capacity from geotechnical report for D+L)

$$q_{ce} = 3(3.5 \text{ ksf}) = 10.5 \text{ ksf}$$

Then the upper bound soil bearing capacity based on the allowable bearing pressure can be determined:

$$q_{c,\,upperbound} = 2q_{ce} \qquad \text{(ASCE/SEI 41-17 Figure 8-1a)}$$

= 2(10.5 ksf)

= 21 ksf

B.4.1.4 EXPECTED SOIL BEARING CAPACITY BASED ON CALCULATED GRAVITY LOAD

The expected bearing capacity for shallow footings can be determined based on the gravity load action on the soil using ASCE/SEI 41-17 Equation 8-3. This results in slightly lower expected bearing capacity than as calculated based on allowable bearing pressure as shown in the calculations below. The calculated bearing demand under the original dead plus live loading in this case is higher than the allowable bearing pressure of 3,500 psf provided by the geotechnical engineer. This discrepancy is attributed to the typically conservative determination of foundation design parameters in current design. Coordination with the geotechnical engineer is recommended to determine a reasonable expected bearing capacity that is consistent with the original foundation design. For the purposes of this study, expected bearing strength determined from the allowable bearing capacity provided in the geotechnical report is utilized in the calculations (the greater 21 ksf value), and recommendations are made to modify the approach to determining expected bearing capacity from calculated gravity load.

The prescriptive expected allowable bearing capacity for this spread footing is:

q_{ce} = 1.5Q_G (ASCE/SEI 41-17 Eq. 8-3)

where:

Q_G = 1.1(Q_D + Q_L + Q_S) (ASCE/SEI 41-17 Eq. 7-1)

Note that this is in accordance with the ASCE/SEI 41-17 equations but it would have been more appropriate to use Q_D+Q_L as that would have been used in the original design.

where:

Q_D = 371.5 kips (calculated using Tributary Area)

Q_L = 0.25(496 kips) (ASCE/SEI 41-17 § 7.2.2, 300 psf floors, 40 psf roof)

Q_G = 1.1(*371.5 kips* + *0.25(496 kips)*) = 545 kips

Then to convert that into a pressure, use the area of the footing

Footing Area = ($L_{footing}$)($b_{footing}$) = (10.5 ft)(10.5 ft) = 110.25 sq ft

Q_G = (545 kips) / (110.25 sq ft) = 4.9 ksf

q_{ce} = 1.5 *(4.9 ksf) = 7.4 ksf*

Then the upper bound soil bearing capacity based on the calculated gravity load can be determined:

$q_{c,\ upperbound} = 2q_{ce}$ (ASCE/SEI 41-17 Figure 8-1a)

$= 2(7.4\ ksf)$

$= 14.8\ ksf$

B.4.1.5 STRUCTURAL FOOTING CAPACITY – EXPECTED STRENGTH

The structural footing capacity was determined by investigating these mechanisms: two-way shear, beam shear, and the flexural capacity which includes assessing reinforcement development. The critical sections for these mechanisms are shown in Figure B-25. Due to the depth of the footing, one-way shear does not apply as the critical section is outside of the footing. The calculations for each mechanism below are based on expected strength of the concrete footing. The limiting mechanism is the flexural capacity, which would limit the maximum bearing delivered to the soil to 20.0 ksf assuming expected material strengths.

First, the flexural capacity of the footing was determined:

$M_{ce} = A_s f_{ye}(d\text{-}a/2)$ (Flexural strength per ACI 318-14 § 22.3)

where:

A_s = 7.75 sq in (footing contains (25) #5 bars at 4.5 inches-on-center)

f'_{ce} = 3.21 ksi (expected compressive strength of the concrete per testing)

f_{ye} = 38.6 ksi (expected yield strength of the reinforcement per testing)

d = 45 in (effective depth per ACI 318-14)

$a = (A_s f_{ye})/(0.85 f'_{ce} b_{footing})$ (depth of equivalent rectangular stress block per ACI 318-14)

$= (7.75\ sq\ in)(38.6\ ksi) / [(0.85(3.21\ ksi)(10.5\ ft) = 0.9$

M_{ce} = (7.75 sq in)(38.6ksi)(45-0.9/2)

$= 1110\ kip\ ft$

The flexural capacity was then used to determine the maximum allowable bearing capacity to the footing (if another mechanism controlled, that mechanism would have been used in this determination instead):

$M_u = q_u[(L_{footing} - c)/2]^2(b_{footing})/2$ (flexural demand at the critical section of a square footing)

where:

c = 4 ft (per Figure B-25)

which can be rearranged to solve for q_u at $M_u = M_{ce}$:

$q_u = 2M_{ce}/\{[(L_{footing}\text{-c})/2]^2(b_{footing})\}$

$= 2(1110\text{ kip ft})/\{[10.5\text{ ft} - 4\text{ ft})/2)^2(10.5\text{ ft})$

$= 20.0$ ksf

The other mechanisms are checked at this bearing capacity to ensure that the flexural mechanism controls. The two-way shear calculations follows:

The two-way shear demand at this soil pressure can be calculated:

$V_u = q_u[(L_{footing})^2\text{-}(c+d)^2]$ (two-way shear at the critical section of a square footing)

where:

c = 4 ft (per Figure B-25)

d = 4 ft (per Figure B-25)

$V_u = (20\text{ ksf})[(10.5\text{ ft})^2\text{-}(4\text{ ft} + 4\text{ ft})^2]$

$= 925$ kips

The shear capacity is:

$V_c = 4\lambda(f'_{ce})^{0.5}b_o\, d$ (ACI 318-14)

Where:

$b_o = 4(c+d) = 4(4\text{ ft} + 4\text{ ft}) = 32\text{ ft}$

$V_c = 4\lambda(3.21\text{ ksi})^{0.5}(32\text{ ft})(4\text{ ft}) = 4178\text{ kips}$

Which is much higher than the demand, so the flexural mechanism controls over the two-way shear.

Finally, the development length is checked per ACI 318-14 Table 25.4.2.2:

$l_d = d_b(f_{ye}\Psi_t\Psi_e)/[25\lambda(f'_{ce})^{0.5}]$ (ACI 318-14 Table 25.4.2.2)

$=(0.625\text{ in})(38.6\text{ ksi})(1)(1)/[25(1)(3.21\text{ ksi})^{0.5}]$

= 17 inches

Therefore, the maximum bearing pressure delivered to the soil based on the deformation-controlled footing capacity is 20 ksf assuming expected material strengths.

B.4.1.6 STRUCTURAL FOOTING CAPACITY – FORCE-CONTROLLED

An approach to determining the maximum force that can be delivered to the soil can be based on ASCE/SEI 41-17 § 10.12.3. This section states that the foundation shall be evaluated as force-controlled; however, the capacity of the foundation components need not exceed 1.25 times the capacity of the supported vertical structural component or element (column or wall). Performing a limit state analysis, where the retrofit wall above is designed to meet the capacity of the existing footing, the flexural strength of the footing is investigated in the calculations below with force-controlled capacities. Based on this approach, the maximum bearing pressure based on these provisions is 16.7 ksf, which is also less than the upper bound soil bearing capacity based on the geotechnical recommendations.

First, the flexural capacity of the footing was determined:

$M_n = A_s f_y (d - a/2)$ (Flexural strength per ACI 318-14 § 22.3)

where:

A_s = 7.75 sq in (footing contains (25) #5 bars at 4.5 inches-on-center)

f'_c = 2.14 ksi (lower bound compressive strength of concrete)

f_y = 25.7 ksi (lower bound yield strength of the reinforcement)

d = 45 in (effective depth per ACI 318-14)

$a = (A_s f_y)/(0.85 f'_c b_{footing})$ (depth of equivalent rectangular stress block per ACI 318-14)

$= (7.75 \text{ sq in})(25.7 \text{ ksi}) / [(0.85(2.14 \text{ ksi})(10.5 \text{ ft}) = 0.9$

M_n = (7.75 sq in)(25.7 ksi)(45-0.9/2)

= 740 kip ft

1.25 M_n = 925 kip ft (ASCE/SEI 41-17 § 10.12.3)

This flexural capacity was then used to determine the maximum allowable bearing capacity to the footing:

$M_u = q_u[(L_{footing} - c)/2]^2 (b_{footing})/2$ (flexural demand at the critical section of a square footing)

where:

$c = 4$ ft (per Figure B-25)

which can be rearranged to solve for q_u at $M_u = 1.25M_n$:

$q_u = 2M_{ce}/\{[(L_{footing}\text{-c})/2]^2(b_{footing})\}$

$= 2(925 \text{ kip ft})/\{[10.5 \text{ ft} - 4 \text{ ft})/2)^2(10.5 \text{ ft})$

$= 16.7$ ksf

B.4.1.7 CONCLUSIONS

These results demonstrate that the determination of expected bearing capacity using gravity loading can provide conservative values compared to the use of allowable bearing pressure from geotechnical studies. It also shows that it is important to evaluate the structural footing strength in addition to the soil bearing capacity as it may be the governing mechanism in the load path, particularly if the footing strength is evaluated as force-controlled.

Table B-16 Summary of Bearing Pressure Capacities

Methodology	Bearing Capacity
ASCE/SEI 41-17 Equation 8-1	21.0 ksf (Upper-bound)
ASCE/SEI 41-17 Equation 8-3	14.8 ksf (Upper-bound)
Footing Capacity (Deformation-Controlled)	20 ksf (limit state analysis)
Footing Capacity (Force-Controlled)	16.7 ksf (limit state analysis)

B.4.1.8 RECOMMENDED CHANGES

Where information on soil bearing capacity in not available in either the construction documents or a geotechnical report, prescriptive expected bearing capacity may be calculated with a 2.5 factor applied to the calculated design gravity loads.

B.4.2 Investigation of Soil Bearing Capacity Bounding

Soil bearing capacity bounding was investigated as a part of Hypothesis 2.

B.4.2.1 METHODOLOGY

Strength bounding was investigated to determine its effect on overturning moment capacity acceptance ratios for the retrofit footing designed using ASCE/SEI 7-10. Current ASCE/SEI 41-17 provisions permit the use of upper-bound bearing capacity for both fixed base (ASCE/SEI 41-17 § 8.4.2.3.2.1) and flexible base (ASCE/SEI 41-17 § 8.4.2.3.2.2) procedures. Table B-17 and

Table B-18 summarize the results of utilizing lower and upper bound bearing capacity when calculating overturning moment capacity. The expected moment capacity is determined using statics to sum forces about point A in Figure B-26. An example calculation for the LSP Fixed Base utilizing upper-bound bearing capacity is included below.

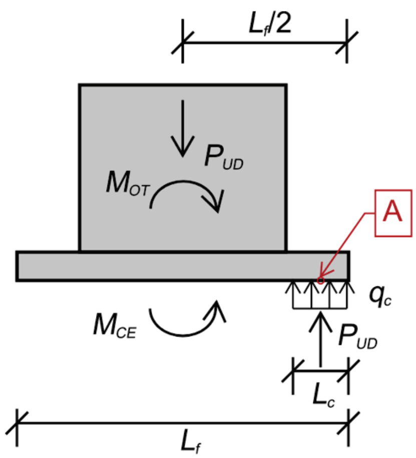

Figure B-26 Overturning and resisting forces on an isolated footing.

The expected moment capacity is calculated:

$M_{CE} = 0.5(L_f\, P_{UD})(1\text{-}q/q_c)$ (ASCE/SEI 41-17 Eq. 8-10)

where:

$L_f = 70.5$ ft (footing length per Figure B-10)

$A_f = 612$ sq ft (footing area per Table B-3)

$q_{c,\ upperbound} = 2q_{ce} = 21$ ksf (ASCE/SEI 41-17 Figure 8-1a)

$P_{UD} = 1662$ kips (Table B-20, $P_{UD} = P_G$ +- P_E/DCR)

$q = P_{UD}/A_f$ = (1662 kips)/(612 sq ft) = 2.71 ksf

M_{CE} = 0.5[*(70.5 ft)(1662 kips)*][1-*(2.71 ksf)/(21 ksf*)]

= *50,964 kip ft*

Then, by comparing this capacity to the demand per the ETABS model, the acceptance ratio can be identified:

M_{base} = 269,427 kip ft

Required m = M_{base} / M_{CE} = (269,427 kip ft) / (50964 kip ft) = 5.3

Allowable m = 4 (ASCE/SEI 41-17 Section 8.4.2.3.2.1)

Acceptance Ratio = Required m / Allowable m = 5.3/4 = 1.32

B.4.2.2 RESULTS AND CONCLUSIONS

For the fixed base case, the m-values are determined per ASCE/SEI 41-17 Section 8.4.2.3.2.1. For all remaining cases, the m-values are linearly interpolated based on the b/L_c and A_c/A_f values as described in ASCE/SEI 41-17 Table 8-3. The use of upper-bound soil bearing strength for fixed-base analysis provides reasonable results compared to ASCE/SEI 7-10 with an acceptance ratio relatively close to 1.0 as shown in Table B-17. The use of lower-bound soil bearing strength does not provide acceptable results for fixed base or flexible-base analyses, with acceptance ratios greater than 1.0 for the LSP – Method 1, using both upper or lower bound spring stiffnesses properties as shown in Table B-18.

Table B-17 Summary of Bearing Capacity Bounding to Determine Moment Capacity

Model	P_{UD} (kip)	q (ksf)	M_{base} (k-ft)	Upper Bound Strength				
				q_c (ksf)	M_{CE} Upper (k-ft)	required m	allowable m	Acceptance Ratio
LSP - Fixed Base	1,660	2.71	269,427	21.0	50,948	5.29	4.00	1.32
LSP - Method 1 Lower Bound Stiffness (Rigid Footing)	1,586	2.59	177,978	21.0	49,002	3.63	6.00	0.61
LSP - Method 1 Upper Bound Stiffness (Rigid Footing)	1,611	2.63	224,538	21.0	49,672	4.52	6.00	0.75
LSP - K_{50} 300$M_{c,foot}$ (Rigid Footing)	1,586	2.59	111,809	21.0	49,003	2.28	6.00	0.38
LSP - K_{50} 550$M_{c,foot}$ (Rigid Footing)	1,586	2.59	146,138	21.0	49,002	2.98	6.00	0.50

(1) For more information on the K_{50} stiffnesses, see Section B.8.3.1.2,
(2) The gravity load distribution between the different models varies slightly due to differences in foundation stiffness.

Table B-18 Summary of Bearing Capacity Bounding to Determine Moment Capacity

Model	P_{UD} (kip)	q (ksf)	M_{base} (k-ft)	Lower Bound Strength				
				q_c (ksf)	M_{CE} Lower (k-ft)	required m	allowable m	Acceptance Ratio
LSP - Fixed Base	1,660	2.71	269,427	5.25	28,284	9.53	4.00	2.38
LSP - Method 1 Lower Bound (Rigid Footing)	1,586	2.59	177,978	5.25	28,310	6.29	2.06	3.05
LSP - Method 1 Upper Bound (Rigid Footing)	1,611	2.63	224,538	5.25	28,314	7.93	2.00	3.97
LSP - K_{50} 300$M_{c,foot}$ (Rigid Footing)	1,586	2.59	111,809	5.25	28,310	3.95	2.06	1.92
LSP - K_{50} 550$M_{c,foot}$ (Rigid Footing)	1,586	2.59	146,138	5.25	28,310	5.16	2.09	2.50

(1) For more information on the K_{50} stiffnesses, see Section B.8.3.1.2,
(2) The gravity load distribution between the different models varies slightly due to differences in foundation stiffness.

For the fixed-base analysis procedure, the use of upper-bound soil bearing strength provides reasonable results compared to ASCE/SEI 7-10. If the expected or lower-bound soil bearing strength were to be used instead, the *m*-factor or footing size would have to be increased to provide comparable results. Increasing the footing size was deemed too conservative. Note that if the *m*-factors for the fixed-base procedures are increased, they could become equal to or greater than the *m*-factors used for the flexible-base procedure (ASCE/SEI 41-17 Table 8-3), which is counterintuitive to the general concept that the fixed-base procedure should provide a more conservative design. Therefore, we suggest that the soil bearing strength equivalent to the upper-bound strength continue to be used for the fixed-base procedure, in which case the terminology will be revised to specify the use of the expected soil bearing strength with a factor of 2 to account for transient, seismic loading effects. Discussions with the geotechnical community have agreed that between 1.5 and 2 would be a more accurate estimate of the overstrength due to the short transient nature of the earthquake loading.

These results can also be compared to those of the Nonlinear Static Procedure as presented in Section B.10 with acceptance ratios in Table B-41. All of the soil acceptance ratios for the NSP methods explored ranged from 0.29 to 0.74 which is more in line with the upper-bound strength for the LSP flexible-base procedures. Lower-bound soil bearing strengths do not provide acceptable results for linear procedures. In any case, design of the structural foundation should be performed with the expected bearing capacity and bounding is not needed.

B.4.2.3 RECOMMENDED CHANGES

The use of upper and lower bound properties for soil bearing should be eliminated and the expected ultimate capacity, q_{cDA}, which includes a factor of 2 for short duration seismic loading, should be utilized for linear analysis procedures The proposed commentary changes delete the statement "To allow for soil variability or uncertainty, an upper- and lower-bound approach to defining stiffness and capacity is required to evaluate the sensitivity of the structural response to these parameters." Instead, it stresses that it is "important that geotechnical engineers report the average expected results obtained and the actual factor of safety applied to arrive at design values for soil strength and stiffness. In the past, design values recommended by geotechnical engineers were often consistent with lower-bound strengths."

B.5. Foundation Overturning Capacity (Proposed ASCE/SEI 41-23 Section 8.4.4.1.1.1): Expected Vertical Load P_U

B.5.1 Motivation

During the investigation of Hypothesis 2, it was found that there is a lack of clarity in how to calculate the expected vertical load, P_{UD} (which is proposed to be P_U in the ballots for the upcoming ASCE/SEI 41-23), in determining the overturning moment capacity, M_{CE}, in Equation 8-10. The P_{UD} is defined in ASCE/SEI 41-17 Section 1.2.2.1 as the expected vertical load on soil at the footing interface caused by gravity and seismic loads, however further guidance on if this includes load factors or the footing dead load is unclear.

B.5.2 Technical Studies

ASCE/SEI 41-17 provides multiple definitions of P_{UD}. The footing acceptance ratio is sensitive to the P_{UD} value. P_{UD} is defined in ASCE/SEI 41-17 §1.2.2.1 as both the expected vertical load on soil as well as the deformation-controlled axial force. The latter definition includes load combination factors. The following P_{UD} calculations were investigated to determine the effect on the calculation of M_{CE}.

1. Expected load (without retrofit footing dead load and no load factors)
2. Expected load (with retrofit footing dead load and no load factors)
3. P_{UD} Factored – $0.9P_G$ (without retrofit footing dead load)
4. P_{UD} Factored – $1.1P_G$ (without retrofit footing dead load)

While there are other conditions that could be examined, these analyses were deemed adequate to form a conclusion. Often in new construction, the weight of the footing is not included and this assumption was used for the factored load scenarios.

B.5.2.1 P_{UD} - EXPECTED LOAD

ASCE/SEI 41-17 § 8.4.2.3.1 states that "the expected vertical load P_{UD} is taken as the maximum action that can be developed based on a limit-state analysis considering the expected strength of the components delivering force to the footing; alternatively, the expected vertical load is determined by dividing the seismic linear elastic load by the maximum demand capacity ratio (DCR) of the components in the load path and summing with the gravity loads." In addition, the following equation is provided:

$$P_{UD} = P_G \pm P_E/\mathrm{DCR}$$

Expected loads were calculated based on this equation with and without the dead load of the footing included. For this case study, vertical seismic loads are essentially zero, so P_{UD} is equal to the gravity load.

B.5.2.2 P_{UD} - FACTORED LOAD

P_{UD} is also calculated using 0.9 and 1.1 load factors in accordance with ASCE/SEI 41-17 § 7.2.2.

B.5.2.3 RESULTS AND CONCLUSIONS

The models created for Hypothesis 2 were used in the parameter study.

The results of this parameter study are shown in Table B-19 (expected load without retrofit footing dead load and no load factors), Table B-20 (expected load with retrofit footing dead load and no load factors), and Table B-21 (P_{UD} factored two ways without retrofit footing dead load). The calculation of P_{UD} affects the overturning soil bearing acceptance ratio by 20% to 30%. The P_G used in these calculations does not include live load. For clarity for users and to be consistent with the original intent of these provisions, we recommend that the text be revised to clarify the nomenclature and calculation of the expected axial load to exclude any load factors (1.1 or 0.9 in ASCE/SEI 41-17 Equations 7-1 and 7-2) and to include the self-weight of the footing. No live load is to be included.

Table B-19 Summary of Methods for Calculating P_{UD} – Expected Load (without Footing Dead Load)

P_{UD} Summary – Expected Load (without Footing Dead Load)								
Model	Initial Fundamental Period (seconds)	Expected Load (w/ DCR, without Footing DL)						
		P_G[2] (kips)	P_E (kips)	C_1C_2	DCR	DCR Used	P_{UD} (kips)	Footing Acceptance Ratio
LSP - Fixed Base	0.43	1295	-6.00	1.10	3.00	2.20	1,292	1.64
LSP - Method 1 Lower Bound (Rigid Footing)	0.58	1219	0.09	1.10	2.38	2.20	1,219	0.65
LSP - Method 1 Upper Bound (Rigid Footing)	0.50	1244	0.06	1.10	2.89	2.20	1,244	0.81
LSP - K_{50} 300$M_{c,foot}$ (Rigid Footing)	0.65	1219	0.22	1.10	6.23	2.20	1,219	0.41
LSP - K_{50} 550$M_{c,foot}$ (Rigid Footing)	0.60	1219	0.17	1.10	5.24	2.20	1219	0.54

(1) Expected Load (w/ DCR) = gravity load combined with axial seismic forces divided by maximum DCR (either with or without footing DL)

(2) The gravity load distribution between the different models varies slightly due to differences in foundation stiffness.

Table B-20 Summary of Methods for Calculating P_{UD} – Expected Load (including Footing Dead Load)

P_{UD} Summary – Expected Load (including Footing Dead Load)								
Model	Initial Fundamental Period (seconds)	Expected Load (w/ DCR, w/ Footing DL)						
		P_G (kips)	P_E (kips)	$C_{(1)}C_{(2)}$	DCR	DCR Used	P_{UD} (kips)	Footing Acceptance Ratio
LSP - Fixed Base	0.43	1662	-6.00	1.10	3.00	2.20	1660	1.32
LSP - Method 1 Lower Bound (Rigid Footing)	0.58	1586	0.09	1.10	2.38	2.20	1586	0.52
LSP - Method 1 Upper Bound (Rigid Footing)	0.50	1611	0.06	1.10	2.89	2.20	1611	0.65
LSP - K_{50} 300Mc,foot (Rigid Footing)	0.65	1586	0.22	1.10	6.23	2.20	1586	0.33
LSP - K_{50} 550Mc,foot (Rigid Footing)	0.60	1586	0.17	1.10	5.24	2.20	1586	0.43

(1) Expected Load (w/ DCR) = gravity load combined with axial seismic forces divided by maximum DCR (either with or without footing DL).
(2) The gravity load distribution between the different models varies slightly due to differences in foundation stiffness.

Table B-21 Summary of Methods for Calculating P_{UD} – Factored Load (ASCE/SEI 41-17 Load Combinations without footing dead load)

Effects of P_{UD} on Overturning Action to Assess Soil Bearing Capacity					
Model	Initial Fundamental Period (seconds)	P_{UD} Factored: $0.9P_G$		P_{UD} Factored: $1.1P_G$	
		P_{UD} (kips)	Footing Acceptance Ratio	P_{UD} (kips)	Footing Acceptance Ratio
LSP - Fixed Base	0.43	1159	1.81	1530	1.42
LSP - Method 1 Lower Bound (Rigid Footing)	0.58	1097	0.72	1447	0.56
LSP - Method 1 Upper Bound (Rigid Footing)	0.50	1119	0.89	1477	0.70
LSP - K_{50} $300M_{c,foot}$ (Rigid Footing)	0.65	1097	0.45	1447	0.35
LSP - K_{50} $550M_{c,foot}$ (Rigid Footing)	0.60	1096	0.59	1447	0.46

Notes: P_{UD} factored = maximum vertical elastic forces delivered to the retrofit footing with load combinations per ASCE/SEI 41-17 7.2.2

B.5.3 Recommended Changes

Based on the results of this study, P_{UD} used in this M_{CE} equation is recommended to be redefined as P_U to avoid confusion with other P_{UD} definitions as it includes a reduction in the earthquake axial load demand by a limit state analysis which is similar to demands to force controlled elements. For clarity for users and to be consistent with the original intent of these provisions, we recommend that the text be revised to clarify the nomenclature and calculation of the expected gravity axial load to exclude any load factors (1.1 or 0.9 in ASCE/SEI 41-17 Equations 7-1 and 7-2) and to include the self-weight of the footing. Therefore, we recommend that in defining the gravity load P_G used in this equation, it is not recommended to reference ASCE/SEI 41-17 Equation 7-1 which includes a load factor. No live load is to be included.

B.6. Seismic Overturning Resisted by Axial and Moment Action (Proposed ASCE/SEI 41-23 Section 8.4.4.1.1.3): Bi-Directional Loading

B.6.1 Motivation

Currently in the standard, overturning acceptance is addressed only for unidirectional moment for a rectangular or I-shaped footing. Provisions are required to allow for assessment of footings under bi-directional loading.

B.6.3 Technical Studies

The design of a retrofit footing for loading in the east-west direction was used to investigate ASCE/SEI 41-17 foundation analysis provisions for a corner wall condition with axial earthquake load from frame action including L-shaped foundation and other complexities. Retrofit footings designed using ASCE/SEI 7-10 and ASCE/SEI 41-17 were compared similar to other hypotheses.

B.6.3.1 METHODOLOGY AND ASSUMPTIONS

Model Summary:

- E-W Direction Seismic Loading
- Fixed Base Model
- Columns fixed at base
- Retrofit shear walls modeled as frame element fixed at base with rigid links to adjacent columns
- Includes 5% accidental torsion (ASCE/SEI 41-17 §7.2.3.2.1 & ASCE/SEI 7-10 §12.8.4.2)
- For ASCE/SEI 7-10, $\rho = 1.0$ consistent with previous case studies
- Bi-directional loading
    - ASCE/SEI 7-10 §12.5.4 requires design for 100% in primary direction and 30% in perpendicular direction
    - ASCE/SEI 41-17 §7.2.5 only requires multidirectional effects for certain conditions.

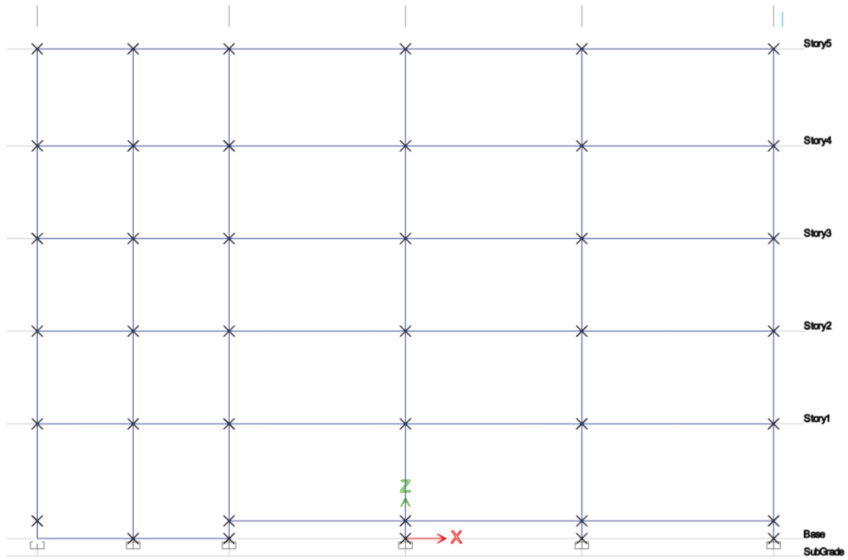

Figure B-27 Elevation from fixed Base model used for Bi-Directional loading analysis, the corner wall is between Gridlines A and A.5.

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B.6.3.2 RECTANGULAR RETROFIT FOOTING

For loading in east direction, the net earthquake reaction is upward due to frame action, so a retrofit footing extending to adjacent bays is investigated by evaluating and comparing the retrofit footing using ASCE/SEI 7-10 and ASCE/SEI 41-17.

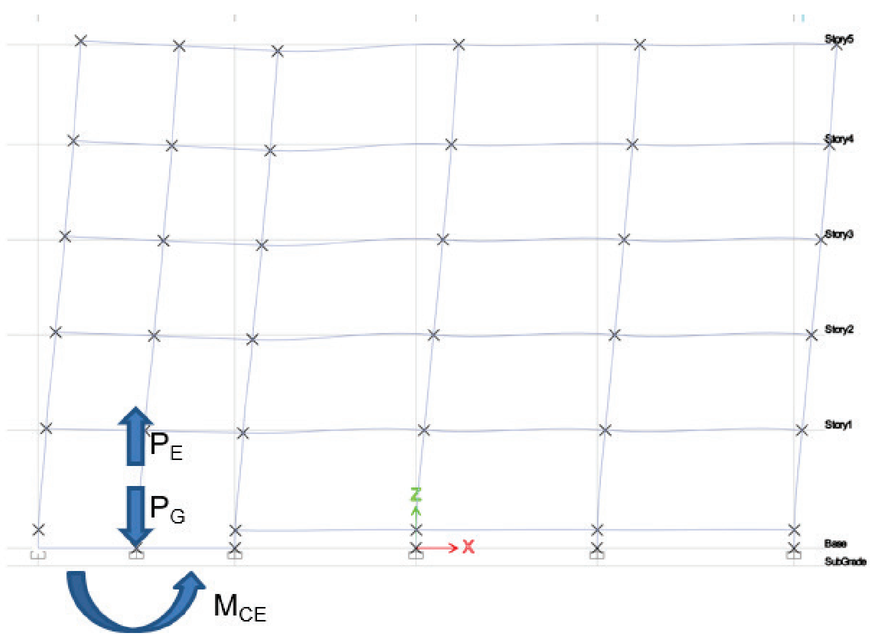


Figure B-28 Frame elevation showing forces for loading in the east direction.

The rectangular footing extends one bay beyond the shear wall, see Figure B-29. This engages additional dead load that reduces the uplift at the foundation. The actual footing, comprised of existing 10'-6" square footings and new connecting grade beams, is idealized as rectangular, see Figure B-11.

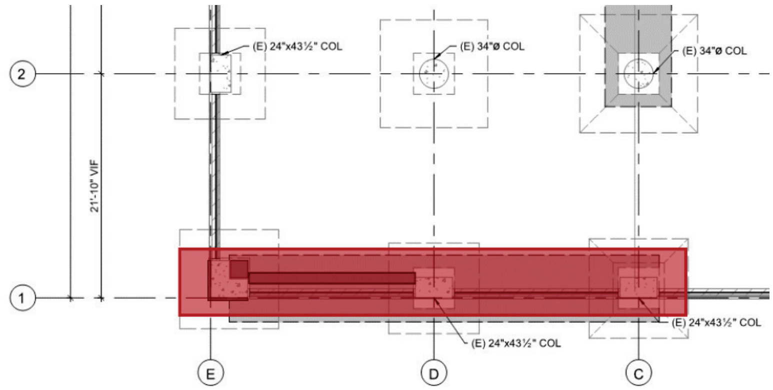


Figure B-29 Foundation plan showing Idealized proposed rectangular foundation at corner wall.

For ASCE/SEI 7-10 design, the bearing pressure is evaluated using an elastic triangular distribution assuming the new footing is rigid compared to the soil.

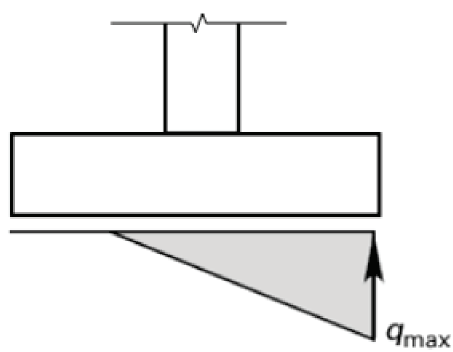


Figure B-30 Elastic triangular bearing pressure distribution at rectangular footing.

For ASCE/SEI 41-17, the expected moment capacity M_{CE} can be derived by summing moments about the center of the resulting compression block from P_{UD}, while neglecting the restoring force from perpendicular slabs at each floor as shown in Figure B- 31 and in the following equations. The footing would transfer the loads if designed as elastic/force-controlled so while there are other possible cases to study, the behavior described is one realistic possibility.

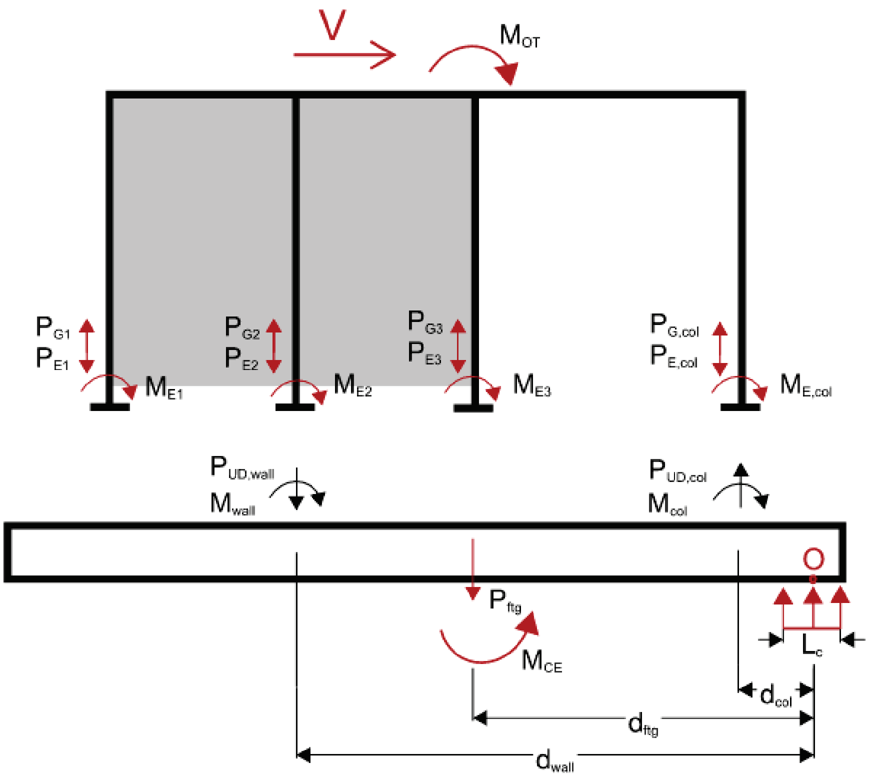


Figure B- 31 Forces acting at concrete footing for expected moment capacity derivation that neglects the restoring force from perpendicular slabs at each floor.

Using statics, the expected moment capacity can be determined:

$P_{G,wall} = P_{G1}+P_{G2}+P_{G3}$

$P_{E,wall} = P_{E1}+P_{E2}+P_{E3}$

$P_{UD,wall} = P_{G,wall} +/- P_{E,wall}/DCR$

$M_{wall} = M_{E1}+M_{E2}+M_{E3}+(P_{E1}-P_{E3})L_{wall}/2$

$P_{UD,col} = P_{G4} +/- P_{E4}/DCR$

$M_{col} = M_{E4}$

$M_{CE} = SP_{UD,i}d_i = P_{UD,wall}d_{wall} + P_{ftg}d_{ftg} + P_{UD,col}d_{col}$

$M_{OT} = M_{wall} + M_{col}$

Using the equations determined above, we can determine the expected moment capacity:

$M_{CE} = SP_{UD,i}d_i = P_{UD,wall}d_{wall} + P_{ftg}d_{ftg} + P_{UD,col}d_{col}$

Where:

$P_{UD,wall} = P_{G,wall}$ +/- $P_{E,wall}/DCR$

Where:

$P_{G,wall} = P_{G1}+P_{G2}+P_{G3}$ = 533 kips

$P_{E,wall} = P_{E1}+P_{E2}+P_{E3}$ = -1285 kips (this is due to the frame action or the coupling between the wall and column)

$P_{UD,wall}$ = (533 kips) +- (1285 kips/2.2) = -51 kips

$P_{UD,col} = P_{G4}$ +/- P_{E4}/DCR = (221 kips) +- (-44 kips/2.2) = 201 kips

$M_{col} = M_{E4}$ = 2036 kip ft

M_{CE} = (-51 kips)(36.1 ft) +(201 kips)(4.1 ft) +(298 kips)(25 ft) = 6440 kip ft

And the overturning moment demand:

$M_{OT} = M_{wall} + M_{col}$

Where:

$M_{wall} = M_{E1}+M_{E2}+M_{E3}+(P_{E1}-P_{E3})L_{wall}/2$

= 154,385 kip ft

$M_{OT} = M_{wall} + M_{col}$ = (154,385 kip ft) + (2036 kip ft) = 156,421 kip ft

As M_{OT} is greater than M_{CE}, this retrofit footing is not adequate.

Alternatively, M_{CE} can be derived by summing moments about the center of the resulting compression block from P_{UD} and including the restoring force from perpendicular slabs at each floor.

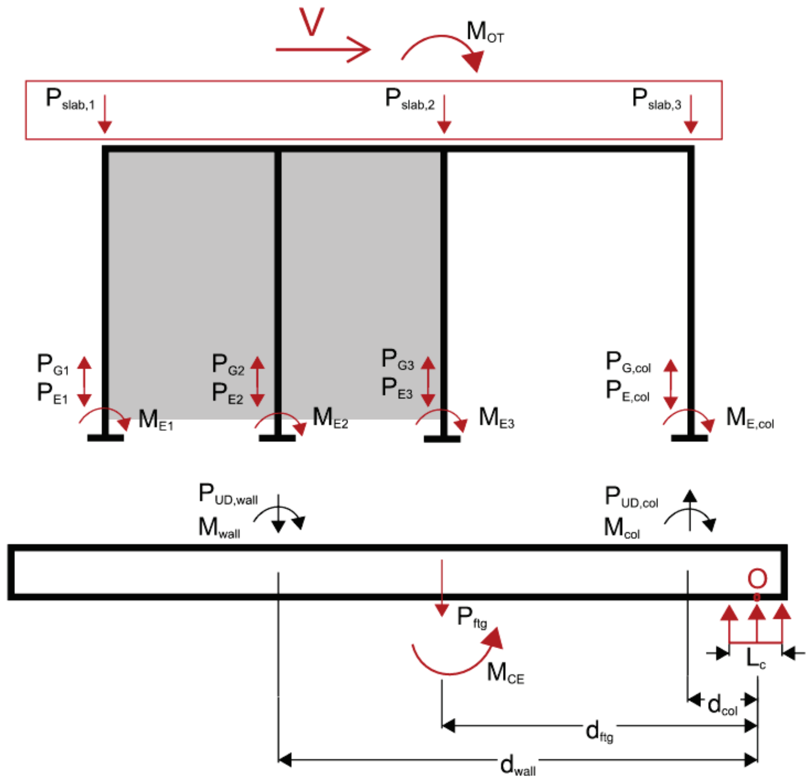


Figure B-32 Forces acting at concrete footing for expected moment capacity derivation that includes the restoring force from perpendicular slabs at each floor.

Details calculations are not provided for this option but would give similar results to the calculations above. For all scenarios with ASCE/SEI 7-10 or ASCE/SEI 41-17, a rectangular retrofit footing is not acceptable, so an L-shaped footing, which extends to adjacent bay perpendicular to retrofit shear wall, is investigated.

B.6.3.3 L-SHAPED RETROFIT FOOTING

The retrofit L-shaped footing extends one bay perpendicular to the retrofit shear wall and is evaluated for the effects of frame action as well as concentrated moment. The actual footing, comprised of existing 10'-6" square footings and new connecting grade beams, is idealized as two rectangular shapes forming an L, see Figure B-12.

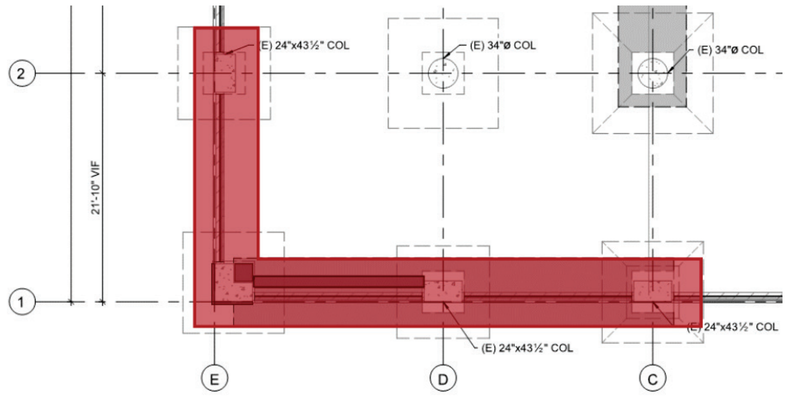


Figure B-33 Foundation plan showing idealized proposed L-shaped foundation at corner wall.

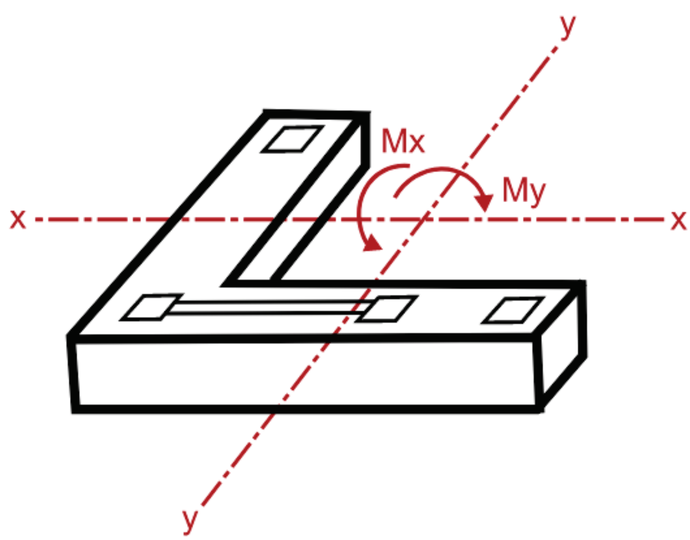


Figure B-34 L-shaped footing with axes and moment shown.

For ASCE/SEI 7-10, the footing is assumed to be rigid and loading includes moment in each direction as well as axial loads, which are combined to determine the maximum bearing pressure described in the equation below:

$$M_{OT} = \left(\frac{P}{A}\right) + \left(\frac{M_y x}{I_y}\right) + \left(\frac{M_x y}{I_x}\right)$$

The acceptance ratio represents the applied bearing pressure compared to the allowable of 4.66 ksf.

For ASCE/SEI 41-17, the expected moment capacity M_{CE} is calculated in each direction with P_{UD} determined similar to a rectangular footing but including the column in the perpendicular direction. Based on the current provisions which dictate that overturning is based on a rectangular compression block, and provided that the footing is strong enough to engage the adjacent column weight, this approach is reasonable.

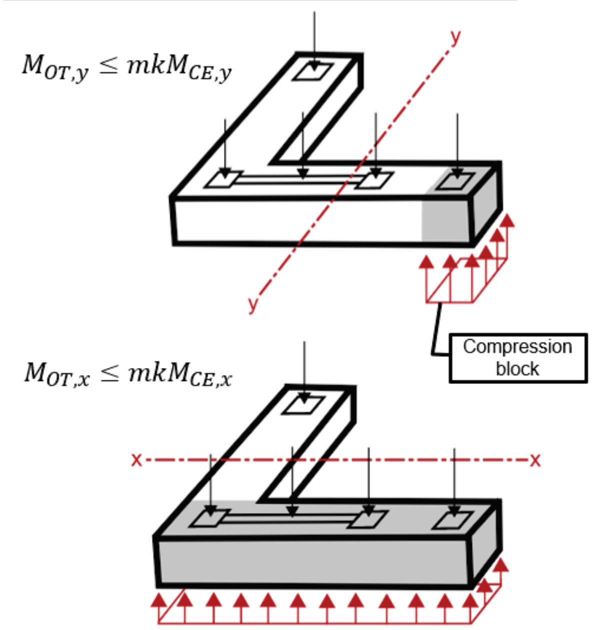


Figure B-35 Compression blocks for each direction at the L-Shaped Footing.

M_{CE} is calculated as the sum of moments about center of calculated compression block from P_{UD}. The combined acceptance ratio is determined by square root sum of squares below per previous direction. However, note that the ASCE/SEI 41-23 committee is currently removing the square root as described in Section B.6.4.

$$\sqrt{\left(\frac{M_{OT,y}}{mkM_{CE,y}}\right)^2 + \left(\frac{M_{OT,x}}{mkM_{CE,x}}\right)^2} \leq 1.0$$

▪

B.6.3.4 RESULTS AND CONCLUSIONS

The results of these studies are summarized in Table B-22 and Table B-23 below. These studies used linear static procedure with fixed base assumptions for loading in the East-West direction. Further discussion of East-West loading can be found in section B.3.6.3. Conditions were evaluated with and without the contributions of slabs and/or perpendicular frames to resist overturning in addition to the bidirectional load cases described in Section B.6.3.1.

Table B-22 Retrofit footing uplift acceptance ratios for comparison of bi-directional load of effects for East-West Direction

Retrofit Footing Uplift Acceptance Ratios (Linear Static Procedure, E-W, Fixed Base)					
Foundation Type	**ASCE/SEI 7-10(2)**	**ASCE/SEI 41-17(1)(2)**	**ASCE/SEI 41-17 (w/ slab)(2)**	**ASCE/SEI 41-17 (100%+30%) (1)**	**ASCE/SEI 41-17 (100%+30% w/ grade beam)**
Rectangular	1.34	5.93	2.03	7.21	4.27
L-Shaped	0.86	1.88	1.07	2.00	1.68

(1) This excludes the contribution of perpendicular frames to resist overturning
(2) This does not include bi-directional load affects.

Table B-23 Retrofit footing compression acceptance ratios for comparison of bi-directional load of effects for East-West Direction

Retrofit Footing Compression Acceptance Ratios (Linear Static Procedure, E-W, Fixed Base)					
Foundation Type	**ASCE/SEI 7-10[2]**	**ASCE/SEI 41-17(1)(2)**	**ASCE/SEI 41-17 (w/ slab)(2)**	**ASCE/SEI 41-17 (100%+30%)(1)**	**ASCE/SEI 41-17 (100%+30% w/ grade beam)**
Rectangular	1.16	2.43	1.17	2.57	2.06
L-Shaped	0.44	1.76	0.86	1.85	1.51

(1) This excludes the contribution of perpendicular frames to resist overturning
(2) This does not include bi-directional load affects.

These results indicate the following:

- ASCE/SEI 41-17 provisions result in larger footing sizes (or higher acceptance ratios) compared with ASCE/SEI 7-10. As can be seen in Table B-22 and Table B-23, the Acceptance Ratios for the ASCE/SEI 41-17 case without the slab or adjacent frames is higher than that of the ASCE/SEI 7-10 results. Some of these results are also repeated and further explored in Table B-9.

- Uplift at the corner column condition, where seismic axial loads contribute to uplift, is significant with linear ASCE 41 procedures. The effect of perpendicular framing to resist uplift should be included to develop a reasonable retrofit foundation design. This is evident in the comparison between the ASCE/SEI 41 results that exclude versus include the slab in Table B-22. Further, from Table B-22, it can be seen that the L-shaped footing acceptance ratios are lower than those of the rectangular footings, indicating that there are restorative effects from including the three-dimensional structure.

- Guidance should be provided for the ASCE/SEI 41-17 user to assist with M_{CE} determination for different foundation configurations modeled as fixed-base (statics with a soil bearing compression block due to rocking).

B.6.4 Recommended Changes

The recommended changes include adding new provisions for determining foundation overturning capacity where the footing is non-rectangular and for bi-directional moments on the footing. Currently in the standard, overturning acceptance is addressed simplistically only for unidirectional moment for a rectangular or I-shaped footing. In addition, a new methodology is outlined in the proposed commentary for evaluating foundations where the footing is required to resist overturning moments simultaneously about the two horizontal principal axes of the footing. This methodology is applicable to isolated footings of any plan geometry.

B.7 Acceptance Criteria for the Structural Footing (Proposed ASCE/SEI 41-23 Section 8.4.4.1.1.2.2)

B.7.1 Motivation

There is currently no specific requirement or acceptance criteria for checking the structural footing in Chapter 8. Evaluation of the concrete foundation structural component is specified in the concrete chapter (§10.12.3) where demands to the foundation are treated as force-controlled. Case studies have shown that this requirement could be overly conservative and as a result, leaves the possibility that the adequacy and strength of the footings may not be checked. Specific requirements have been introduced specifying the magnitude and application of the soil pressures as loads to the footing.

B.7.3 Technical Studies

See discussion and results of Hypothesis 1 and 2.

B.7.4 Recommended Changes

The recommended change clarifies what the acceptance criteria is for a structural footing. The proposed language points to evaluation per the material chapters and defines the appropriate demands. Alternatively, it allows for evaluation of the footing as force controlled for the soil pressure distribution under specific conditions.

B.8 Soil Stiffness for Shallow Foundations (Proposed ASCE/SEI 41-23 Sections 8.4.5.1, 8.4.5.2.1.2)

B.8.1 Motivation

ASCE/SEI 41-17 has three methods (Methods 1, 2 and 3) for determining and modelling soil spring stiffness as well as lower and upper bounding requirements. The goal is to simplify modelling approaches and eliminate redundant or unused options.

B.8.3 Technical Studies

B.8.3.1 COMPARISON OF METHODOLOGIES FOR DERIVING SOIL SPRINGS

The following soil spring methodologies are investigated. Studies conducted under Hypothesis 2 indicated that ASCE/SEI 41-17 Method 2 was not practical nor necessary since it must be calibrated to Method 1.

- Method 1 – ASCE/SEI 41-17 Figure 8-2
- K_{50} Stiffness (see Section B.8.3.1.2)
- Method 3 – ASCE/SEI 41-17 Equation 8-11

B.8.3.1.1 Method 1 Soil Stiffness Derivation

The existing, new, or retrofit footing is treated as a rigid body for the Method 1 spring stiffness derivations. Method 1 uses uncoupled moment and axial springs to model rigid foundations such that moment and shear behaviors are independent of axial load. Shear springs may also be used. However, in this case, and for all methods in this investigation, lateral movement is restrained within the analysis model. A graphical representation of the Method 1 springs is shown in Figure B-36. The parameters described in this figure, K_{yy} and K_z, correspond to the uncoupled spring stiffness coefficients in overturning (rotation) and in the vertical direction, respectively per ASCE/SEI 41-17 Figure 8-2.

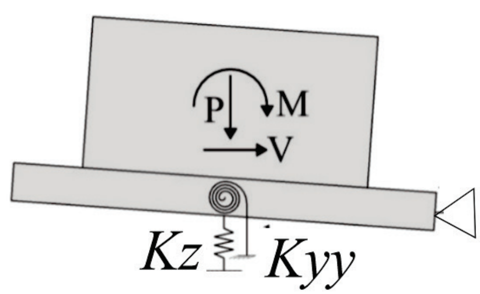


Figure B-36 Method 1 foundation springs (adapted from FEMA P-2006).

The footing geometric parameters are shown in Figure B-37.

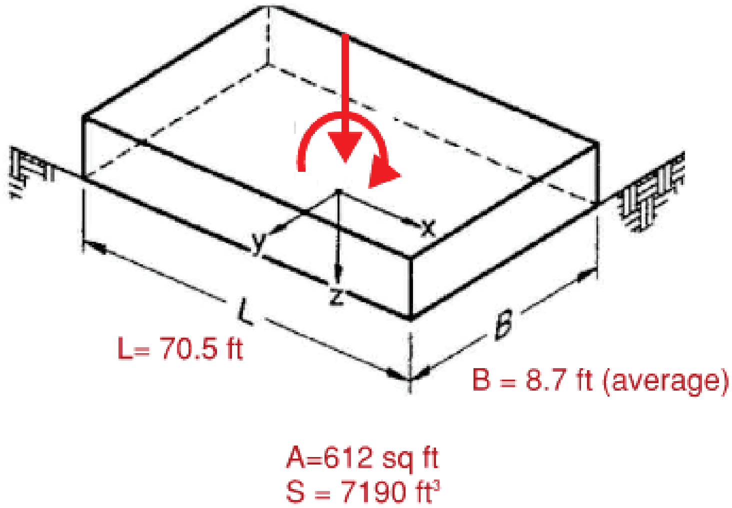


Figure B-37 Retrofit footing dimensions for soil spring calculations.

The shear modulus is calculated utilizing ASCE/SEI 41-17 Equation 8-5 with the standard penetration test blow count provided by the geotechnical engineer. The axial spring, K_z, and rotational spring, K_{yy}, are calculated using the equations in ASCE/SEI 41-17 Figure 8-2. The example calculations below are for the retrofit footing; a similar procedure is done for the individual existing footings supporting existing columns throughout the rest of the building. The stiffness coefficients used in ETABS are summarized in Table B-24. Upper and lower bound stiffness is defined as twice the expected stiffness and one-half the calculated stiffness respectively in accordance with ASCE/SEI 41-17 § 8.4.2.

The initial shear modulus was determined to be:

$G_0 =\sim 120\, p_a (N_{60})^{0.77}$

Where:

$N_{60} = 25.0$ (per the geotechnical report)

$p_a = 2.12$ ksf (atmospheric pressure)

$G_0 =\sim 120\, (2.12)(25)^{0.77} = 3028$ ksf

Then, the shear modulus can be determined using ASCE/SEI 41-17 Table 8-2:

$G/G_0 = 0.3$ (Site Class D, $S_{xs}/2.5 = 0.6$)

$G = 0.3G_0$

$= 0.3\,(3028 \text{ ksf}) = 908$ ksf

The spring stiffness coefficients can then be determined from ASCE/SEI 41-17 Figure 8-2:

$K_z = GB/(1\text{-}v)[1.55(L/B)^{0.75} + 0.8]$

Where:

v = 0.35 (per the geotechnical report)

L = 70.5 ft

B = 8.7 ft (average)

$K_z = (908\ ksf)(8.7\ ft)/(1\text{-}0.36)[1.55(70.5/8.7)^{0.75} + 0.8]$

$= 100167\ kip/ft = 8347\ kip/in$

And

$K_{yy} = GB^3/(1\text{-}v)[0.47(L/B)^{2.4} + 0.034]$

$= (908\ ksf)(8.7\ ft)^3/(1\text{-}0.36)[0.47(70.5/8.7)^{2.4} + 0.034]$

$= 786{,}390{,}000\ kip\text{-}in/radian$

Table B-24 Method 1 Soil Spring Stiffnesses for ETABs for the Retrofit and Existing Footings

	Method 1 Soil Spring Stiffnesses (Kz in kip/in and Kyy in kip-in/radian)			
		Lower Bound (0.5x)	**Expected**	**Upper Bound (2x)**
Retrofit	K_z	4174	8347	16694
	K_{yy}	393,174,556	786,349,112	1,572,698,224
(E) Ftg	K_z	1437	2874	5747
	K_{yy}	4,892,313	9,784,627	19,569,254

B.8.3.1.2 K_{50} Soil Stiffness Derivation

K_{50} boundary rotation stiffness assumes that 50% of the moment capacity is mobilized and accounts for non-service level actions and displacements (EQ actions) and includes gapping between soil and footing. The calculation discussed in this section is based on the findings provided in *"Validation of ASCE/SEI 41-13 Modeling Parameters and Acceptance Criteria for Rocking Shallow Foundations"* (by Hakhamaneshi et al. dated May 2016).

Figure B-38 shows the derivation of the rocking moment capacity used to determine stiffness parameters.

$$M_{c\text{-}foot} = \frac{PL}{2}\left(1 - \frac{q}{q_c}\right)$$

from the two methods would be more similar. Deng et al. (2014) observed that the stiffness, K_{50}, was approximately proportional to the rocking moment capacity when 50% of the capacity is mobilized, and that the ratio K_{50}/M_{c-foot} ranged from 230 to 460. For design of rectangular rocking footings, they suggested that $K_{50} \approx 300 M_{c\text{-}foot}$. Figure 3b plots the measured K_{50} and the $M_{c\text{-}foot}$ values from the data generated by Johnson (2012) and the MAHS test series. It is noted that about 68% of the data points (mean plus/minus one standard deviation) lie between the $K_{50}/M_{c\text{-}foot}$ ratios of 190 and 550. As it will be shown later, I-shaped footings with a larger MAR were found to have larger $K_{50}/M_{c\text{-}foot}$ ratios than other footings.

Figure B-38 Rocking moment capacity equation.

The test data within this report determined that the rotation stiffness of I-shaped footings largely fell within $K_{50}/M_{c,foot}$ ratios of 190 and 550 depending on the missing area ratio (MAR). Based on the configuration of the retrofit footing investigated within this study, $K_{50} = 300M_{C,foot}$ is used for lower bound stiffness and $K_{50} = 550M_{C,foot}$ for upper bound stiffness values. The correlation between rocking moment capacity and stiffness based on testing is shown in Figure B-39.

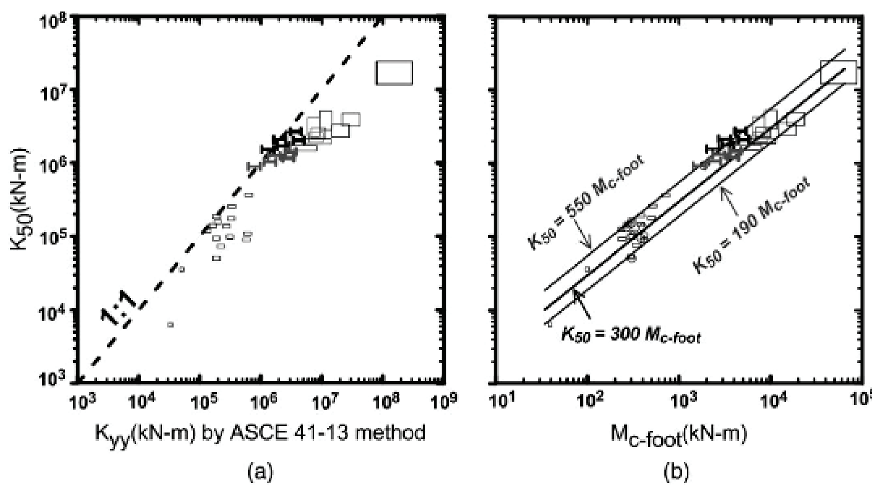


Figure 3. (a) Measured K_{50} vs. ASCE 41-13 method, (b) measured K_{50} vs. measured $M_{c\text{-}foot}$.

Figure B-39 Rocking moment to stiffness correlation.

Table B-25 shows the calculations for the derivation of both upper and lower bound K_{50} values used in the proceeding analyses. The expected q_{CE} is used for the stiffness calculations.

Table B-25 Lower and Upper Bound K_{50} Soil Spring Derivations

	Lower-Bound		Upper Bound	
	Retrofit	**(E) Footing**	**Retrofit**	**(E) Footing**
P_{UD} (kips)	1,440	680	1,440	605
q_{CE} (ksf)	10.5	10.5	10.5	10.5
A_c/A	0.22	0.59	0.22	0.52
$M_{c,foot}$ (k-ft)	39,385	1,473	39,385	1,516
$K_{50} = 300M_{c,foot}$ (kip-ft/radian)	$11.8\text{x}10^6$	442,000	$21.7\text{ x}10^6$	834,000
K_{yy} (kip-in/radian)	$142\text{ x}10^6$	$5.3\text{ x}10^6$	$260\text{ x}10^6$	$10\text{x}10^6$
K_z (kip-in)	4,174	1,437	4,174	1,437

B.8.3.1.3 Method 3 Soil Stiffness Derivation

Method 3 soil springs utilize decoupled Winkler springs. Method 3 diverges from Method 2 because it is intended for shallow foundations not rigid relative to the soil with uniform springs beneath a footing. The stiffness derivation in accordance with ASCE/SEI 41-17 Equation 8-11 is shown in the calculations below. Note that this stiffness calculation requires the footing width, B_f, which is not defined for a mat foundation.

The unit subgrade spring coefficient was determined:

$k_{sv} = 1.3G/[B_f(1\text{-}v)]$ (ASCE/SEI 41-17 Equation 8-11)

Where:

G = 908 ksf (calculated in Section B.8.3.1.1)

v = 0.35 ksf (Poisson's Ratio)

B_f = 8.7 ft (average)

$k_{sv} = 1.3(908\text{ ksf})/[(8.7\text{ ft})(1\text{-}0.35)]$

= 209 kips/ cubic ft

Then, the stiffness per spring can be determined:

$k = k_{sv} B_f l_i$

where

l_i = 2.94 ft (distance between springs)

k = *(209 kips/cubic foot)(8.7 ft)(2.94 ft)*

= *5337 k/ft*

B.8.3.1.4 Stiffness Comparison

Figure B-40 provides a comparison of the rotational stiffness values based on the derivations summarized in the previous sections. For comparison purposes, the rotations of 0 and 0.1 radians were selected (x-axis). The Method 1 moments at the selected rotations are determined by using the rotational spring stiffness values, K_{yy}, in Table B-24 (as the moment is the product of the K_{yy} and the rotation). The K_{50} moments are the product of the retrofit condition K_{50} stiffnesses in Table B-25 and the selected rotations. As previously described, Method 3 Equation 8-11 provides a translational stiffness, which is applied over point springs along the footing. To compare with Method 1 and K_{50} values, the resulting rotational stiffness was calculated based on the overturning moment and rotation measured at the ends of the shear wall.

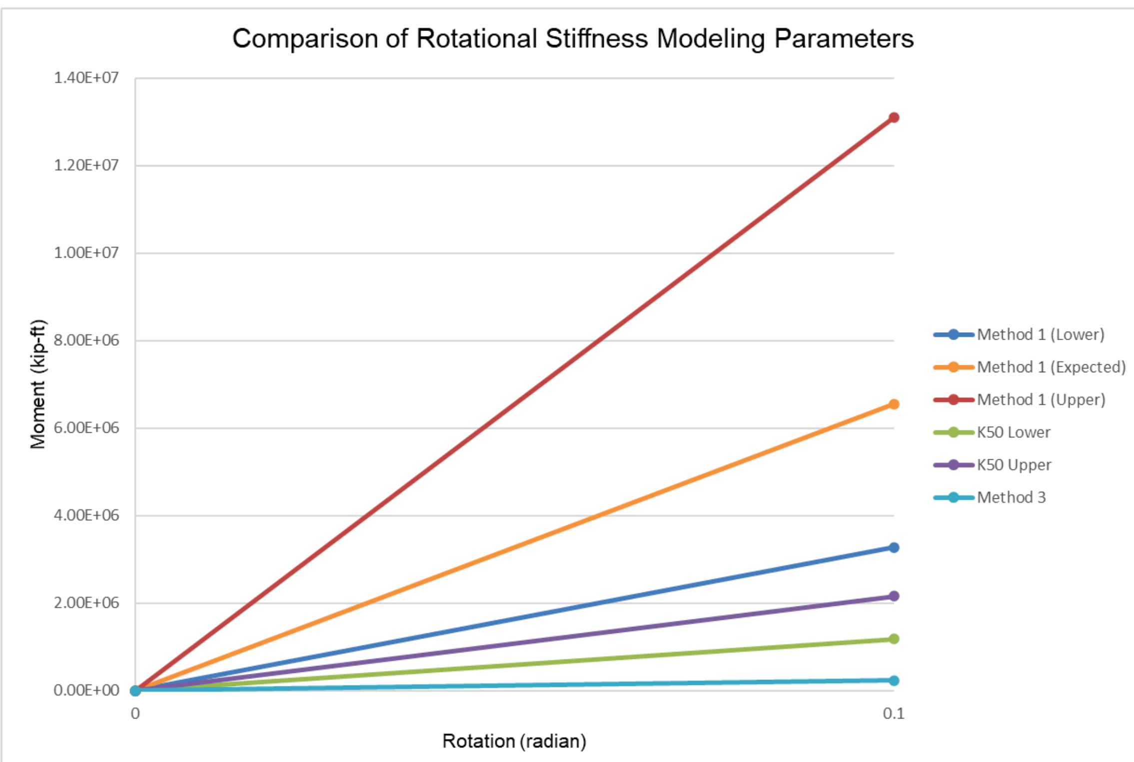


Figure B-40 Comparison of rotational stiffness modeling parameters.

B.8.3.2 MODELING PARAMETERS AND ACCEPTANCE CRITERIA FOR COMPARISON OF METHODOLOGIES FOR DERIVING SOIL SPRINGS

B.8.3.2.1 Method 1 – Modeling Parameters and Acceptance Criteria

For the linear application of Method 1, the springs are defined as linear with the stiffness calculated as discussed above. The capacity of the soil is then evaluated in accordance with ASCE 41-17 § 8.4.2.3.2.2. The upper-bound capacity is permitted for compression in accordance with ASCE 41-17 § 8.4.2.3.2 and *m*-factors are applied as specified in ASCE 41-17 Table 8-3.

For an NSP analysis, ASCE 41-17 § 8.4.2.3.3 specifies the modeling parameters for the soil springs and references Figure 8-4 and Table 8-4, which are used to define the springs shown in Figure B-41. The moment capacity, M_{CE}, is calculated in accordance with ASCE 41-17 Equation 8-10 and the expected soil bearing capacity is utilized for axial compression actions. Nonlinear acceptance criteria is based on overall footing rotation at the target displacement as specified in ASCE 41-17 Table 8-4.

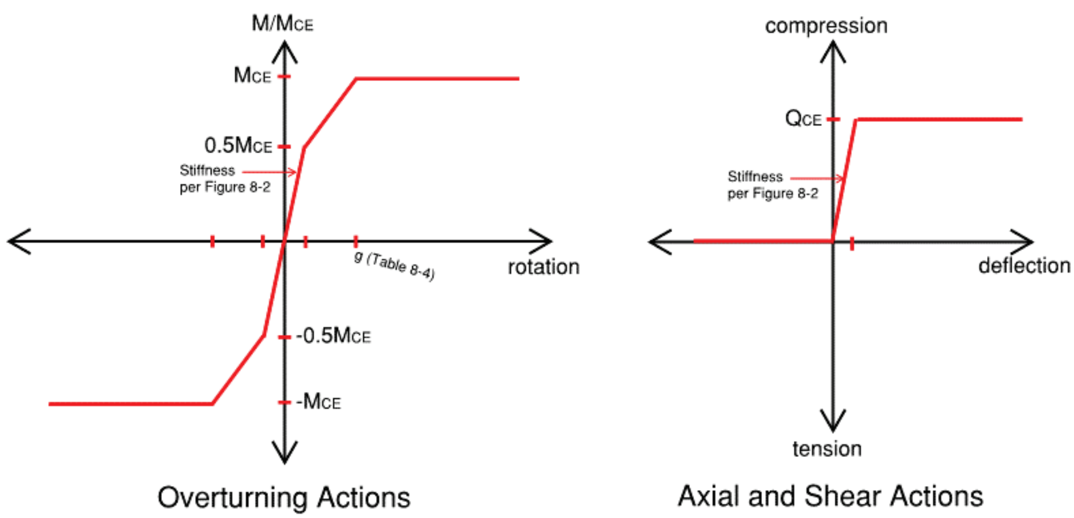


Figure B-41 Method 1 NSP modeling parameters.

B.8.3.2.2 K50 Stiffness – Modeling Parameters and Acceptance Criteria

K_{50} boundary rotation stiffness is applied in the models similarly to Method 1. Acceptance criteria provided in ASCE/SEI 41-17 Table 8-3 is used to evaluate the footing acceptance ratio similar to the Method 1 approach.

B.8.3.2.3 Method 3 Stiffness – Modeling Parameters and Acceptance Criteria

The capacity of the soil springs for Method 3 is noted in ASCE/SEI 41-17 § 8.4.2.5.2 as equal to the expected bearing capacity of the soil in compression and zero in tension. A representative Method 3 soil spring is shown in Figure B-42. The acceptance criteria is based on the rotation limits of ASCE/SEI 41-17 Table 8-4. The rotation modeling parameters noted in ASCE/SEI 41-17 Table 8-4 are not utilized in Method 3, since the springs are for axial actions. The rotation is dependent on the axial soil stiffness and the rigidity of the footing.

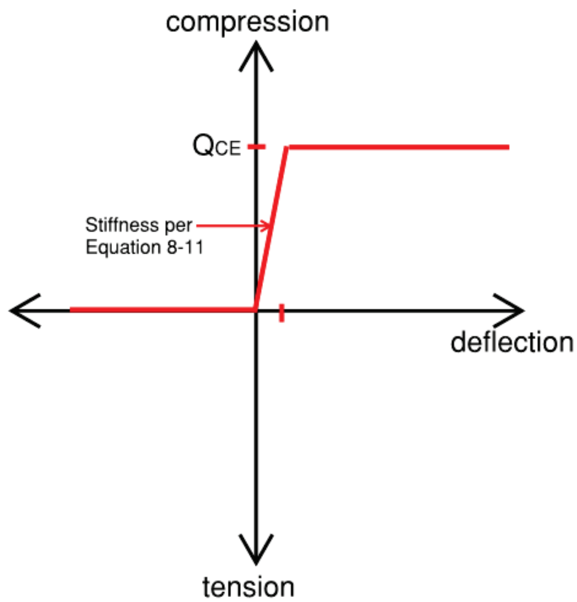


Figure B-42 Method 3 NSP modeling parameters.

The nonlinear static analysis results are discussed in the following sections and compared against the linear analysis.

B.8.3.3 EFFECTS ON SUPERSTRUCTURE AND FOUNDATION METHODOLOGIES FOR DERIVING SOIL SPRINGS

This section evaluates the effect of lower or upper-bound soil stiffness on soil bearing due to overturning and forces in the superstructure. The following analysis models are analyzed with the ASCE/SEI 7-10 designed retrofit footing. The Case 1 and Case 2 models are analyzed for both lower and upper-bound stiffness. Upper-bound bearing strength capacity is used for all cases.

- Case 1: LSP – Fixed Base Condition
- Case 2: LSP – Flexible Base Condition (Method 1)
- Case 3: LSP – Flexible Base Condition (K_{50} Stiffness)
- Case 4: NSP – Flexible Base Condition (Method 3)

B.8.3.3.1 Case 1: LSP – Fixed Base Condition

The seismic base shear based on the same site-specific design criteria used in Hypothesis 1, which corresponds to an S_{XS} of 1.5 and a base shear of 1.32 times the seismic weight of the building (7,200 kips). This is also true for all other LSP models contained within this hypothesis.

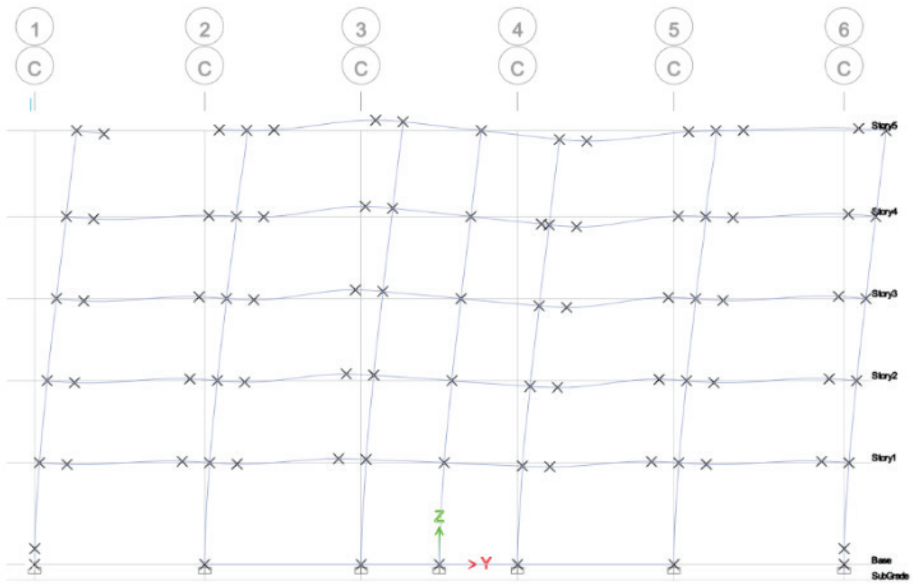


Figure B-43 LSP analysis model with fixed base.

The retrofit footing was then evaluated for bearing pressure due to overturning using ASCE/SEI 41-17 Equation 8-10 as shown in the calculations in Section B.4.2.1 and summarized below.

The expected moment capacity is calculated:

$M_{CE} = 0.5(L_f P_{UD})(1-q/q_c)$ (ASCE/SEI 41-17 Eq. 8-10)

$= 50{,}964$ *kip ft*

Then, by comparing this capacity to the demand per the ETABS model, the acceptance ratio can be identified:

$M_{base} = 269{,}427$ kip ft

Required $m = M_{base} / M_{CE}$ = (269,427 kip ft) / (50964 kip ft) = 5.3

Allowable m = 4 (ASCE/SEI 41-17 Section 8.4.2.3.2.1)

Acceptance Ratio = Required m / Allowable m = 5.3/4 = 1.32

The overturning moment capacity is dependent on the expected vertical load P_{UD}. Further discussion on the calculation of P_{UD} is provided in a Section B.5. For this and subsequent calculations, P_{UD} is equal to the unfactored, expected vertical load including the self-weight of the footing.

These fixed base results are compared against the ASCE/SEI 7-10 allowable bearing pressure calculation. For the purposes of this evaluation, the site-specific seismic S_{DS} of 1g is used. The redundancy factor, ρ, is taken as 1.0. The base shear is calculated including the R-factor for a special concrete shear wall (R=6). ASD load cases are utilized to evaluate the allowable bearing capacity for comparison.

The acceptance ratio for the LSP Analysis Results with Fixed base using ASCE/SEI 7-10:

$q_{max} = 2P_u/(3\ B_f\ e')$ (footing pressure with e>L/6)

where

B_f = 8.7 ft (average) (footing width per Figure B-10)

L_f = 70.5 ft (footing length per Figure B-10)

M_u = 18,365 kip ft (ASD D+L Load Case from ETABS with 25% reduction in LL)

P_u = 997 kips (Load from ETABS, 0.6D+0.7E)

$e' = L_f/2 - e$

where

$L_f/6$ = 11.8 ft

$e = M_u / P_u$ = 18.4 ft > L/6

$e' = L_f/2 - e$ = (70.5 ft)/2 – 18.4 ft = 16.85 ft

q_{max} = 2(997 kips)/(3 (8.7 ft)(16.85 ft))

= 4.55 ksf

Acceptance ratio = q_{max} / q_{allow}

= 4.55 ksf / 4.66 ksf

= 0.98

When evaluated with ASCE/SEI 41-17, the footing is not adequate with an overturning soil bearing acceptance ratio of 1.32. The footing is acceptable based on an ASCE/SEI 7-10 analysis with a bearing pressure acceptance ratio of 0.98.

B.8.3.3.2 Case 2: LSP – Flexible Base Condition (Method 1)

This analysis model includes the addition of a rigid retrofit footing and Method 1 linear soil springs. A single rotational and axial spring is assigned directly below the shear wall. Method 1 rotational and axial springs are also added beneath each existing footing at each column. The deflected shape under dead load and earthquake load is shown in Figure B-44.

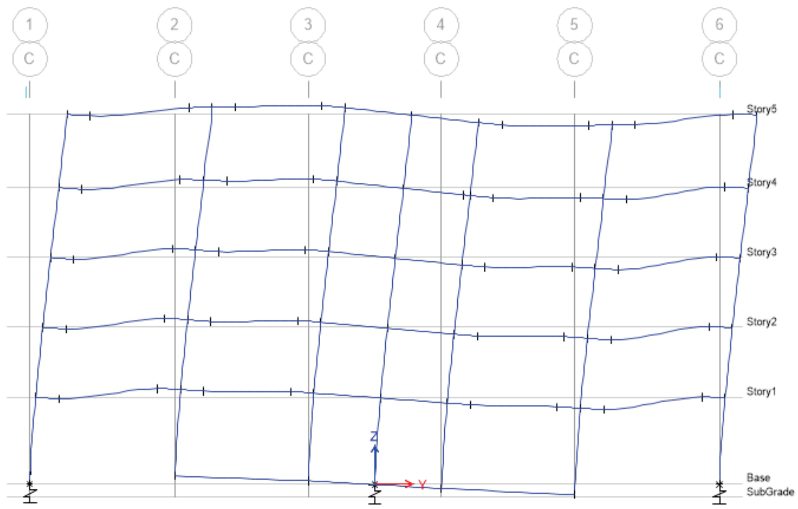


Figure B-44 LSP analysis model with Method 1 soil springs.

The moment demand at the base of the footing is the determined from the rotational soil spring. Resulting footing acceptance ratios are shown in below for lower bound and for upper bound stiffness. In accordance with ASCE/SEI 41-17 § 8.4.2, the lower bound stiffness is calculated as half of the expected value and the upper bound stiffness is calculated as twice the expected value.

ASCE 41-17 with footing retrofit, check overturning compression				
P_{UD} =	1586	kips	(expected load w/ footing DL)	
q =	2.59	ksf		
qc =	21.0	ksf, upper-bound in accordance with 8.4.2.3.2		
A_f =	612	ft^2		
L_f =	70.5	ft		
$A_c = P_{UD}/q_c$=	75.5	ft^2		
b =	8.7	ft		
$L_c = A_c/b$ =	8.7	ft		
b/L_c =	1.00			
A_c/A_f =	0.12			
MCE =	49,002	k-ft		
Mbase =	177,978	k-ft (ETABS)		(corresponding moment at spring)
required m =	**3.6**			
allowable m =	6.0	Section 8.4.2.3.2.1		
Acceptance ratio =	0.61			

Figure B-45 LSP analysis overturning check for Method 1 flexible base (lower bound).

ASCE 41-17 with footing retrofit, check overturning compression				
P_{UD} =	1611	kips	(expected load w/ footing DL)	
q =	2.63	ksf		
qc =	21.0	ksf, upper-bound in accordance with 8.4.2.3.2		
A_f =	612	ft^2		
L_f =	70.5	ft		
$A_c = P_{UD}/q_c$=	76.7	ft^2		
b =	8.7	ft		
$L_c = A_c/b$ =	8.8	ft		
b/L_c =	0.99			
A_c/A_f =	0.13			
MCE =	49,672	k-ft		
Mbase =	224,538	k-ft (ETABS)		(corresponding moment at spring)
required m =	**4.5**			
allowable m =	6.0	Section 8.4.2.3.2.1		
Acceptance ratio =	0.75			

Figure B-46 LSP analysis overturning check for Method 1 flexible base (lower bound).

B.8.3.3.3 Case 3: LSP – Flexible Base Condition (K_{50} Stiffness)

This case is modeled as explained in Case 2 above with a single spring at the base of the shear wall with K_{50} rotational spring stiffness. The $300M_{c,foot}$ value is the expected rotational stiffness for a rectangular footing and $550Mc_{,foot}$ is applicable to an "I" shaped footing and is provided for comparison.

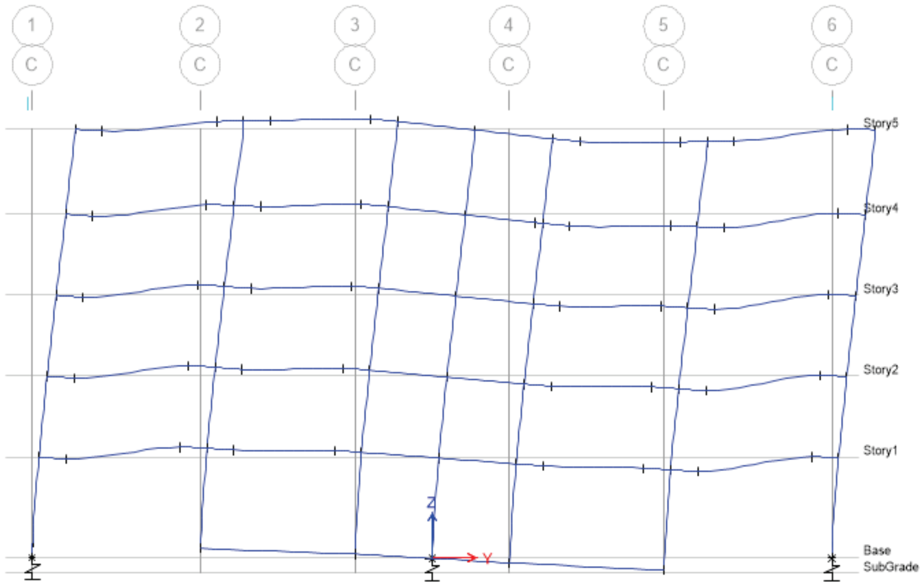


Figure B-47 LSP analysis model with K_{50} soil springs (lower and upper bound).

Resulting footing acceptance ratio determinations are shown below for lower bound (Figure B-48) and upper bound stiffness (Figure B-49).

ASCE 41-17 with footing retrofit, check overturning compression			
Retrofit Footing			
P_{UD} =	1586	kips	(expected load w/ footing DL)
q =	2.59	ksf	
q_c =	21.0	ksf, upper-bound in accordance with 8.4.2.3.2	
A_f =	612	ft^2	
L_f =	70.5	ft	
$A_c = P_{UD}/q_c$=	75.5	ft^2	
b =	8.7	ft	
$L_c = A_c/b$ =	8.7	ft	
b/L_c =	1.00		
A_c/A_f =	0.12		
M_{CE} =	49,003	k-ft	
M_{base} =	111,809	k-ft (ETABS)	(corresponding moment at spring)
required m =	**2.3**		
allowable m =	6.0	Section 8.4.2.3.2.1	
Acceptance ratio =	0.38		

Figure B-48 LSP analysis results with K_{50} flexible base (lower bound *300$M_{c,foot}$*).

ASCE 41-17 with footing retrofit, check overturning compression			
Retrofit Footing			
P_{UD} =	1586	kips	(expected load w/ footing DL)
q =	2.59	ksf	
q_c =	21.0	ksf, upper-bound in accordance with 8.4.2.3.2	
A_f =	612	ft^2	
L_f =	70.5	ft	
$A_c = P_{UD}/q_c$=	75.5	ft^2	
b =	8.7	ft	
$L_c = A_c/b$ =	8.7	ft	
b/L_c =	1.00		
A_c/A_f =	0.12		
M_{CE} =	49,002	k-ft	
M_{base} =	146,138	k-ft (ETABS)	(corresponding moment at spring)
required m =	**3.0**		
allowable m =	6.0	Section 8.4.2.3.2.1	
Acceptance ratio =	0.50		

Figure B-49 LSP analysis results with K_{50} flexible base (upper bound $500M_{c,foot}$).

B.8.3.3.4 Case 4: NSP – Flexible Base Condition (Method 3)

The Method 3 soil springs (Section B.8.3.2.3) are incorporated into the model and the same footing is assessed for rotation acceptance criteria. The Method 3 pushover analysis is shown in Figure B-50 and Figure B-74. Nonlinear hinge definitions for structural components are not outlined herein as our focus is on ASCE/SEI 41-17 Chapter 8.

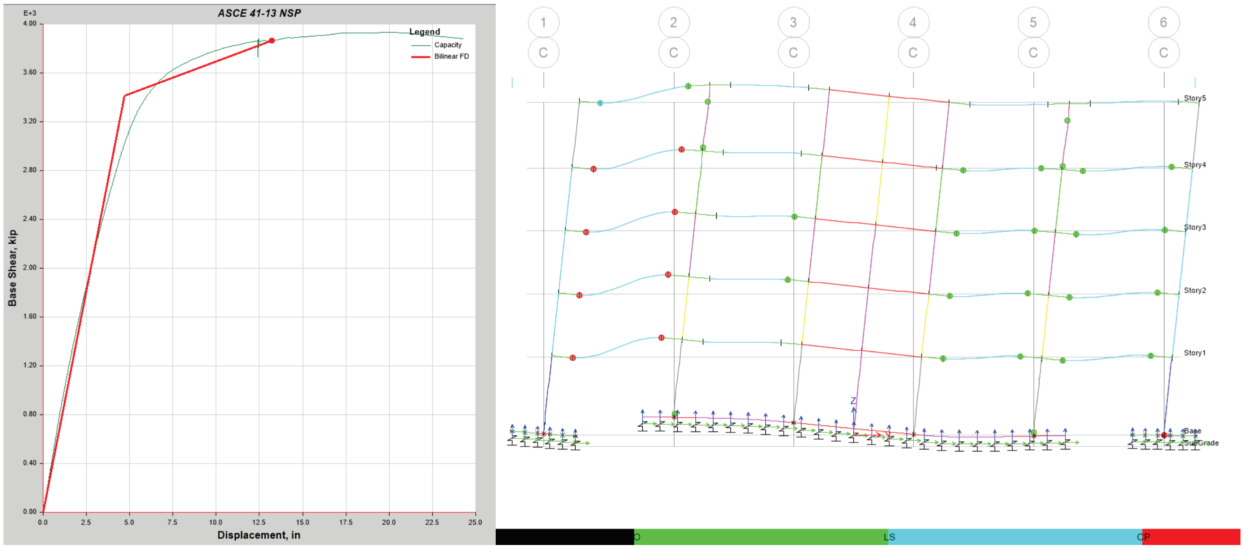


Figure B-50 ASCE/SEI 41-17 Nonlinear static pushover, Method 3 flexible base model.

The pushover curve and deflected shape at the target displacement are shown in Figure B-50. The effective fundamental period of this model is 0.70 seconds (calculated from the pushover curve) and the target displacement is 12.8 inches.

The rotation at the base of the shear wall at the target displacement is compared to the acceptance criteria for footing rotation from ASCE/SEI 41-17 Table 8-4. The acceptance criteria calculations are included below.

P_{UD}=	1769	kips
q_C=	10.5	ksf
A_C=	168	ft2
A_f=	613	ft2
A_C/Af=	0.27	
L_c =A_c/b =	19.41	ft
b/Lc =	0.45	
allowable rotation =	0.0224	radians, 41-17 Table 8-4
rotation at target =	0.0176	radians
acceptance ratio =	0.78	

Figure B-51 ASCE/SEI 41-17 Method 3, footing acceptance criteria.

B.8.3.4 RESULTS

The resulting effects of the soil modeling assumptions on the superstructure were captured for each analysis case. Acceptance ratios were calculated for columns, shear walls, and slabs as shown in Table B-26 through Table B-31. The LSP acceptance ratios compare the analysis demand to the deformation-controlled capacity in accordance with ASCE/SEI 41-17 Equation 7-36. None of the superstructure elements shown in the tables below have been evaluated as force-controlled. The acceptance ratios for the NSP analyses compare the hinge rotation to the acceptance criteria for Collapse Prevention as specified in ASCE/SEI 41-17 Chapter 10. If there is no inelastic rotation in the hinge at the target displacement, the acceptance criteria is listed as 0.00. Maximum values for each action and analysis have been highlighted.

Similar to the soil foundation acceptance ratios, the superstructure results indicate a nominal difference in forces in the superstructure between lower and upper-bound stiffness for each flexible foundation analysis. The K_{50} analysis procedures have higher acceptance ratios than the Method 1 analyses, because of the increased flexibility in the soil springs.

Table B-26 Existing Interior Column - Moment Acceptance Ratios (CP Limit State)

Existing Interior Columns – Moment Acceptance Ratios by Story						
Analysis Model	**1st Story**	**2nd Story**	**3rd Story**	**4th Story**	**5th Story**	**Outcome**
LSP - Fixed Base	0.77	0.62	0.26	0.41	0.60	OK
LSP - Method 1 Lower Bound (Rigid Footing)	1.22	0.86	0.39	0.60	0.82	NG
LSP - Method 1 Upper Bound (Rigid Footing)	1.06	0.76	0.33	0.51	0.75	NG
LSP - K_{50} 300$M_{c,foot}$ (Rigid Footing)	1.57	0.76	0.50	0.51	0.77	NG
LSP - K_{50} 550$M_{c,foot}$ (Rigid Footing)	1.52	0.73	0.45	0.45	0.70	NG
NSP - Method 3	0.54	0.00	0.15	0.35	0.66	OK

Note: a DCR equal to 0.00 indicates no inelastic behavior occurs at the target displacement

Table B-27 Existing Interior Column - Shear Acceptance Ratios (CP Limit State) (For Nonlinear Cases Acceptance Ratio is the Same as the Moment Acceptance Ratio)

Existing Interior Columns – Shear Acceptance Ratios by Story						
Analysis Model	**1st Story**	**2nd Story**	**3rd Story**	**4th Story**	**5th Story**	**Outcome**
LSP - Fixed Base	0.34	0.45	0.20	0.34	0.49	OK
LSP - Method 1 Lower Bound (Rigid Footing)	0.64	0.72	0.31	0.51	0.67	OK
LSP - Method 1 Upper Bound (Rigid Footing)	0.51	0.59	0.26	0.43	0.62	OK
LSP - K_{50} $300M_{c,foot}$ (Rigid Footing)	0.89	0.80	0.41	0.43	0.63	OK
LSP - K_{50} $550M_{c,foot}$ (Rigid Footing)	0.83	0.68	0.36	0.38	0.57	OK
NSP - Method 3	See moment acceptance ratios for nonlinear cases above					

Table B-28 Retrofit Shear Wall - Shear Acceptance Ratios (CP Limit State)

Retrofit Shear Walls – Shear Acceptance Ratios by Story						
Analysis Model	**1st Story**	**2nd Story**	**3rd Story**	**4th Story**	**5th Story**	**Outcome**
LSP - Fixed Base	0.85	0.93	0.75	0.05	0.29	OK
LSP - Method 1 Lower Bound (Rigid Footing)	0.53	0.82	0.63	0.05	0.20	OK
LSP - Method 1 Upper Bound (Rigid Footing)	0.74	0.85	0.69	0.05	0.25	OK
LSP - K_{50} $300M_{c,foot}$ (Rigid Footing)	0.14	0.80	0.57	0.05	0.15	OK
LSP - K_{50} $550M_{c,foot}$ (Rigid Footing)	0.28	0.83	0.61	0.04	0.18	OK
NSP - Method 3	0.00*	0.00*	0.00*	0.00*	0.00*	OK

Note: a DCR equal to 0.00 indicates no inelastic behavior occurs at the target displacement

Table B-29 Retrofit Shear Wall - Moment Acceptance Ratios (CP Limit State)

Retrofit Shear Walls – Moment Acceptance Ratios by Story						
Analysis Model	**1st Story**	**2nd Story**	**3rd Story**	**4th Story**	**5th Story**	**Outcome**
LSP - Fixed Base	1.00	0.83	0.55	0.78	0.30	OK
LSP - Method 1 Lower Bound (Rigid Footing)	0.79	0.76	0.46	0.59	0.20	OK
LSP - Method 1 Upper Bound (Rigid Footing)	0.96	0.82	0.51	0.69	0.25	OK
LSP - K_{50} 300$M_{c,foot}$ (Rigid Footing)	0.52	0.71	0.40	0.48	0.14	OK
LSP - K_{50} 550$M_{c,foot}$ (Rigid Footing)	0.64	0.76	0.44	0.56	0.18	OK
NSP - Method 3	0.00*	0.00*	0.00*	0.00*	0.00*	OK

Note: a DCR equal to 0.00 indicates no inelastic behavior occurs at the target displacement

Table B-30 Existing Slab – Flexure Acceptance Ratios (CP Limit State)

Existing Slab – Flexure Acceptance Ratios by Story						
Analysis Model	**1st Story**	**2nd Story**	**3rd Story**	**4th Story**	**5th Story**	**Outcome**
LSP - Fixed Base	0.42	0.59	0.68	0.77	0.49	OK
LSP - Method 1 Lower Bound (Rigid Footing)	1.28	1.38	1.39	1.32	0.75	NG
LSP - Method 1 Upper Bound (Rigid Footing)	0.80	0.92	0.95	0.94	0.51	OK
LSP - K_{50} 300$M_{c,foot}$ (Rigid Footing)	1.83	1.89	1.89	1.77	1.00	NG
LSP - K_{50} 550$M_{c,foot}$ (Rigid Footing)	1.46	1.53	1.54	1.44	0.83	NG
NSP - Method 3	1.33	1.29	1.29	1.17	0.83	NG

Table B-31 Story Drift – Drift Ratio per Story

Story Drift – Ratios by Story					
Analysis Model	**2nd Story**	**3rd Story**	**4th Story**	**5th Story**	**Roof**
LSP - Fixed Base	0.003	0.006	0.007	0.008	0.008
LSP - Method 1 Lower Bound (Rigid Footing)	0.008	0.012	0.013	0.013	0.013
LSP - Method 1 Upper Bound (Rigid Footing)	0.005	0.009	0.010	0.011	0.011
LSP - K_{50} $300M_{c,foot}$ (Rigid Footing)	0.011	0.015	0.016	0.016	0.016
LSP - K_{50} $550M_{c,foot}$ (Rigid Footing)	0.009	0.013	0.014	0.014	0.014
NSP - Method 3	0.012	0.013	0.013	0.013	0.013

Table B-32 and Table B-33 summarize the previous analyses and the foundation acceptance ratios. The results indicate that the ASCE/SEI 41-17 fixed base analysis provides reasonable correlation to the ASCE/SEI 7-10 foundation design. The flexible-base analysis procedures have lower acceptance ratios which is consistent with the reduced force attracted to the shear wall because of flexibility in the supporting foundation as well as higher m-factors permitted for the flexible-base analysis. The difference between acceptance ratios for lower and upper-bound analyses is relatively negligible for this case study.

Table B-32 Summary of Analysis Results for the Retrofit Structure Foundation

Assessment of Soil Bearing due to Overturning Action						
Model	**Initial Fundamental Period (seconds)**	**Effective Fundamental Period (seconds)**	**Target Displacement (inches)**	**Base Shear (kips)(1)**	**Max. Vertical Uplift at Shear wall(2) (in)**	**Max. Vertical Uplift(3) (in)**
LSP - ASCE/SEI 7-10	0.43	-	-	0.17W	-	-
LSP - Fixed Base	0.43	-	-	1.3W	-	-
LSP Method 1 Lower Bound (Rigid Footing)	0.58	-	-	1.3W	0.88	2.26
LSP Method 1 Upper Bound (Rigid Footing)	0.50	-	-	1.3W	0.49	1.12
LSP - K_{50} 300$M_{c,foot}$ (Rigid Footing)	0.65	-	-	1.3W	1.29	3.46
LSP K_{50} 550$M_{c,foot}$ (Rigid Footing)	0.60	-	-	1.3W	1.01	2.64
NSP Method 3	0.70	0.76	12.8	0.54W	4.30	6.30

Note: Analysis includes retrofit footing sized previously in hypothesis 1. Footing was designed to conform with ASCE/SEI 7-10 provisions and is 6-feet wide by 4-feet deep with (30) #11 bars top and bottom and (4) #5 stirrups at 6”oc.

(1) W is the effective seismic weight of the superstructure equal to 7,200 kips.
(2) Maximum vertical uplift taken at face of shear wall
(3) Maximum vertical uplift at any location along the retrofit footing

Table B-33 Summary of Analysis Results for the Retrofit Structure Foundation

Assessment of Soil Bearing due to Overturning Action							
Model	**Allowable Rotation(2) (radians)**	**Rotation at Target - at shear wall(3) (radians)**	**Rotation at Target - at ends of ftg(4) (radians)**	**Rotation at Target - at inflection pts(5) (radians)**	**Rotational stiffness (kip-ft/rad)**	***m*-factor**	**Footing Acceptance Ratio(2,8)**
LSP - ASCE/SEI 7-10	-	-	-	-	-	-	0.98
LSP - Fixed Base	-	-	-	-	-	4.0	1.32
LSP Method 1 Lower Bound (Rigid Footing)	-	-	-	-	32,764,546	6.0	0.61
LSP Method 1 Upper Bound (Rigid Footing)	-	-	-	-	131,058,185	6.0	0.75
LSP - K_{50} 300$M_{c,foot}$ (Rigid Footing)	-	-	-	-	11,815,563	6.0	0.38
LSP K_{50} 550$M_{c,foot}$ (Rigid Footing)	-	-	-	-	21,661,866	6.0	0.50
NSP Method 3	0.022	0.018	0.004	0.008	2,411,500	-	0.78

Note: Analysis includes retrofit footing sized previously in hypothesis 1. Footing was designed to conform with ASCE/SEI 7-10 provisions and is 6-feet wide by 4-feet deep with (30) #11 bars top and bottom and (4) #5 stirrups at 6 inches on center.

(1) W is the effective seismic weight of the superstructure equal to 7,200 kips.
(2) Allowable rotation and footing acceptance ratios calculated using entire footing length and effective footing width, rotation demand taken at end of shear wall. P_{UD} used to calculate footing acceptance ration is expected load including footing dead load.
(3) Rotation at target displacement taken at ends of shear wall.
(4) Rotation at target displacement taken at ends of retrofit footing.
(5) Rotation at target displacement taken at inflection points of footing by using entire length of retrofit footing but measuring the rotation demand between inflection points of the deformed footing (see Section B.10.5.8).
(6) Maximum vertical uplift taken at face of shear wall
(7) Maximum vertical uplift at any location along the retrofit footing
(8) Expected moment capacity used to determine Footing Acceptance Ratios based on upper-bound bearing capacities per 8.4.2.3.2 for LSP models.

B.8.3.5 CONCLUSIONS

The results indicate that the ASCE/SEI 41-17 fixed base analysis provides reasonable correlation to the ASCE/SEI 7-10 foundation design, though the ASCE/SEI 41/-17 fixed base analysis indicated that the footing is not adequate due to an overturning soil bearing acceptance ratio of 1.32 while that of the ASCE/SEI 7-10 analysis was 0.98.

The flexible-base analysis procedures have lower acceptance ratios than the fixed-base and ASCE/SEI 7-10 which is consistent with the reduced force attracted to the shear wall because of flexibility in the supporting foundation as well as higher m-factors permitted for the flexible-base analysis.

Similar to the soil foundation acceptance ratios, the superstructure results indicate a nominal difference in forces in the superstructure between lower and upper-bound stiffness for each flexible foundation analysis. The K_{50} analysis procedures have higher acceptance ratios than the Method 1 analyses, because of the increased flexibility in the soil springs. In general, the K_{50} stiffness models (with gapping) correlate better with nonlinear analysis methods (Method 3). In addition, the upper-bound stiffness does not yield sufficiently different results (superstructure component actions and foundation overturning acceptance ratios) to warrant the additional effort to include in the analysis procedures,

B.8.4 Recommended Changes

The recommended change clarifies how to calculate the soil stiffness for shallow foundations by providing a specific equation that is a function of the shear modulus and footing length and width (specified for mat foundation also) and Poisson's ratio. It also allows for this modulus to be provided by the geotechnical engineer.

B.9. Acceptance Criteria for Isolated Spread Footings with Foundation Interface Modeled as a Flexible Base Footing Not Rigid Relative to Soil (Proposed ASCE/SEI 41-23 Section 8.4.5.2.1.3)

B.9.1 Motivation

ASCE/SEI 41-17 has limited guidance on modeling and assessment of foundations that are flexible relative to the soil. Method 3 only provides provisions for nonlinear analysis and acceptance criteria. The goal is to provide better guidance and provisions for linear flexible foundation modeling and acceptance criteria for the user.

B.9.2 Technical Studies

In addition to evaluating the effect of soil stiffness bounding on foundation design and superstructure performance, the structural foundation components are also evaluated for each model and compared to each other and the ASCE/SEI 7-10 calculations. The goal is to assess different options for how to evaluate foundations using linear procedures that includes flexibility of the foundation itself. Although ASCE/SEI 41-17 specifies that structural foundations be evaluated as force-controlled components, foundation strength is also evaluated as deformation-controlled in this case study for comparison as yielding of the foundation contributes to its flexibility.

B.9.2.1 STRUCTURAL FOUNDATION DESIGN

The structural foundation designed using ASCE/SEI 7-10 as discussed in previous sections is evaluated for the ASCE/SEI 41-17 analysis models. The ASCE/SEI 7-10 foundation design was based on an elastic beam on elastic soil analysis. For the ASCE/SEI 41-17 linear procedures (fixed base, Method 1 and K_{50}), the loads on the retrofit footing are obtained from the analysis models and applied to an elastic beam supported by soil springs to determine the internal forces in the footing. We note that a thorough analysis of the footing would include evaluation at multiple locations (existing footing at face of column, new footing at face of existing footing, etc.) as shown in Figure B-52. For simplicity, the results shown herein are determined at the new footing section at the face of the existing footing.

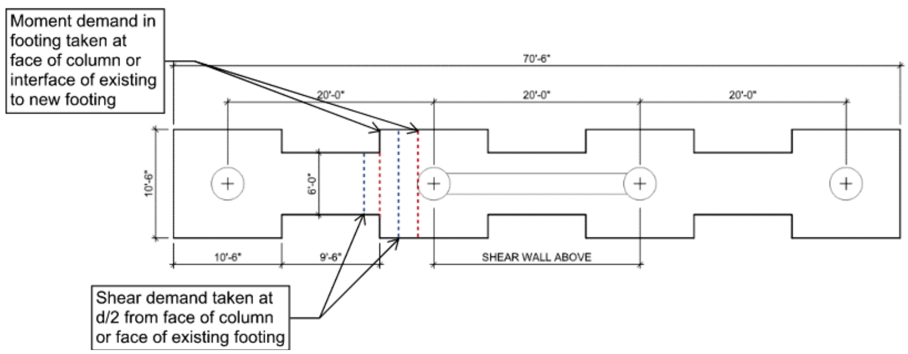


Figure B-52 Critical sections for foundation design.

Two approaches to modeling of the structural foundation and supporting soil were investigated for the linear fixed-base procedure:

1. The structural foundation is modeled as an elastic concrete beam on elastic (tension/compression) foundation springs with vertical stiffness calculated from ASCE/SEI 41-17 Fig. 8-2. Unreduced, pseudo-elastic forces are applied to the foundation as determined from the analysis model and as shown in Figure B-53. See Figure B-44 for deformed shape of footing using this approach.

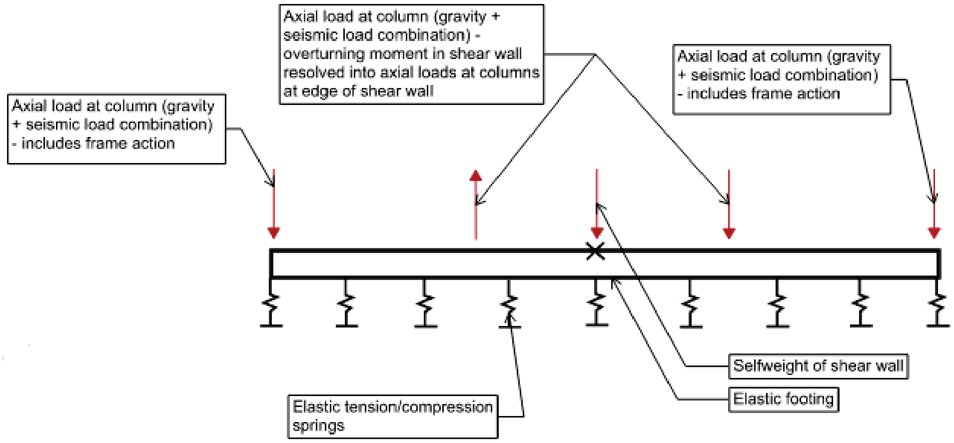


Figure B-53 Load application on elastic foundation model.

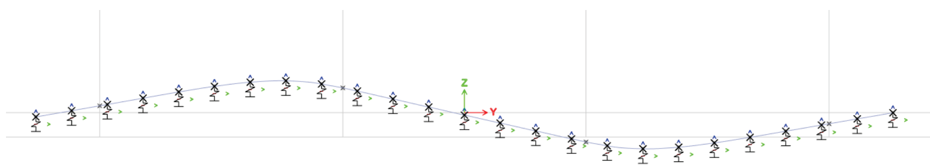


Figure B-54 Elastic tension and compression springs.

2. The structural foundation is modeled as an elastic concrete beam on nonlinear (compression-only) foundation springs with vertical compression stiffness calculated from ASCE/SEI 41-17 Fig. 8-2. This approach is equivalent to the conventional foundation design approach using SAFE with compression-only springs. In order to create a stable analysis model, the pseudo-elastic forces are reduced by an *m*-factor or DCR from the superstructure above. These reduced loads are applied to the foundation similar to the approach shown in Figure B-53. A representative deformed shape of the foundation using this approach is shown in Figure B-55.

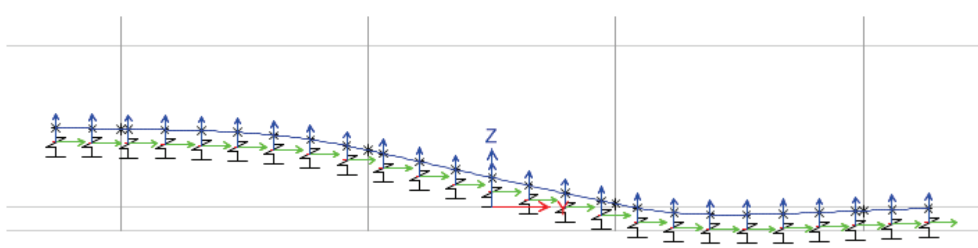


Figure B-55 Elastic compression -only springs.

The compression-only spring analysis was considered for three cases:

1. Reducing pseudo-elastic forces by *m* = 4, which is equivalent to the m-factor for soil bearing pressure compression due to overturning. The use of this m-factor is somewhat arbitrary, but we understand that this approach is often used for similar SAFE-type analyses. For this example, the reduction by *m* did not provide a stable analysis model and at the end of the uplift side of the footing, the displacement was significant (~30 inches). This indicates that the footing is undersized, or the m-factors are too small. This confirms the fixed base procedure may be overly conservative relative to ASCE/SEI 7-10 and the results of the NSP.
2. Reducing pseudo-elastic forces by DCR calculated from superstructure delivering load to the foundation. The DCR was calculated for the shear wall and was determined to be approximately 3. This DCR is less than m = 4 and therefore also provides an unstable analysis model.
3. Reducing pseudo-elastic forces by m = 7.76, which is equal to the ratio of ASCE/SEI 41-17 (1.32 W per Table B-41) to ASCE/SEI 7-10 base shear (0.17W per Table B-41) for comparison between ASCE 7 and ASCE 41. Note that this is not a realistic m or DCR factor and this analysis is only performed to investigate the feasibility of the compression-only analysis approach.

For the fixed-base analysis, the comparisons of the following approaches are evaluated for the elastic beam supported by springs with results shown in Table B-34 and Table B-35 for both lower and upper-bound soil stiffness:

1. Elastic tension/compression springs with concrete foundation analyzed for deformation-controlled actions (expected strength with m-factors from ASCE/SEI 41-17 Chapter 10).
2. Nonlinear, compression-only springs with concrete foundation analyzed as force-controlled (lower bound strength with no additional *m*-factors).

Table B-34 Comparison of Fixed-Base Approaches

Utilizing m-factors for LSP Fixed Base Strength Design					
Model	**Lower-Bound Stiffness**				
	Deflection (in)	**Moment (k-ft)**	**Moment Acceptance Ratio**	**Shear (kip)**	**Shear Acceptance Ratio**
Tens/Comp Spring w/ component m-factor	6.45	62,128	1.89	5,209	1.29
Compression-only Demand/(m = 4)	Unstable				
Compression-only Demand/(m = 7.76)	3.84	12,450	1.52	1,235	0.89

Table B-35 Comparison of Fixed-Base Approaches

Utilizing *m*-factors for LSP Fixed Base Strength Design					
Model	**Upper-Bound Stiffness**				
	Deflection (in)	**Moment (k-ft)**	**Moment Acceptance Ratio**	**Shear (kip)**	**Shear Acceptance Ratio**
Tens/Comp Spring w/ component m-factor	2.49	47,266	1.44	7,000	1.26
Compression-only Demand/(m = 4)	Unstable				
Compression-only Demand/(m = 7.76)	3.80	12,369	1.51	1,235	0.89

The results of this comparison show that the compression-only approach yields similar results to the tension/compression analysis when a large *m* (7.76) or *DCR* factor is used to reduce pseudo-elastic forces. Reduction of pseudo-elastic forces by an *m*-factor equal to 4 (which is also equivalent to the *m*-factor for overturning soil bearing and the *m*-factor for the structural concrete beam in Chapter 10) does not provide realistic results and would results in a larger footing.

The results of this case study indicate that the compression-only spring approach could be used provided that the applied m-factor or DCR is such that the reduced, applied loads do not make the model unstable. However, for this case study, with a shear wall designed using the fixed-base analysis procedure, this approach is not feasible. Given that we understand that this approach has been used successfully on building design by practicing engineers, we recommend additional case studies to investigate the use of this approach.

For the purposes of this case study and comparison between linear analysis procedures, the structural foundation will be evaluated using the elastic beam on elastic (tensions/compression) springs for the remainder of this section. The analyses utilize lower-bound spring stiffness. A comparison between lower and upper-bound stiffness is provided at the end of this section. For the nonlinear static model, the forces in the foundation are taken directly from the model since the footing is explicitly modeled.

For each analysis model, the structural foundation is evaluated as force-controlled and deformation-controlled for comparison as discussed below.

B.9.2.2 FORCE-CONTROLLED – EVALUATION

Structural foundation evaluation in accordance with ASCE/SEI 41-17 provisions requires that concrete structural components be evaluated assuming force-controlled actions. The loads on the foundation are applied as discussed above and the internal forces in the footing are compared to the calculated strength using lower bound strength properties with no *m*-factors applied. Two approaches to the force-controlled evaluation were performed.

1. Unreduced, pseudo-elastic forces are applied to the foundation based on the superstructure analysis model.
2. Alternatively, in accordance with ASCE/SEI 41-17 § 10.12.3, the capacity of the foundation components need not exceed 1.25 times the capacity of the supported vertical structural component or element (column or wall). In this case, the overturning forces applied to the footing are based on 1.25 times the flexural strength of the shear wall.

The acceptance ratios for the force-controlled structural foundation evaluations are summarized in Table B-36 and Table B-37. Based on these results, it is clear for all analysis procedures that the force-controlled analysis yields significantly higher acceptance ratios than the ASCE/SEI 7-10 foundation design and therefore significantly more conservative structural foundation designs.

B.9.2.3 DEFORMATION-CONTROLLED – EVALUATION

A similar evaluation of the structural foundation using deformation-controlled methodology was also performed with loading applied as described above. For linear procedures, the internal forces in the footings were compared to flexure and shear capacities calculated using expected strength properties and *m*-factors from the concrete material chapter (Chapter 10), specifically for concrete beams. For the NSP model, rotation demand in the structural foundation was compared to allowable rotation in Chapter 10.

The results of the deformation-controlled analysis are also shown in Table B-36 and Table B-37. Typically, this approach yields results more similar to the ASCE/SEI 7-10 design results.

B.9.2.4 SUMMARY OF FOUNDATION EVALUATION RESULTS

Included below are the structural foundation results for multiple cases showing their results for both force-controlled and deformation-controlled cases. For simplicity, the results shown herein are determined at the new footing section at the face of the existing footing.

Table B-36 Summary of Retrofit Structure Foundation Design – Unreduced Loading[1]

Retrofit Footing Design Comparison - Unreduced, Pseudo-Elastic Forces[2]					
Model	**Design Moment in Footing (k-ft)**	**Moment Acceptance Ratio**	**Design Shear in Footing (k)**	**Shear Acceptance Ratio**	**Action Classification**
ASCE/SEI 7-10 (for comparison)	7,734	0.99	797	0.73	Force-controlled
LSP - Fixed Base	62,128	7.56	7,163	5.16	Force-controlled
		1.89		1.29	Deformation-Controlled[4]
LSP - Method 1 Lower Bound (Rigid Footing)[3]	45,240	5.51	3,546	2.55	Force-controlled
		1.07		0.51	Deformation-Controlled[4]
LSP - Method 1 Upper Bound (Rigid Footing)[3]	56,462	6.87	4,386	3.16	Force-controlled
		1.33		0.51	Deformation-Controlled[4]
LSP - K_{50} 300$M_{c,foot}$ (Rigid Footing)[3]	28,195	3.25	2,268	1.56	Force-controlled
		0.67		0.33	Deformation-Controlled[4]
LSP - K_{50} 550$M_{c,foot}$ (Rigid Footing)[3]	36,780	4.23	2,932	2.02	Force-controlled
		0.87		0.43	Deformation-Controlled[4]
NSP - Method 3	16,180	1.86	1,547	1.06	Force-controlled
	N/A	0.00	N/A	0.00	Deformation-Controlled[5]

(1) Analysis includes retrofit footing sized previously in Hypothesis 1. Footing was designed to conform with ASCE/SEI 7-10 provisions and is 6 feet wide by 4 feet deep with (30) #11 top and bottom and (4) #5 stirrups at 6”oc.
(2) Design moment and shear for LSP models are amplified elastic forces.
(3) Footing designed based on elastic beam methodology with lower-bound soil springs.
(4) Footing strength is equal to the expected strength multiplied by a m-factor of 4 (determined from Chapter 10).
(5) There was no plastic rotation in the footing, therefore acceptance ratio is 0

Table B-37 Summary of Retrofit Structure Foundation Design – Capped Loading[1]

<table>
<tr><th colspan="6">Retrofit Footing Design Comparison - Forces Limited by 1.25 x Expected Force to Footing[2]</th></tr>
<tr><th>Model</th><th>Design Moment in Footing (k-ft)</th><th>Moment Acceptance Ratio</th><th>Design Shear in Footing (k)</th><th>Shear Acceptance Ratio</th><th>Action Classification</th></tr>
<tr><td>ASCE/SEI 7-10 (for comparison)</td><td>7,734</td><td>0.99</td><td>797</td><td>0.73</td><td>Force-controlled</td></tr>
<tr><td rowspan="2">LSP - Fixed Base</td><td rowspan="2">37,209</td><td>4.53</td><td rowspan="2">4,286</td><td>3.09</td><td>Force-controlled</td></tr>
<tr><td>0.88</td><td>0.62</td><td>Deformation-Controlled[4]</td></tr>
<tr><td>LSP - Method 1 Lower Bound (Rigid Footing)[3]</td><td rowspan="4">12,275</td><td rowspan="2">1.49</td><td rowspan="4">1,079</td><td rowspan="2">0.78</td><td rowspan="2">Force-controlled</td></tr>
<tr><td>LSP - Method 1 Upper Bound (Rigid Footing)[3]</td></tr>
<tr><td>LSP - K_{50} 300$M_{c,foot}$ (Rigid Footing)[3]</td><td rowspan="2">0.29</td><td rowspan="2">0.16</td><td rowspan="2">Deformation-Controlled[4]</td></tr>
<tr><td>LSP - K_{50} 550$M_{c,foot}$ (Rigid Footing)[3]</td></tr>
<tr><td rowspan="2">NSP - Method 3</td><td>16,180</td><td>1.86</td><td>1,547</td><td>1.06</td><td>Force-controlled</td></tr>
<tr><td>N/A</td><td>0.00</td><td>N/A</td><td>0.00</td><td>Deformation-controlled[5]</td></tr>
</table>

(1) Analysis includes retrofit footing sized previously in Hypothesis 1. Footing was designed to conform with ASCE/SEI 7-10 provisions and is 6 feet wide by 4 feet deep with (30) #11 top and bottom and (4) #5 stirrups at 6”oc.
(2) Design forces for footing capped at 1.25x the maximum expected strength (moment and shear) of the retrofit shear wall provided above the footing per ASCE/Sei 41-17 Section 10.12.3.
(3) Footing designed based on elastic beam methodology with lower-bound soil springs.
(4) Footing strength is equal to the expected strength multiplied by a m-factor of 4 (determined from Chapter 10)
(5) There was no plastic rotation in the footing, therefore acceptance ratio is 0.

B.9.2.5 STIFFNESS BOUNDING COMPARISON OF LINEAR PROCEDURES

The effects of lower and upper-bound linear stiffness assumptions on the supporting springs were also investigated for the linear procedures. Results are shown in Table B-38. All analyses are based on evaluation of force-controlled actions.

Table B-38 Summary of Retrofit Structure Foundation Design – Unreduced Loading

Stiffness Bounding for Footing Strength Design								
Model	**Lower-Bound Stiffness**				**Upper-Bound Stiffness**			
	Moment (k-ft)	**Moment Acceptance Ratio**	**Shear (kip)**	**Shear Acceptance Ratio**	**Moment (k-ft)**	**Moment Acceptance Ratio**	**Shear (kip)**	**Shear Acceptance Ratio**
LSP - Fixed Base	62,128	7.56	7,163	5.16	47,266	5.75	7,000	5.04
LSP - Method 1 Lower Bound (Rigid Footing)	45,240	5.51	3,546	2.55	-	-	-	-
LSP - Method 1 Upper Bound (Rigid Footing)	-	-	-		56,462	6.87	4,386	3.16
LSP - K_{50} 300$M_{c,foot}$ (Rigid Footing)	28,195	3.25	2,268	1.56	-	-	-	-
LSP - K_{50} 550$M_{c,foot}$ (Rigid Footing)	-	-	-	-	29,124	3.54	3,074	2.21

Note: Moment and Shear design forces represent unreduced pseudo-elastic forces. Moment and shear acceptance ratios reflect force-controlled methodology.

B.9.2.6 CONCLUSIONS

Design of the footing using ASCE/SEI 41-17 force-controlled actions is significantly conservative compared to ASCE/SEI 7-10 design. Footing design based on deformation-controlled actions using acceptance criteria from the appropriate material chapters should be permitted. The use of an elastic beam modeling approach utilizing expected stiffness, elastic springs provides a reasonable approach to foundation design and evaluation. Note that the use of upper- or lower-bound stiffness may yield unconservative results depending on the foundation condition; therefore expected stiffness is recommended. As an alternative, reduction of pseudo-elastic forces by an *m*-factor or *DCR* may provide reasonable results depending on stability of the compression-only soil spring analysis model.

B.9.3 Recommended Changes

In the ASCE/SEI 41-23 code, footing design based on deformation-controlled actions will be included explicitly and foundation compression action m-factors will be included in a new table.

B.10 Nonlinear Static Procedure

B.10.1 Motivation

Many of the hypotheses compare linear results from the linear static procedure (LSP) to nonlinear results from the nonlinear static procedure (NSP). In these comparisons, the nonlinear results are utilized as the benchmark to calibrate linear procedures to. However, nonlinear procedures are not presumed to precisely estimate the real building performance, nor does this study seeking to prove accuracy of nonlinear modeling compared to true building performance. The nonlinear methodology of ASCE 41 has been calibrated to research-based testing data; therefore, for this study, it is assumed the nonlinear analyses are more accurate in determining structural response and provide sufficient data to examine the hypotheses related to linear analyses. In the process of performing the NSP analyses for calibration with liner procedures, some interesting topics and interpretations related to the NSP were discovered and are discussed herein for reference.

B.10.3 Technical Studies

B.10.3.1 METHOD 1 - STIFFNESS DERIVATION

The retrofit footing is treated as a rigid body for the Method 1 spring stiffness derivations. Method 1 uses uncoupled moment and axial springs to model rigid foundations such that moment and shear behaviors are independent of axial load. Shear springs may also be used, in this case, and for all methods in this investigation, lateral moment is restrained within the analysis model. See Section B.8.3.1.1 for more information on the derivation of these springs for this case study.

B.10.3.2 METHOD 2 - STIFFNESS DERIVATION (ASCE/SEI 41-17 FIGURE 8-5)

Method 2 provides an alternative approach for rigid foundations that uses a bed of nonlinear springs that accounts for coupling between vertical loads and moments. The moment-rotation and vertical load deformation characteristics are modeled as a beam on a nonlinear Winkler foundation with stiffer vertical springs at the end regions to allow for tuning of the springs to approximately match the elastic vertical and rotational stiffness from Method 1. A graphical representation of the Method 2 springs is shown in Figure B-56.

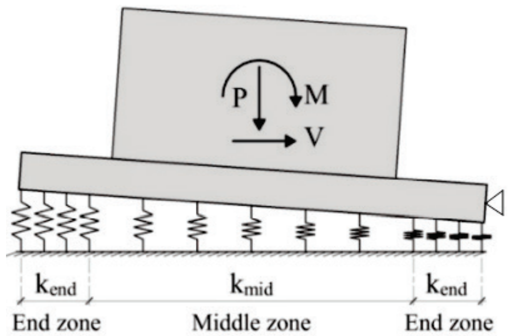


Figure B-56 Method 2 foundation springs (FEMA P-2006).

ASCE/SEI 41-17 § 8.4.2.4.1 requires that the Method 2 springs be tuned to approximately match the stiffness from Method 1 but does not provide a variable for tuning in the equations provided in Figure 8-4. There is a reference to Gajan et al. (2010) in ASCE/SEI 41-17 § C8.4.2.4.1 which provides a methodology for tuning. For the derivation shown below, no tuning is performed on the calculated stiffness. The next section utilizes the Gajan et al. approach for tuning. The results of the analysis with both approaches are compared. The un-tuned Method 2 springs are derived in accordance with ASCE/SEI 41-17 Figure 8-5.

k_{end}=	9545	kip/ft/unit length			factor =	6.83
k_{middle}=	1020	kip/ft/unit length			factor =	0.73
lend=	1.45	ft (B/6)				
li=	3.38	ft				
# springs =	20.0	middle zone				
K_{end}=	13810	kip/ft	1150.8	kip/in	<-	go into etabs
K_{middle}=	3449	kip/ft	287.4	kip/in	<-	go into etabs

Figure B-57 Derivation of Method 2 soil springs – ASCE/SEI 41-17 Figure 8-5.

B.10.3.3 METHOD 2 SOIL STIFFNESS DERIVATION – TUNED PER GAJAN ET AL. (2010)

Gajan et al. provides a methodology for tuning the middle and end Winkler springs to match the Method 1 stiffness values. This is done by first determining the length tributary to the end spring based on aspect ratio, then determining the factors for each of the springs based on the footing aspect ratio. The relationship of these parameters to the aspect ratio are shown in Figure B-58 and Figure B-59.

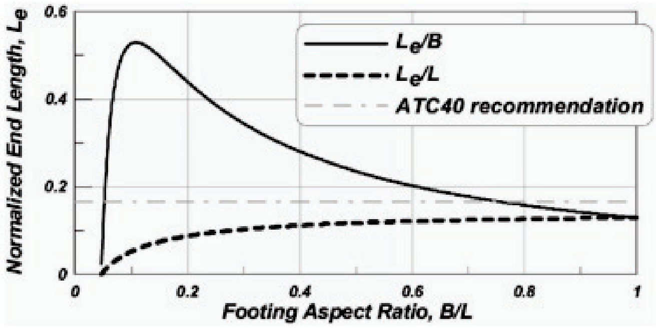


Figure B-58 End length versus aspect ratio (Gajan et al, from Harden and Hutchinson, 2009).

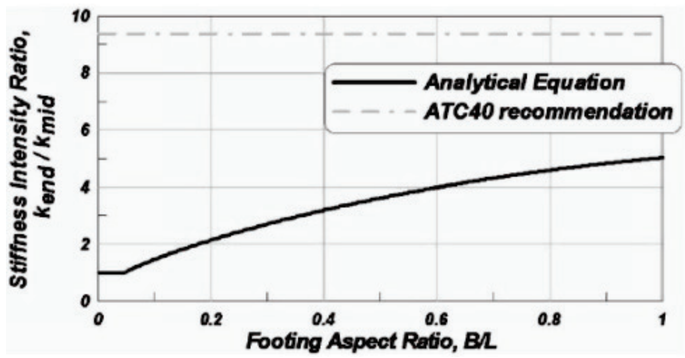


Figure B-59 Stiffness intensity ratio versus aspect ratio (Gajan et al, from Harden and Hutchinson, 2009).

The end length and the intensity ratio are determined in accordance with the figures above. The factors on the middle and end springs are then determined by comparing the total and end deflection to the results from Method 1. The Method 1 test loads and deflections are shown below.

Method 1			
	test loads for tuning		
	M =	500	kip-ft
	P =	1000	kips
	Deflection at end of footing		
	x_z=	0.1198	inch (axial spring)
	x_{yy}=	0.0032	inch (rotational spring)
	x_{total} =	0.1230	inch

Figure B-60 Method 1 test loads and associated deflections.

The Method 2 soil springs, tuned to the Method 1 deflections, are shown below. Further explanation of soil spring tuning is provided in FEMA P-2006.

B/L =	0.12						
Le/B =	0.50						
k_{end}/kmiddle =	1.53						
k_{end}=	2096	kip/ft/unit length				factor =	1.5
k_{middle}=	1370	kip/ft/unit length				factor =	1.0
lend=	2.50	ft					
li=	3.28	ft					
# springs =	20.0	middle zone					
K_{end}=	5241	kip/ft	436.7	kip/in	<-	go into etabs	
K_{middle}=	4485	kip/ft	373.8	kip/in	<-	go into etabs	
K_{total}=	100189	kip/ft					
x_z=	0.1198	inch	OK				
F_{end}=	1.5	kips					
x_{yy}=	0.0033	inch	OK				
x_{total} =	0.1231	inch					

Figure B-61 Derivation of Method 2 soil springs – tuned per Gajan et al.

B.10.3.4 METHOD 3 - STIFFNESS DERIVATION (ASCE/SEI 41-17 EQUATION 8-11)

Similar to Method 2 soil springs, Method 3 soil springs utilize decoupled Winkler springs. Method 3 diverges from Method 2 because it is intended for shallow foundations not rigid relative to the soil. It also has uniform springs beneath a footing, whereas the Method 2 springs have stiffer end bearing springs. Method 3 soil stiffness is derived in Section B.8.3.1.3.

B.10.4 Modeling Parameters and Acceptance Criteria

B.10.4.1 METHOD 1 – MODELING PARAMETERS AND ACCEPTANCE CRITERIA

Method 1 modeling parameters and acceptance criteria are described in Section B.8.3.2.1.

The capacity of the soil springs for Method 2 is noted in ASCE/SEI 41-17 § 8.4.2.4.2 as equal to the expected bearing capacity of the soil in compression and equal to zero in tension. A representative Method 2 soil spring is shown in Figure B-62. The acceptance criteria is based on the rotation limits of ASCE/SEI 41-17 Table 8-4. The rotation modeling parameters noted in ASCE/SEI 41-17 Table 8-4 are not utilized in Method 2, since the springs are for axial actions.

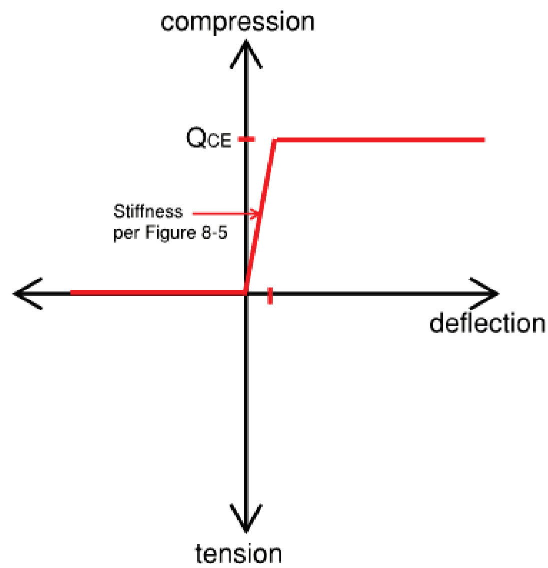


Figure B-62 Method 2 NSP modeling parameters.

B.10.4.3 METHOD 3 - MODELING PARAMETERS AND ACCEPTANCE CRITERIA

Method 3 modeling parameters and acceptance criteria are described in Section B.8.3.2.3.

Within ASCE/SEI 41-17, it is noted that the Winkler springs should be tuned to the Method 1 Springs for Method 2 (§ 8.4.2.4.1) yet the equations do not include variables that can be tuned. Specifying a tuning approach, if required for Method 2, is recommended in the future development of ASCE 41 Chapter 8.

B.10.5 Results

B.10.5.1 HYPOTHESIS 1 LSP CASE 8: FLEXIBLE BASE CONDITION (METHOD 1), WITH FOUNDATION RETROFIT

For comparison, Case 8 as described in Hypothesis 1 in Section B.3.1.1 is the Linear Case 5 analysis model from Hypothesis 1 utilized with the addition of a rigid retrofit footing and Method 1 linear soil springs. A single rotational and axial spring is assigned directly below the shear wall. Method 1 rotational and axial springs associated with the existing pad foundations at each column are also added beneath each existing footing. The deflected shape under dead load and earthquake load is shown in Figure B-63.

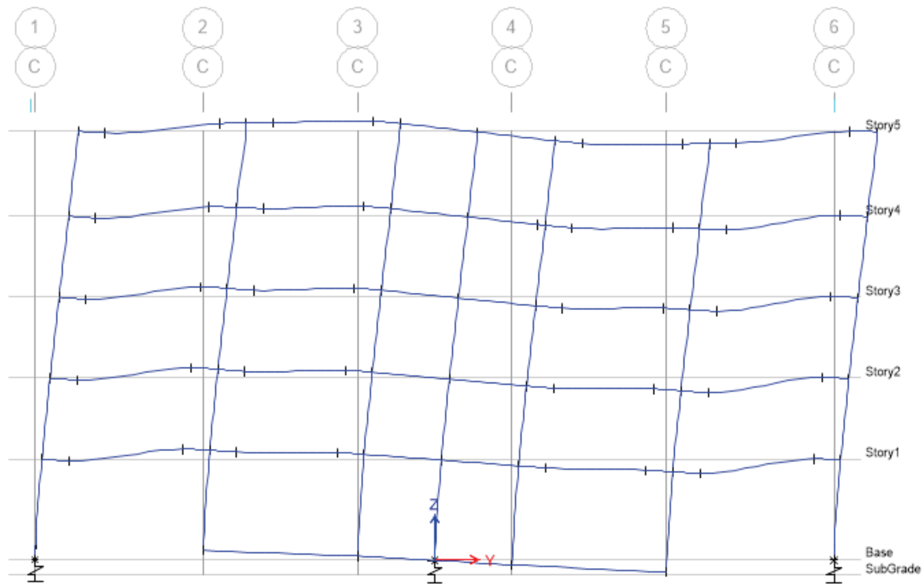


Figure B-63 LSP Analysis model with Method 1 soil springs.

The foundation acceptance criteria for this analysis can be seen in Table B-7. The moment demand at the base of the footing is the output from the rotational soil spring. The acceptance ratios from Method 1 flexible base and Method 1 fixed base in this table show similar results for bearing pressure, uplift, and overall overturning stability.

B.10.5.2 NSP CASE 1: NSP ANALYSIS – FIXED BASE

Figure B-64 shows the fixed base nonlinear pushover analysis force-displacement curve and deformed shape of the structure in elevation. Note that the shear walls are modeled as frame elements with flexural hinges top and bottom and a shear hinge at the center of each wall element. The wall frame element is located at the center of the elevation between gridlines 3 and 4. The calculated target displacement is equal to 5.3 inches. The fundamental period of the structure is 0.45 seconds, which matches the LSP analysis. There is no acceptance criteria in ASCE/SEI 41-17 for fixed base nonlinear procedures; these results are used as a comparison to the following flexible base analyses.

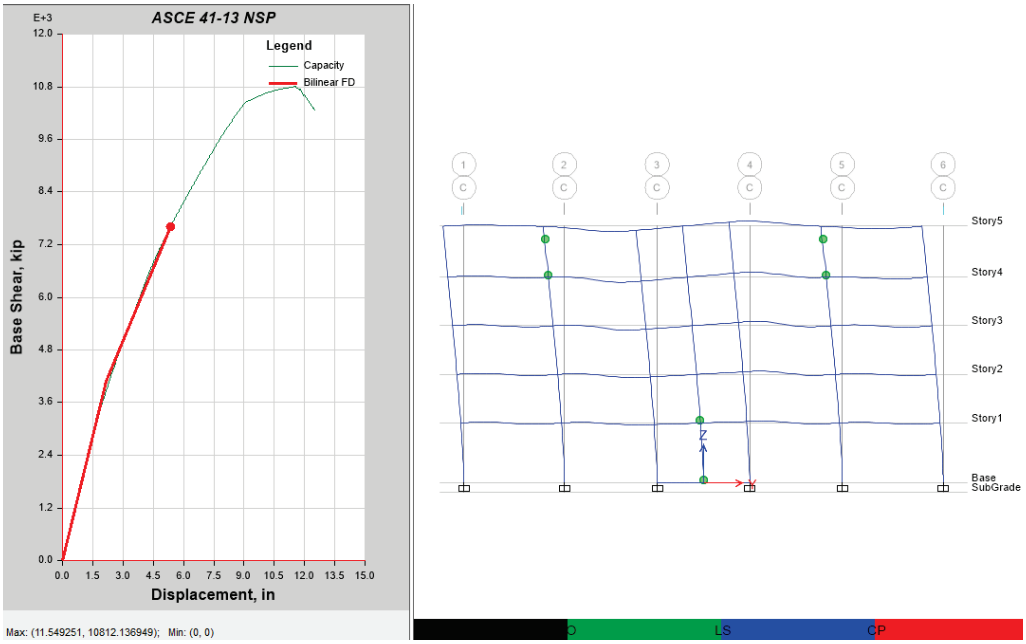


Figure B-64 ASCE/SEI 41-17 Nonlinear static pushover, fixed base model.

B.10.5.3 NSP CASE 2: NSP ANALYSIS – METHOD 1

Figure B-65 shows the nonlinear pushover force-displacement curve and displacements for the nonlinear model with Method 1 foundation springs. The relative superstructure hinge acceptance ratios are shown graphically, but are investigated in a later section. The horizontal displacement at each node at the base of the structure is restrained. Rotational and compression-only springs are applied at each individual existing footing. At the retrofit foundation, a single rotational and compression-only spring is located at the center of the base of the shear wall to capture the stiffness of the entire footing. The footing designed using ASCE/SEI 7-10 procedures (6-foot-wide by 4-foot-deep with (30) #11s top and bottom) is used in the analysis model. The calculated target displacement for this model is 10.8 inches and the fundamental period is 0.54 seconds.

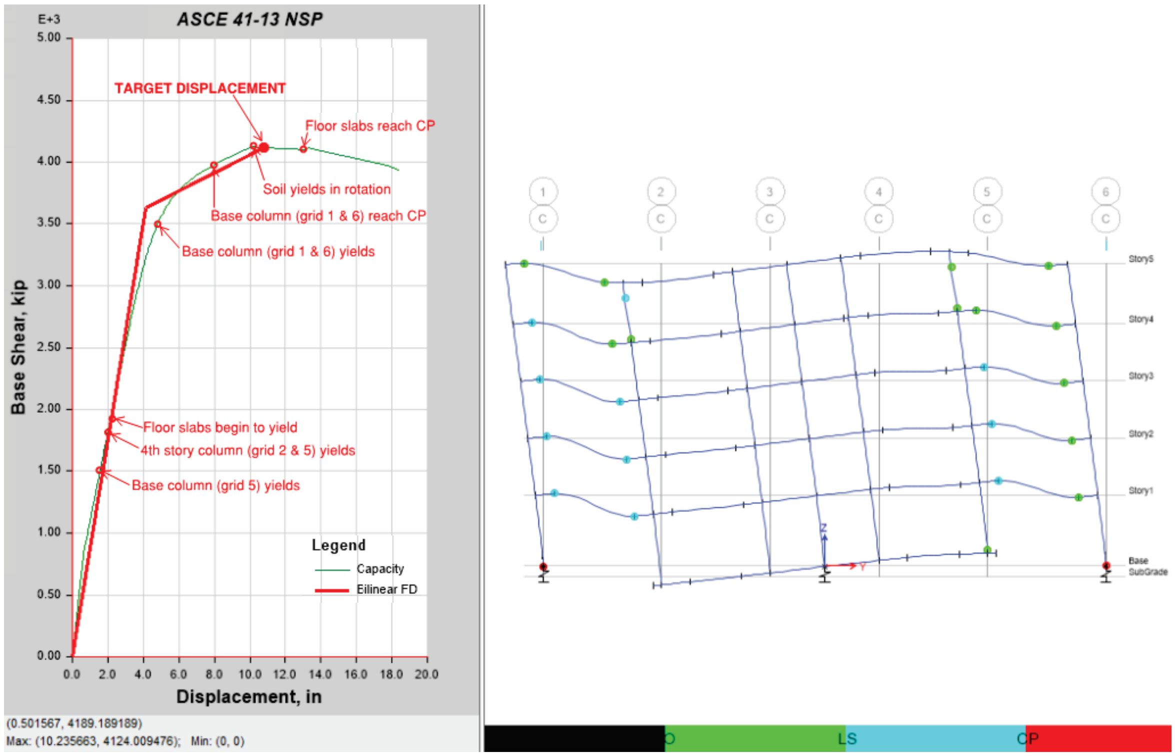


Figure B-65 ASCE/SEI 41-17 Nonlinear static pushover, Method 1 flexible base model.

The base reactions of the flexible base (Method 1) model are displayed in Figure B-66 at the target displacement. The rotation demand in the spring at the base of the shear wall is utilized to evaluate the acceptance criteria for the foundation rotation.

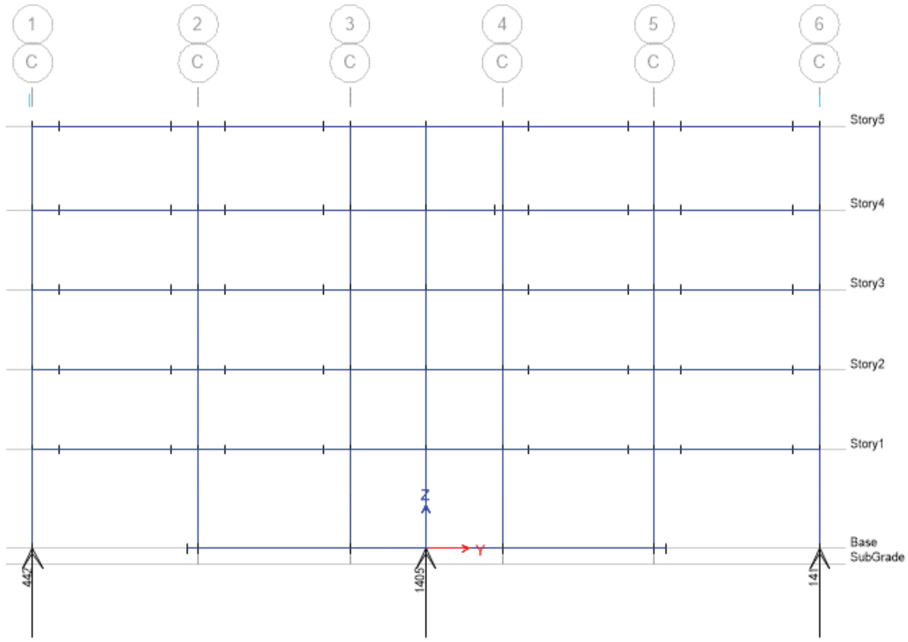


Figure B-66 ASCE/SEI 41-17 Nonlinear static pushover, Method 1 flexible base model, base reactions at target displacement.

The rotation of the retrofit footing at the target displacement is displayed in Figure B-67.

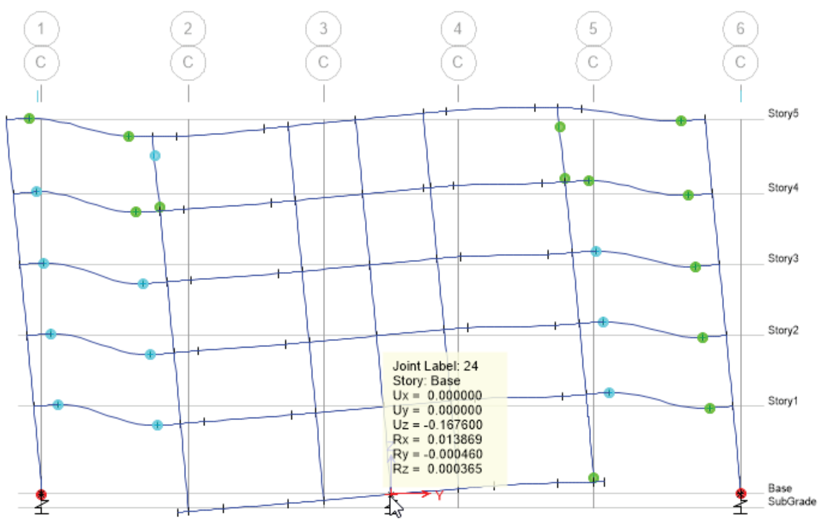


Figure B-67 ASCE/SEI 41-17 Nonlinear static pushover, Method 1 flexible base model foundation rotation at target displacement (Rotation units are radians; displacement units are inches).

The axial load shown in Figure B-66 is defined as P_{UD} in the calculations shown below. The footing rotation (Figure B-67) is then compared to the allowable rotation from ASCE/SEI 41-17 Table 8-4 to determine an acceptance ratio.

	P_{UD}=	1405	kips		
	q=	2.30	ksf		
	q_C=	10.5	ksf, expected strength		
	A_C=	134	ft2		
	A_f=	612	ft2		
	A_C/Af=	0.22			
	b/Lc =	0.12			
allowable rotation =		0.0238	radians, 41-17 Table 8-4		
rotation at target =		0.0139	radians		
acceptance ratio =		0.58			

Figure B-68 ASCE/SEI 41-17 Nonlinear static procedure soil acceptance criteria and acceptance ratio.

The flexural demand in the retrofit footing is reported at the target displacement to assess the ASCE/SEI 7-10 designed footing. The footing is evaluated as force-controlled with the lower-bound strength as specified in ASCE/SEI 41-17 § 10.12.3. The footing flexural action has an acceptance ratio of 1.37, so it is not acceptable and would require additional strength with this analysis approach.

ASCE/SEI 41-17 provides guidance for determining when a foundation is rigid compared to soil in the commentary, § C8.4.2.1 by comparing the foundation stiffness to the soil stiffness in Equation C8-1. The calculations for this footing are shown below. Based on this definition, the footing is not rigid compared to the soil; therefore, Method 1 is not applicable.

ksv =	0.09	kip/in3 (stiffness / (width x length)		
B =	8.7	ft (average width)		
I =	960033	in4	(8.7 feet wide x 4 feet deep)	
E =	4030.5	ksi	57000*sqrt(f'c)	
EI/L^4 =	0.008	ksi		
2/3ksvB =	6.6	ksi	>>EI/L4, not rigid	

Figure B-69 ASCE/SEI 41-17 § C8.4.2.1 flexibility of shallow foundation.

B.10.5.4 NSP CASE 3: NSP ANALYSIS – METHOD 2 – NON-TUNED SPRING

The soil springs derived in Section B.10.3.2 are utilized in this analysis model and the same footing is used in this analysis case. The flexural demand is then assessed at the target displacement to verify the footing. The footing is undersized for the force-controlled flexural demands, with an acceptance ratio of 2.29.

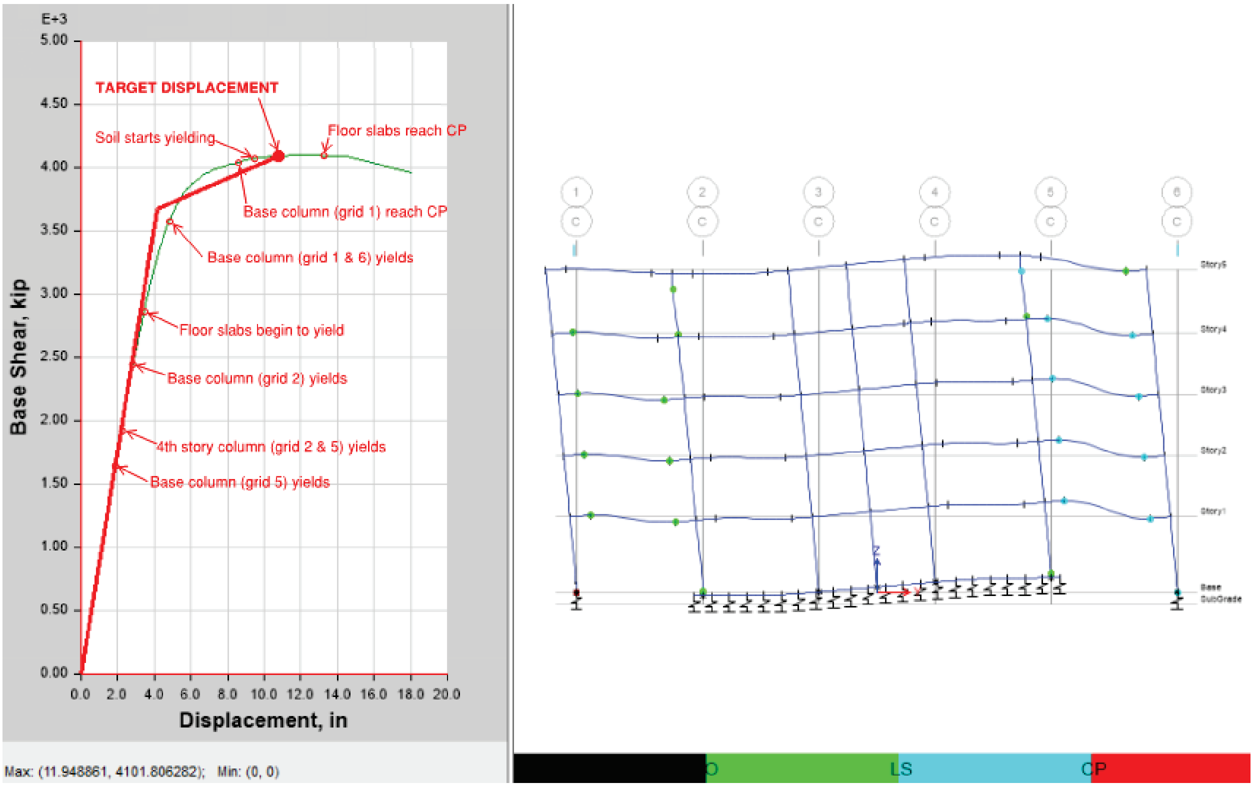


Figure B-70 ASCE/SEI 41-17 Nonlinear static pushover, Method 2 flexible base model, ASCE/SEI 41-17 Figure 8-5.

The pushover is shown in Figure B-70 and the acceptance criteria is determined in the calculations below. The footing rotation at the target displacement meets the acceptance criteria. The flexural action in the footing is also assessed at the target displacement to verify the footing strength. Based on a force-controlled evaluation, the footing strength is inadequate with an acceptance ratio of 2.29.

P_{UD}=	1793	kips
q_C=	10500	psf
A_C=	171	ft2
A_f=	612	ft2
A_C/Af=	0.28	
b/Lc =	0.12	
allowable rotation =	0.0195	radians, 41-17 Table 8-4
rotation at target =	0.0144	radians
acceptance ratio =	0.74	

Figure B-71 ASCE/SEI 41-17 Method 2, Figure 8-5 – soil acceptance criteria.

One discussion point to note, the acceptance criteria in Table 8-4 is highly dependent on the A_c/A_f factor and the b/L_c of the footing. The allowable rotation is highly sensitive to the footing area, and in this case the footing width since the length is constrained. The sensitivity of the acceptance criteria to the footing width is shown in Table B-39; when the footing width doubles, the allowable rotation increases by a factor of 5.7. Further investigation into the allowable rotation sensitivity to footing geometry and the A_c/A_f factor in accordance with ASCE/SEI 41-17 Table 8-4 is recommended.

Table B-39 Summary of Footing Retrofit Allowable Rotations from ASCE/SEI 41-17 Table 8-4 (for rectangular footing)

Comparison of Footing Size and Allowable Rotation	
Footing Width	**Allowable Rotation**
4’-0”	0.0030 radians
5’-0”	0.0040 radians
6’-0”	0.0100 radians
7’-0”	0.0141 radians
8’-0”	0.0172 radians

B.10.5.5 NSP CASE 4: NSP ANALYSIS – METHOD 2 –TUNED SPRING, GAJAN ET AL.

The analysis model is then updated with the Method 2 tuned springs derived in Section B.10.3.3. The same ASCE/SEI 7-10 designed footing is used in this analysis and is evaluated for force-controlled flexure at the target displacement. The revised soil springs negligibly change the response of the structure, shown in Figure B-72.

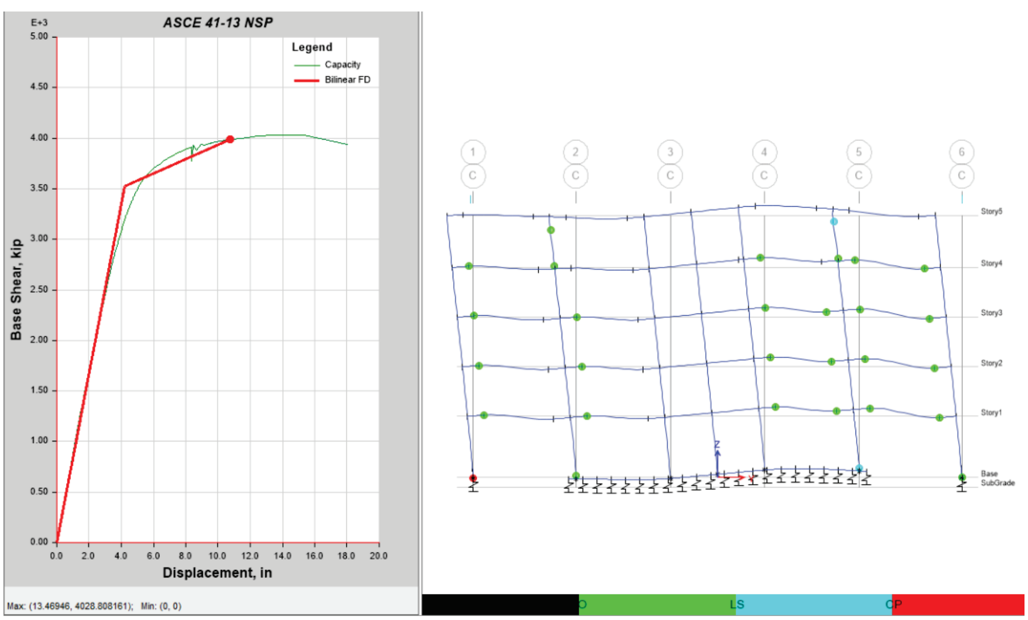


Figure B-72 ASCE/SEI 41-17 Nonlinear static pushover, Method 2 flexible base model, Gajan tuned springs.

The acceptance ratio for the footing rotation is similar to the initial Method 2 results as shown in the calculations below. The flexural foundation acceptance ratio is 1.69.

P_{UD}=	1810	kips
q_C=	10500	psf
A_C=	172	ft2
A_f=	612	ft2
A_C/Af=	0.28	
b/Lc =	0.12	
allowable rotation =	0.0193	radians, 41-17 Table 8-4
rotation at target =	0.0131	radians
acceptance ratio =	0.68	

Figure B-73 ASCE/SEI 41-17 Method 2, Gajan tuned – soil acceptance criteria.

B.10.5.6 NSP CASE 5: NSP ANALYSIS – METHOD 3 – FORCE-CONTROLLED FOUNDATION

The Method 3 soil springs (Section B.8.3.1.3) are incorporated into the model and the same footing is assessed for the force-controlled flexure at the target displacement. The Method 3 pushover analysis is shown in Figure B-74.

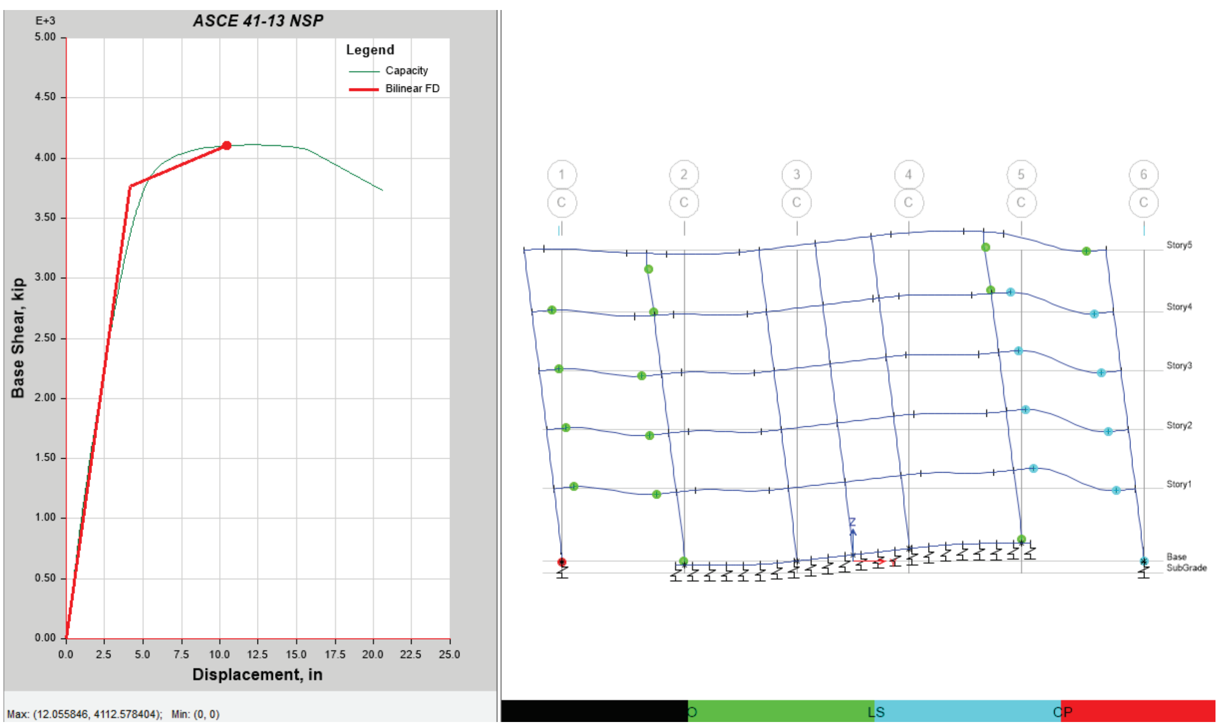


Figure B-74 ASCE/SEI 41-17 Nonlinear static pushover, Method 3 flexible base model.

The ASCE/SEI 7-10 footing is evaluated for force-controlled flexure and is not acceptable with an acceptance ratio of 2.39. The acceptance criteria calculations for the footing rotation are shown below.

	P_{UD}=	1842	kips		
	q_C=	10500	psf		
	A_C=	175	ft2		
	A_f=	612	ft2		
	A_C/Af=	0.29			
	b/Lc =	0.12			
allowable rotation =		0.0190	radians, 41-17 Table 8-4		
rotation at target =		0.0137	radians		
acceptance ratio =		0.72			

Figure B-75 ASCE/SEI 41-17 Method 3, force-controlled – soil acceptance criteria.

B.10.5.7 NSP CASE 6: NSP ANALYSIS – METHOD 3 – DEFORMATION-CONTROLLED FOUNDATION

A separate analysis using Method 3 soil springs is performed with the foundation structure evaluated as deformation-controlled. Although foundations are typically required to be evaluated as force-controlled in accordance with ASCE/SEI 41-17 § 10.12.3, the nonlinear modeling and acceptance criteria provisions for concrete beams within ASCE/SEI 41-17 Chapter 10 are applied to the foundation structure for this case. The ASCE/SEI 7-10 designed retrofit footing (6-foot-wide by 4-foot-deep with (30) #11 bars top and bottom) is incorporated into the analysis model with flexural hinges assigned to each end of the footing beams between the existing footings. The footing beam hinge collapse prevention acceptance criteria is assessed in accordance with the provisions of ASCE/SEI 41-17 Chapter 10. The acceptance ratio for the footing at the target displacement is 0.43. Therefore, the retrofit footing design is acceptable based on a deformation-controlled foundation design.

The pushover curve and deflected shape at the target displacement are shown in Figure B-76. The fundamental period of this model is 0.63 seconds and the target displacement is 10.7 inches.

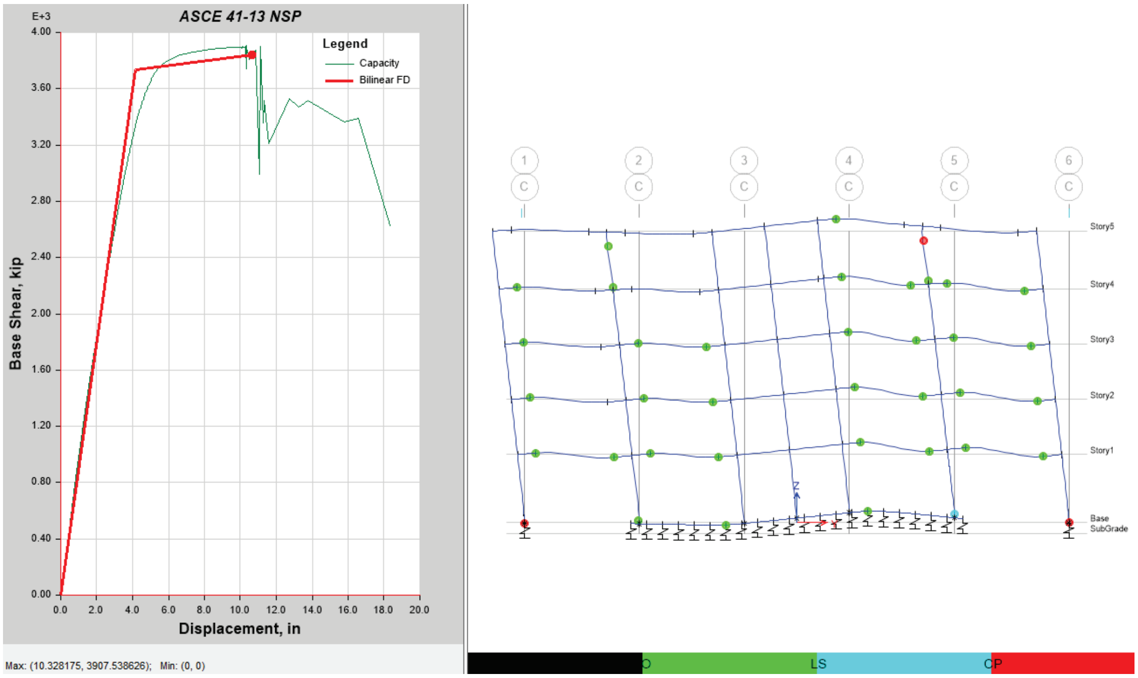


Figure B-76 ASCE/SEI 41-17 Nonlinear static pushover, Method 3 flexible base model.

The rotation at the base of the shear wall at the target displacement is compared to the acceptance criteria for footing rotation from ASCE/SEI 41-17 Table 8-4. The acceptance criteria calculations are shown in Figure B-77. L_c is taken as the full length of the footing, similar to previous calculations. In the next analysis case, a different interpretation of L_c is explored.

P_{UD}=	1768	kips
q_C=	10500	psf
A_C=	168	ft2
A_f=	612	ft2
A_C/Af=	0.28	
b/Lc =	0.12	
allowable rotation =	0.0198	radians, 41-17 Table 8-4
rotation at target =	0.0145	radians
acceptance ratio =	0.73	

Figure B-77 ASCE/SEI 41-17 Method 3, deformation-controlled – soil acceptance criteria.

B.10.5.8 NSP CASE 7: NSP ANALYSIS – METHOD 3 – DEFORMATION-CONTROLLED FOUNDATION, ACCEPTANCE CRITERIA DETERMINED AT INFLECTION POINTS

FEMA P-2006 is an Example Application Guide for ASCE/SEI 41-13 Seismic Evaluation and Retrofit of Existing Buildings with Additional Commentary for ASCE/SEI 41-17. As discussed in FEMA P-2006 § 5.7.6.1, a flexible footing could be assessed by evaluating individual sections separated at inflection points. See FEMA P-2006 for more information. For this case, the acceptance criteria is recalculated for a similar condition to NSP Case 6 but with the soil acceptance criteria evaluated with L_c defined for individual segments based on flexural inflection point locations. The three footing portions are shown in Figure B-78.

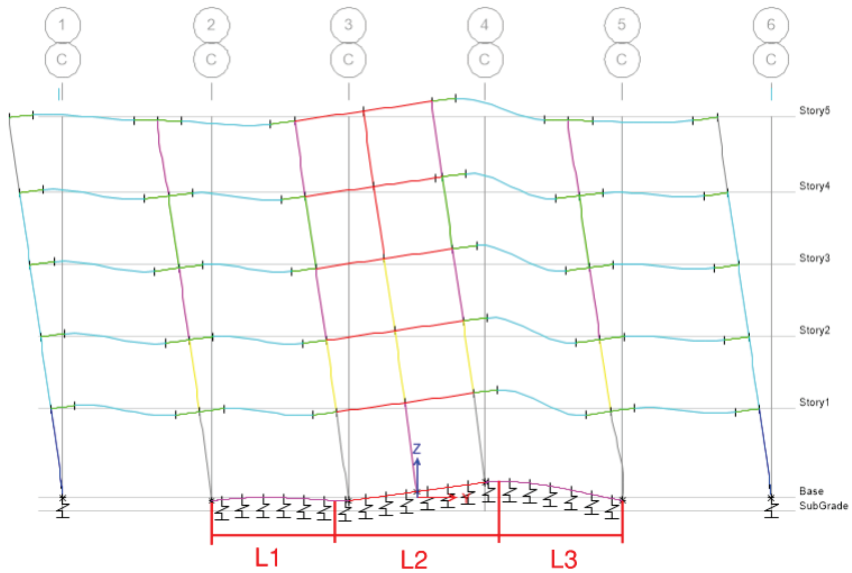


Figure B-78 ASCE/SEI 41-17 Nonlinear static pushover, Method 3 flexible base model (Equation 8-11). Footing divided into three sections for acceptance criteria calculations.

The results of the acceptance criteria calculations and acceptance ratios are shown in Table B-40. For each footing segment, the axial load and footing dimensions are used to calculate allowable rotations. All of the segments meet their acceptance criteria. The highest loaded segment, L1, also has the lowest rotation as the beam hinge adjacent to it is yielding which reduces the rotation demand.

Table B-40 Summary of Analysis Results for the Retrofit Structure Foundation

	A_c/A_f	Allowable Rotation, CP (radians)	Actual Rotation at Target (radians)	Acceptance Ratio
L1	0.78	0.0018	0.0009	0.49
L2	0.06	0.0424	0.0145	0.34
L3	0.00	0.0500	0.0143	0.20

Further investigation into the determination of L_c should be used for flexible foundations in accordance with ASCE/SEI 41-17 Table 8-4 is recommended.

B.10.5.9 SUMMARY OF FOUNDATION ACCEPTANCE CRITERIA

Table B-41 summarizes the previous seven nonlinear analyses, along with LSP for comparison, and the foundation acceptance criteria. For each analysis case, the soil acceptance ratio and the footing acceptance ratio are summarized. The footing acceptance ratio is calculated for each case utilizing the retrofit footing designed to ASCE/SEI 7-10 provisions. The only ASCE/SEI 41-17 analysis case where the footing is acceptable is the case where the footing is evaluated as deformation-controlled.

Table B-41 Summary of Analysis Results for the Retrofit Structure Foundation

Model	Initial Fundamental Period (seconds)	Target Displacement (inches)	Base Shear (kips) (1)	P_{UD} (kips)	A_c/A_f	Allowable rotation (radians)	Rotation at target (radians)	Soil Acceptance Ratio	Footing(2) Acceptance Ratio	Overall Outcome
LSP – ASCE/SEI 7-10	0.45	-	0.17W	1304	-	-	-	0.95 (max)	0.97	OK
LSP – fixed base	0.45	-	1.3W	1304	-	-	-	1.32 (max)	4.90	NG
LSP – method 1(3)	0.54	-	1.3W	1304	-	-	-	1.16	1.37	NG
NSP – fixed base	0.45	5.3	1.0W	-	-	-	-	-	-	
NSP – method 1(3)	0.54	10.8	0.57W	1405	0.22	0.0238	0.0138	0.55	1.37	NG

(1) W is equal to the effective seismic weight of the superstructure equal to 7,200 kips,

(2) Footing is 6 ft wide by 4 feet deep with (30) #11 top and bottom. The acceptance ratio is based on a force-controlled design of the footing in flexure unless stated otherwise,

(3) Method 1 and Method 2 are not applicable as the footing is not rigid relative to the soil, but are investigated here for comparison.

Table B-41 Summary of Analysis Results for the Retrofit Structure Foundation (Continued)

Model	Initial Fundamental Period (seconds)	Target Displacement (inches)	Base Shear (kips) [1]	P_{UD} (kips)	A_c/A_f	Allowable rotation (radians)	Rotation at target (radians)	Soil Acceptance Ratio	Footing[2] Acceptance Ratio	Overall Outcome
NSP – Method 2 (non-tuned) [3]	0.66	10.7	0.57W	1793	0.28	0.195	0.0144	0.74	2.29	NG
NSP – Method 2 (tuned) [3]	0.68	10.7	0.55W	1810	0.28	0.0193	0.0131	0.68	1.69	NG
NSP – method 3 (force-controlled)	0.63	10.5	0.57W	1842	0.29	0.0190	0.0137	0.72	2.39	NG
NSP – method 3 (deformation -controlled)	0.63	10.7	0.53W	1768	0.28	0.0198	0.0145	0.73	0.43	OK
NSP – method 3 Acceptance Criteria in sections	“	“	“	1633 135 0	0.78 0.06 0.00	0.0018 0.0424 0.0143	0.0009 0.0145 0.0143	0.49 0.34 0.29	“	OK

(1) W is equal to the effective seismic weight of the superstructure equal to 7,200 kips,
(2) Footing is 6 ft wide by 4 feet deep with (30) #11 top and bottom. The acceptance ratio is based on a force-controlled design of the footing in flexure unless stated otherwise,
(3) Method 1 and Method 2 are not applicable as the footing is not rigid relative to the soil, but are investigated here for comparison.

B.10.5.10 SUPERSTRUCTURE RESULTS

The superstructure results are determined for columns, slabs and the shear wall for each analysis model. The acceptance ratios from the superstructure are shown in Table B-42 through Table B-46. The LSP acceptance ratios compare the demand from ETABS to the deformation-controlled capacity in accordance with ASCE/SEI 41-17 equation 7-36. None of the superstructure elements investigated here are evaluated as force-controlled. The acceptance ratios for the NSP analyses compare the hinge rotation in the member under consideration to the acceptance criteria for collapse prevention as specified in ASCE/SEI 41-17 Chapter 10. If there is no inelastic rotation in the hinge at the target displacement, the acceptance criteria is listed as 0.00.

The same retrofit shear walls were utilized for all cases for effective comparison. Based on the results, the shear wall design could be optimized for the nonlinear design. In the nonlinear analysis no inelastic behavior occurs in the shear wall, the deformations are pushed into the surrounding structure which can be seen in the acceptance ratios of the existing columns and slabs.

The exterior columns do not meet the acceptance criteria as shown in the NSP pushover deformed shape with hinge acceptance figures in the previous results. These columns are intended to be retrofit and are not shown in the tables below. The existing structure was modeled as-is for all the analysis, however additional retrofit scope is likely required depending on the approach.

Table B-42 Existing Interior Column - Moment Acceptance Ratios (CP Limit State)

Analysis Model	1st Story	2nd Story	3rd Story	4th Story	5th Story	Outcome
LSP – Fixed Base	0.99	0.55	0.37	0.56	1.35	NG
NSP – Method 1 Springs	0.20	0.00*	0.00*	0.00*	0.81	OK
NSP – Method 2 Springs Tuned	0.66	0.00*	0.00*	0.00*	0.82	OK
NSP – Method 3 Deformation Controlled	0.91	0.00*	0.00*	0.00*	1.00	OK

Note: A DCR equal to 0.00 indicates no inelastic behavior occurs at the target displacement.

Table B-43 Existing Column - Shear Acceptance Ratios (CP Limit State) (For Nonlinear Cases Acceptance Ratio is the Same as the Moment Acceptance Ratio)

Analysis Model	1st Story	2nd Story	3rd Story	4th Story	5th Story	Outcome
LSP – Fixed Base	0.49	0.43	0.29	0.47	1.13	NG
NSP – Method 1 Springs	0.00*	0.00*	0.00*	0.00*	0.00*	OK
NSP – Method 2 Springs Tuned	0.00*	0.00*	0.00*	0.00*	0.00*	OK
NSP – Method 3 Deformation Controlled	0.00*	0.00*	0.00*	0.00*	0.00*	OK

Note: A DCR equal to 0.00 indicates no inelastic behavior occurs at the target displacement.

Table B-44 Retrofit Shear Wall - Shear Acceptance Ratios (CP Limit State)

Analysis Model	1st Story	2nd Story	3rd Story	4th Story	5th Story	Outcome
LSP – Fixed Base	0.48	0.96	0.87	0.72	0.34	NG
NSP – Method 1 Springs	0.00*	0.00*	0.00*	0.00*	0.00*	OK
NSP – Method 2 Springs Tuned	0.00*	0.00*	0.00*	0.00*	0.00*	OK
NSP – Method 3 Deformation Controlled	0.00*	0.00*	0.00*	0.00*	0.00*	OK

Note: A DCR equal to 0.00 indicates no inelastic behavior occurs at the target displacement.

Table B-45 Retrofit Shear Wall - Moment Acceptance Ratios (CP Limit State)

Analysis Model	1st Story	2nd Story	3rd Story	4th Story	5th Story	Outcome
LSP – Fixed Base	0.87	0.99	0.65	0.93	0.36	OK
NSP – Method 1 Springs	0.00*	0.00*	0.00*	0.00*	0.00*	OK
NSP – Method 2 Springs Tuned	0.00*	0.00*	0.00*	0.00*	0.00*	OK
NSP – Method 3 Deformation Controlled	0.00*	0.00*	0.00*	0.00*	0.00*	OK

Note: A DCR equal to 0.00 indicates no inelastic behavior occurs at the target displacement.

Table B-46 Existing Slab – Flexure Acceptance Ratios (CP Limit State)

Analysis Model	2nd Floor	3rd Floor	4th Floor	5th Floor	Roof	Outcome
LSP – Fixed Base	0.48	0.62	0.72	0.73	0.48	OK
NSP – Method 1 Springs	0.82	0.98	0.98	0.97	0.68	OK
NSP – Method 2 Springs Tuned	0.58	0.57	0.55	0.42	0.15	OK
NSP – Method 3 Deformation Controlled	0.56	0.58	0.59	0.56	0.28	OK

B.10.6 Methods for Determining Allowable Rotation at Footing-Soil Interface

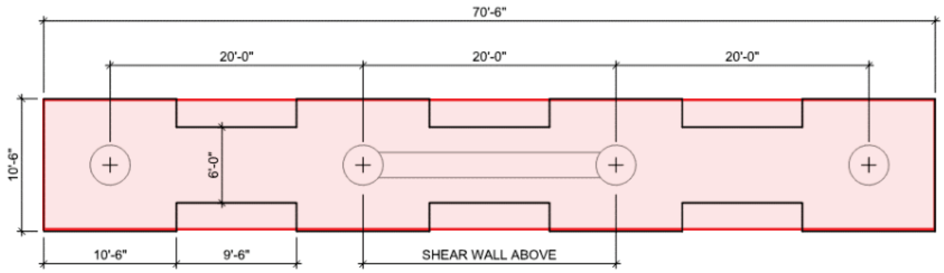
Tables 8-3 and 8-4 in Chapter 8 provide acceptance criteria for I-shaped footings when b/L_c is between 1 and 10. The retrofit footing used in this study was determined to have a b/L_c ratio outside of 1 and 10. Further investigation into determining footing allowable rotation was completed with the following methodologies for comparison:

- Entire retrofit footing length and width
- Entire retrofit footing length and effective footing width
- Equivalent I-shaped footing
- Inflection points
- I-shaped footing neglecting interior existing footings

Resulting values for comparison are provided in Table B-47.

B.10.6.1 ENTIRE RETROFIT FOOTING LENGTH AND WIDTH

The first method uses the entire retrofit footing length and width, neglecting the missing footing area at the locations of new foundation between existing footings. This method is consistent with the guidance provided in Figure 8-3 for an idealized footing. The idealized flange thickness is taken as the width of the existing isolated footing. The total area is the product of the idealized flange width and footing length.

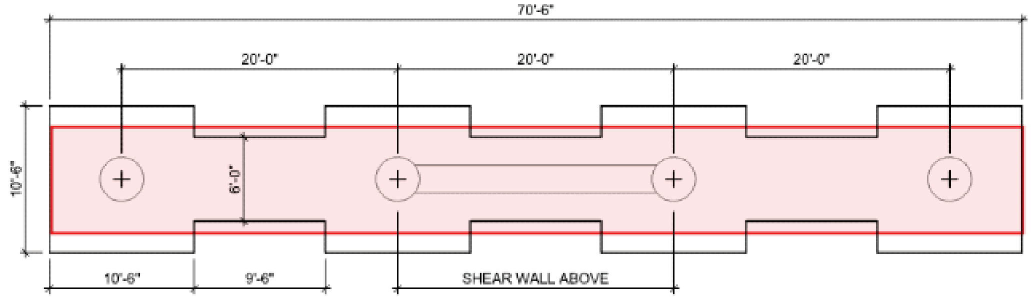


b=	10.5	ft
L =	70.5	ft
A_f=	740	ft^2
P_{UD}=	1769	kips
q_C=	10.5	ksf
A_C=	168	ft^2
A_f=	740	ft^2
A_{rect} =	740	ft^2
A_{rect} -A_f/A_{rect} =	0.00	
A_C/Af=	0.23	
L_c =A_c/b =	16.05	ft
b/Lc =	0.65	
allow. rotation =	0.0306	radians, 41-17 Table 8-4

Figure B-79 Allowable footing rotation utilizing entire footing length and width.

B.10.6.2 ENTIRE RETROFIT FOOTING LENGTH AND EFFECTIVE WIDTH

This method idealizes the footing as rectangular. An effective width is calculated by dividing the footing area of the retrofit footing by the length. This effective width is used to calculate the footing acceptance criteria.



b=	8.7	ft
L =	70.5	ft
A_f=	613	ft^2
P_{UD}=	1769	kips
q_C=	10.5	ksf
A_C=	168	ft^2
A_f=	613	ft^2
A_{rect} =	613	ft^2
A_{rect} -A_f/A_{rect} =	0.00	
A_C/Af=	0.27	
L_c =A_c/b =	19.37	ft
b/Lc =	0.45	
allow. rotation =	0.0224	radians, 41-17 Table 8-4

Figure B-80 Allowable footing rotation utilizing entire footing length and effective width.

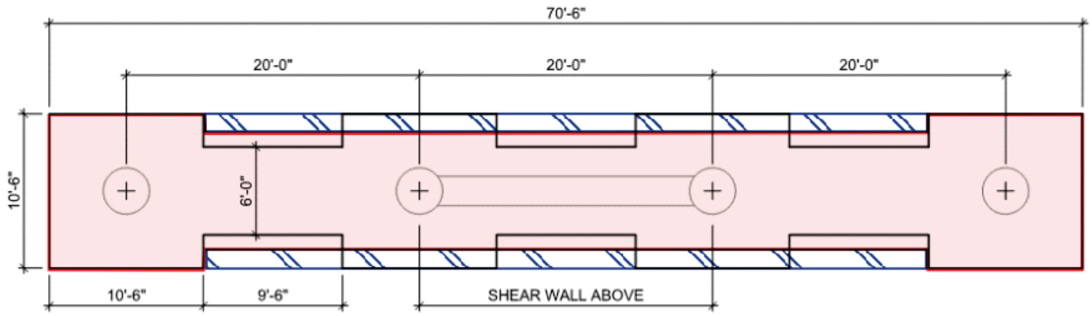
B.10.6.3 EQUIVALENT I-SHAPED FOOTING

Equivalent I-shaped footing dimensions are calculated to provide an equivalent moment of inertia as the actual footing configuration. The moment of inertia calculation is provided in Figure B-81.

Moment of Inertia - Retrofit Footing			
Contribution of interior existing footings			
b =	10.5	ft	
h =	10.5	ft	
A =	110.25	ft^2	
d=	10	ft	middle interior footings
$I = bh^3/12 + ad^2 =$	24,076	ft^4	
Contribution of web (between existing footings)			
b =	6	ft	
h =	9.5	ft	
A =	57	ft^2	
$d_1 =$	0	ft	middle web
$d_2 =$	20	ft	exterior web
$I = bh^3/12 + ad^2 =$	46,886	ft^4	
Total I =	70,962	ft^4	
Moment of Inertia of web - determine added width for transformed section			
h =	49.5	ft	
I =	70961.9	ft^4	
$b = 12I/h^3 =$	7.02	ft	
b =	7.02	ft, width used for transformed footing	

Figure B-81 Equivalent I-shaped footing moment of inertia.

The calculated b/L_c ratio for the equivalent I-shaped footing does not fall within the range specified in ASCE/SEI 41-17 (1 to 10). Allowable rotation varies based on use of rectangular versus I-shaped footing acceptance criteria values provided in Table 8-4. Figure B-82 provides both results for comparison.

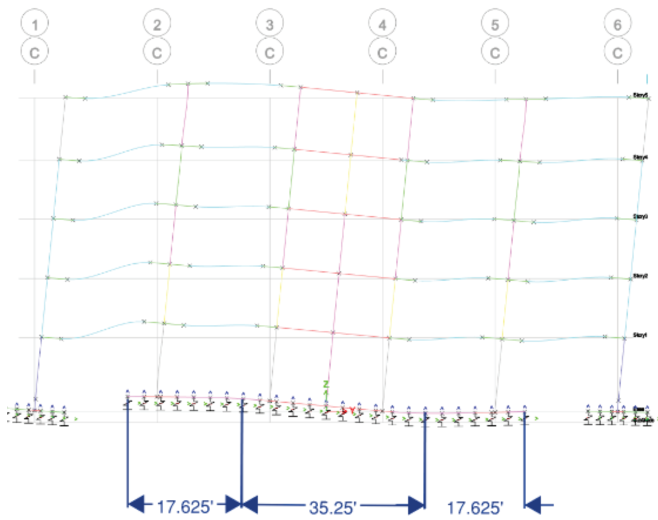


3) I-transform: convert retrofit footing to transformed section		
b=	10.5	ft
L =	70.5	ft
A_f=	568	ft^2
P_{UD}=	1769	kips
q_C=	10.5	ksf
A_C=	168	ft^2
A_f=	568	ft^2
A_{rect} =	740	ft^2
A_{rect} -A_f/A_{rect} =	0.23	
A_C/A_f=	0.30	
L_c =A_c/b =	16.05	ft
b/Lc =	0.65	
allow. rotation =	0.0239	radians, rectangular ftg
	0.0289	I-shaped footing

Figure B-82 Allowable footing rotation utilizing equivalent I-shape.

B.10.6.4 INFLECTION POINTS

Acceptance criteria is calculated for the portion of the footing between the inflection points based on displaced shape of the footing. The effective footing width is used for this approach.

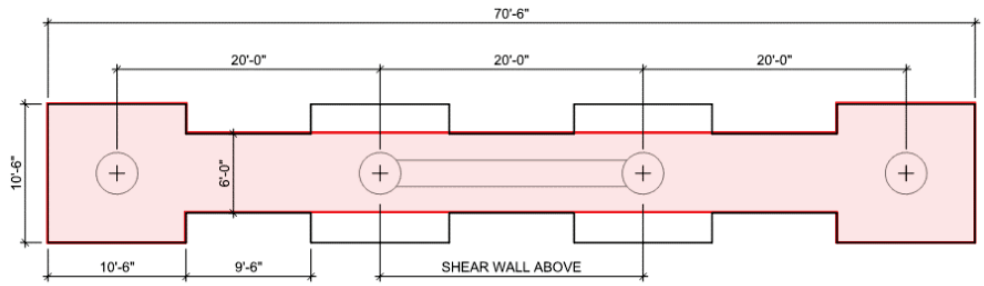


b=	8.7	ft
L =	35.3	ft
A_f=	307	ft^2
P_{UD}=	756	kips
q_C=	10.5	ksf
A_C=	72	ft^2
A_f=	307	ft^2
A_{rect} =	307	ft^2
A_{rect} -A_f/A_{rect} =	0.00	
A_C/Af=	0.23	
L_c =A_c/b =	8.27	ft
b/Lc =	1.05	
allow. rotation =	0.0374	radians, 41-17 Table 8-4

Figure B-83 Allowable footing rotation utilizing inflection points.

B.10.6.5 I-SHAPED FOOTING

An I-shaped footing neglecting existing pad footings at the interior columns on either side of the retrofit shear wall is assumed. The calculated b/L_c ratio for the equivalent I-shaped footing does not fall within the range specified in ASCE/SEI 41-17 (1 to 10). Allowable rotation varies based on use of rectangular versus I-shaped footing acceptance criteria values provided in Table 8-4. Figure B-84 provides both results for comparison.



b=	10.5	ft
L =	70.5	ft
A_f=	518	ft^2
P_{UD}=	1769	kips
q_C=	10.5	ksf
A_C=	168	ft^2
A_f=	518	ft^2
A_{rect} =	740	ft^2
A_{rect} -A_f/A_{rect} =	0.30	
A_C/Af=	0.33	
L_c =A_c/b =	16.05	ft
b/Lc =	0.65	
allow. rotation =	0.0210	radians, rectangular ftg
	0.0251	I-shaped footing

Figure B-84 Allowable footing rotation utilizing I-shaped footing.

B.10.6.6 SUMMARY

The following table compares the allowable footing rotation values using each of the methodologies described above.

Table B-47 Summary of Results for Allowable Footing Rotation

Approach	Ac/Af	b/Lc	Allowable Rotation (rad)
Entire footing length and width	0.23	0.65	0.0306
Entire footing length and effective width	0.27	0.45	0.0224
Equivalent I-shaped footing	0.30	0.65	0.0239
Inflection Points	0.27	0.45	0.0374
I-shaped footing neglecting interior existing footings	0.33	0.65	0.0210

Conclusions

- The current provisions can be applied to non-rectangular or non-I-shaped footings by a number of methods.
- Guidance should be provided to the user for cases where I-shaped footings when b/Lc is not between 1 and 10.
- There are multiple approaches to determining allowable rotations for the atypical foundation configuration.
- I-shaped vs. rectangular footings provide numerically different allowable rotations.
- Rotation demand can be determined as rotation between end points of wall or between points of contraflexure.

B.11 References

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Part 3, Appendix C: Archetype Building 2

C.1 Motivation/Goal of the Case Study

The motivation/goal of the study was to evaluate and revise current provisions for clarity, technical accuracy, optimal computations and ease of use using an actual building for the case study. Compare outcomes from analysis of the building designed using ASCE/SEI 7-10 evaluated as a new building using ASCE/SEI 41-17.

Objectives of the study were to 1) Calibrate the *m*-factors and acceptance criteria if necessary, so that outcomes show acceptable performance analytically consistent with engineering judgment. 2) Suggest simplifications in the analysis methods possible if they were too conservative than a more in-depth analysis would produce. 3) Look at the various modeling approaches for evaluation of the building either as fixed-base or flexible-base including bounding provisions for the soil and evaluate the impact on the elements of the superstructure and the foundation system. 4) Identify areas were the provisions lacked guidance or where there were gaps in the application of the provisions.5) Recommend changes based on the findings from the case study looking at all modeling approaches linear and nonlinear.

C.2 Case Study Model – Archetype 2, Concrete Moment Frame Building

This case study for Archetype Building 2 similar to the case study for Archetype Building 1 investigates the application of some of the methods specified in ASCE/SEI 41-17 Chapter 8 for clarity, usability and technical content. Various combinations of shallow foundation modeling options are created, to evaluate the shallow foundation provisions related to overturning actions from seismic loads. Sliding is not considered in this case study example and is assumed as fixed for all modeling cases. A baseline model is created where superstructure and foundations are designed to meet the requirements of ASCE/SEI 7-10. Parametric case studies are performed to investigate selected topics related to overturning actions on shallow foundations using ASCE/SEI 41-17. Foundation acceptance criteria and corresponding superstructure acceptance criteria are evaluated, and results compared for reasonableness assuming the building was designed to meet the requirements of the new building designed using ASCE/SEI 7-10 to avoid the requirement for a site-specific ground motion analysis or amplification of the response spectrum.

C.2.1 Building Description

The subject building is a modified version of seven-story reinforced concrete special moment frame building on shallow foundations (Figure C-1), located in a high seismic region, Van Nuys, California which is redesigned to satisfy the requirements in ASCE/SEI 7-10 for a new building in Risk Category 2. This case study considers the building to be on individual/spread footings to investigate the shallow foundation provisions of ASCE/SEI 41-17. Note: Some aspects of building may not conform to the requirements of current code but are used for illustrative purposes to highlight use of the foundation provisions in ASCE/SEI 41-17 and compare outcomes with the provisions for new buildings using ASCE/SEI 7.

The gravity system consists of reinforced concrete flat slabs supported by interior concrete columns and perimeter concrete beams Supported by concrete columns. The concrete slabs are 10 inches thick at the second floor, 8.5 inches thick at the third through seventh floors, and 8-inches thick at the roof. The typical framing consists of columns spaced at approximately 20-foot centers in the transverse (north-south) direction and 18 feet 9 inches on centers in the longitudinal direction.

Lateral forces in each direction are resisted by the interior column-slab frames, and by the perimeter column-spandrel beam frames. Interior columns are 18 inches square and exterior columns are 14 inches × 20 inches.

A complete three-dimensional mathematical model is created for this building incorporating the stiffness, strength and deformation characteristics as specified in ASCE/SEI 41.

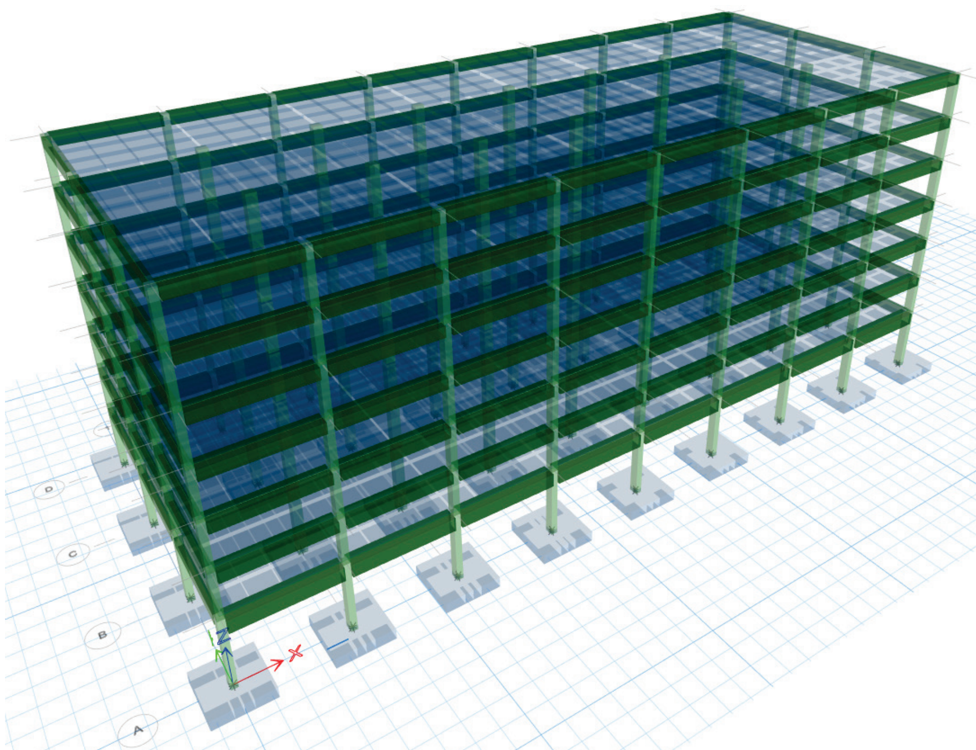


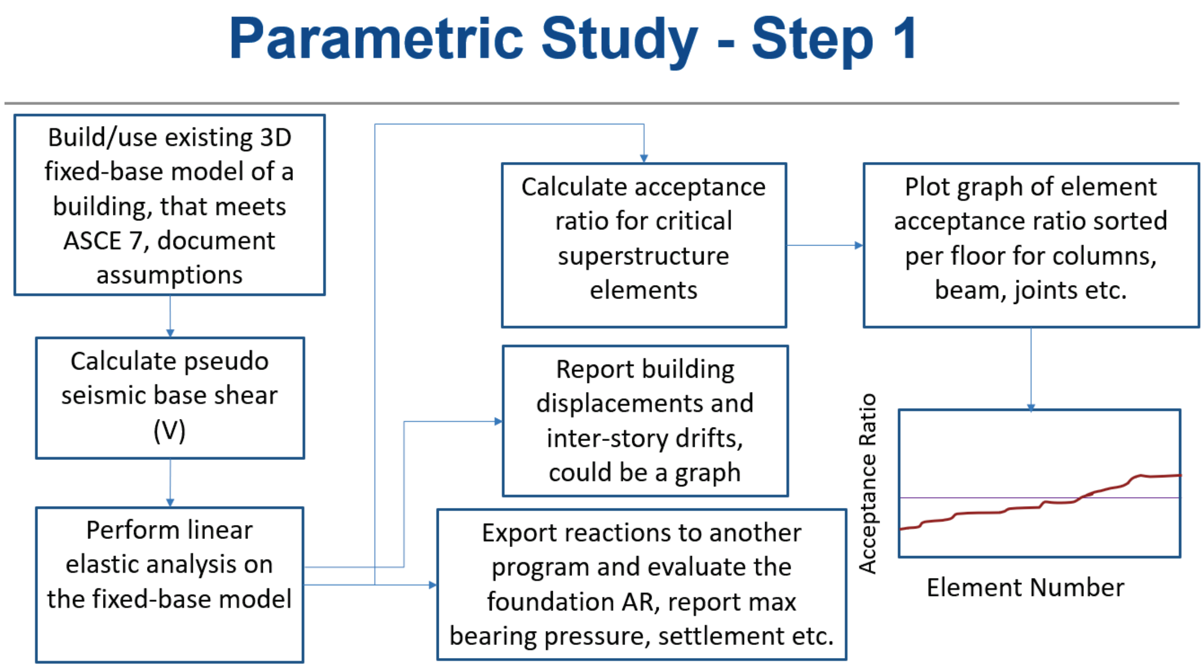
Figure C-1 7-story Reinforced Concrete Building – Archetype Building 2

C.2.2 Planned Approach

Prior to creating the case study models a roadmap was developed to establish a step-by-step approach which was used as a guide, to execute the parametric case studies.

PARAMETRIC STUDY - STEP 1

Create a fixed base mode, with a list a assumptions. Evaluate footing acceptance criteria and record superstructure acceptance ratios.

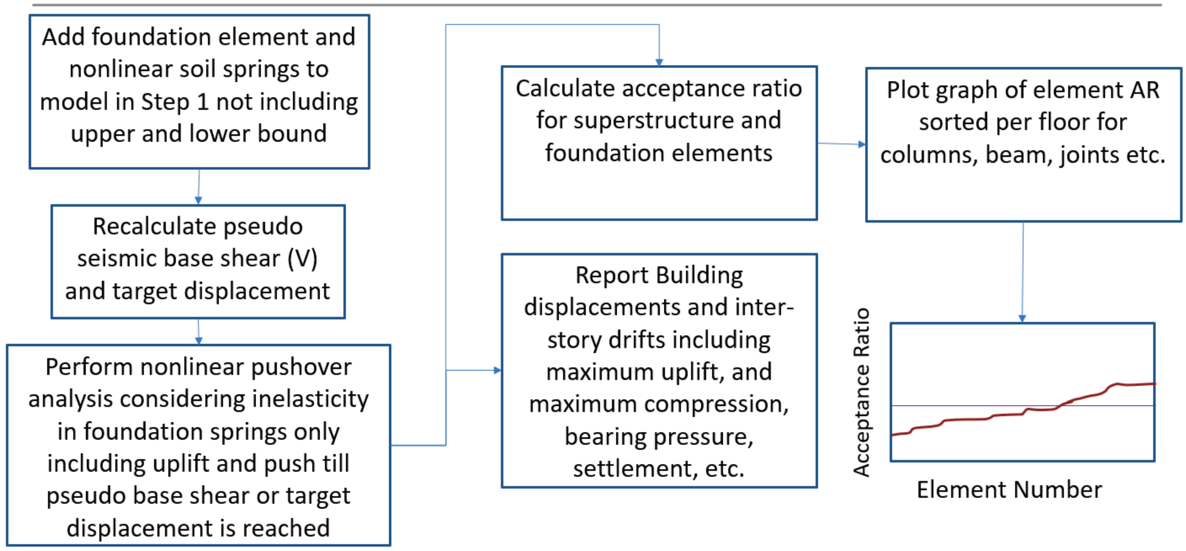


PARAMETRIC STUDY - STEP 2

Create the flexible base model as an extension of the fixed-base model with the associated assumptions. Model the foundation springs using the elastic equations for vertical, horizontal and rocking stiffnesses as required. Note: for this study, the horizontal degree of freedom at the base is fixed, so the effects of horizontal flexibility were not considered.

Evaluate footing acceptance criteria and record superstructure acceptance ratios.

Parametric Study - Step 2



PARAMETRIC STUDY - STEP 3

Compare results from the fixed-base and flexible-base models to study the impact of including foundation flexibility on superstructure response.

PARAMETRIC STUDY – STEP 4

Update the model to include nonlinear properties in the superstructure and permit nonlinear foundation uplift using the expected values for the soil, not the upper and lower bound properties. Repeat processes starting from Step 2.

PARAMETRIC STUDY – STEP 5

Compare the results between the various parametric studies. Perform a critical analysis of the results based on judgements, performance of new buildings, etc. Suggest modifications to procedures based on the comparisons and engineering judgement.

C.2.3 General Modeling Assumptions

- Model is 3D but only loading in the longitudinal direction is considered in the analysis.
- Soil properties are uniform over the footprint of the building. Variable soil properties, or liquefaction potential is not considered.
- For the flexible base option, the soil support for the building is modeled using area springs with assumed soil properties for stiff clay of 0.1 ksi.

- Horizontal degrees of freedom at base are modeled as fixed and deformations due to sliding are not considered.
- Floor diaphragms are modeled as rigid
- Ground motion, mapped values for site class D (Van Nuys, California).
- Column and beam section properties modifiers for stiffness are per ASCE/SEI 41-17 Table 10-5.
- Beam reinforcement is designed to meet the detailing requirements of ACI 318 for qualification as a special reinforce concrete moment frame.
- Column reinforcement is not designed, but moment capacities adjusted to meet the strong column weak beam check

C.2.4 Building Demand Parameters of Interest

The following demand parameters were tracked for comparison between the methods for: A) superstructure, and B) foundation,

A) SUPERSTRUCTURE

- Pseudo seismic force and vertical distribution of forces for LSP
- Building displacement and inter-story drift
- Demands in the Lateral Force Resisting System (LFRS) elements of the superstructure
- Acceptance ratio in the superstructure LFRS elements per story for each element type

B) FOUNDATION

- Bearing pressure
- Acceptance criteria for soil and foundation using the procedures in ASCE/SEI 41

C.2.5 Analyses Performed

The following analyses were performed to evaluate the building superstructure and foundation performance to confirm the fundamental concept: if the building foundation is sufficiently robust and satisfies the acceptance criteria of ASCE/SEI 41 for the desired performance level, the superstructure demands are reasonable. Example: a new building designed using ASCE/SEI 7, should also satisfy the basic safety objective for new buildings (BPON).

Sequence of steps required to corroborate the hypotheses concept:

- Develop a baseline computer model considering foundations as fixed and with member properties that satisfy the requirements of ASCE/SEI 7-10 and ACI 318-14.
- Footings are sized to meet the ASCE/SEI 7-10 demands.
- The foundation footprint and thickness were incorporated into the model for use in the parametric studies when foundations are modeled as flexible.
- Building is analyzed for ASCE/SEI 41 demands and with a performance objective of BPON, or Life Safety (LS) structural performance at BSE -1N and Collapse Prevention (CP) structural performance at BSE-2N
- Results are compared with the baseline ASCE/SEI 7 acceptance criteria.

To execute the parametric case studies, linear and nonlinear analysis procedures were conducted with the following boundary conditions assumed for the foundation:

LINEAR STATIC PROCEDURE (LSP)

- Fixed base model: soil foundation interface, modeled as fixed.
- Flexible base model: foundation supports are modeled as area springs using the following:
    - Soil springs are elastic and resists both tension and compression assuming the same stiffness value.
    - Soil springs are modeled as nonlinear compression only springs, and do not resist tension.

NONLINEAR STATIC PROCEDURE (NSP)

- Fixed base model: soil foundation interface, modeled as fixed.
- Flexible base model: Soil springs are modeled as nonlinear compression only springs, and do not resist tension.

COMPUTER MODELS CREATED:

Four separate computer models were created to represent the base fixity which influenced the elastic period of the building. Nonlinear hinges were assigned and included in the base model which was replicated in the other analysis models. Nonlinear hinge properties in the superstructure were only activated for analysis cases involving analysis using the NSP and were not activated for all the models.

- Model A: Fixed-base Analysis procedures: LSP and NSP.
- Model B: Flexible-base, building on area springs using expected stiffness properties. Analysis procedures:

  - LSP, two case are considered, 1) springs are elastic and resist tension and compression, and 2) springs do not resist tension act as nonlinear compression only springs.
  - NSP, springs do not resist tension act as nonlinear compression only springs.

- Model C: Flexible-base, building on area springs using Lower Bound (LB)* stiffness properties, Analysis procedure: LSP, springs are elastic and resist tension and compression
- Model D: Flexible base, building on area springs using Upper Bound (UB)* stiffness properties, Analysis procedure: LSP, springs are elastic and resist tension and compression.

  * Upper bound and lower bound soil stiffness models were created but results are only presented for the superstructure demands for earthquake demands at the BSE-1N hazard level. Since the results between the bounded values used for the soil did not vary significantly. results presented for the building modeled as a flexible-base are for Model B, which uses the expected soil stiffness values.

ANALYSIS CASES RUN:

For the four models created, which resulted in different periods of the building, various analysis cases were run. These varied from linear to nonlinear where the foundation was modeled either as a fixed-base or a flexible-base. For the models where the building was modeled as a flexible base, two analysis scenarios were considered, one where the soil supports resisted both tension and compression, and one where the soil supports acted nonlinearly as compression only springs. The various analysis cases run on the different models are given below:

Case 1) ASCE/SEI 7-10, for BSE-1N (which is 2/3rd of the MCE$_R$ value using ASCE 7-10) Earthquake demand (**Baseline**)– Model A

Case 2) ASCE/SEI 41-17, LSP, LS structural performance for a BSE-1N Earthquake demand – Model A

Case 3) ASCE/SEI 41-17, LSP, Foundation Springs w/Tension, LS structural performance for a BSE-1N Earthquake demand – Model B

Case 4) ASCE/SEI 41-17, LSP, Foundation Springs w/Tension, LS structural performance for a BSE-1N Earthquake demand – Model C

Case 5) ASCE/SEI 41-17, Foundation Springs w/Tension, LS structural performance for a BSE-1N Earthquake demand – Model D

Case 6) ASCE/SEI 41-17, LSP, Foundation Springs no Tension, LS structural performance for a BSE-1N Earthquake demand – Model B

Case 7) ASCE/SEI 41-17, LSP, CP structural performance BSE-2N Earthquake demand – Model A

Case 8) ASCE/SEI 41-17, LSP, Foundation Springs w/Tension, CP structural performance BSE-2N Earthquake demand – Model B

Case 9) ASCE/SEI 41-17, LSP, Foundation Springs no Tension, CP structural performance for a BSE-2N Earthquake demand – Model B

Case 10) ASCE/SEI 41-17, NSP, LS structural performance for a BSE-2N Earthquake demand – Model B

Case 11) ASCE/SEI 41-17, NSP, CP structural performance for a BSE-2N Earthquake demand – Model A

C.2.6 Baseline Model Designed to ASCE/SEI 7-10 (Model A)

The superstructure and the foundations were designed to the ASCE/SEI 7-10 seismic demands at the BSE-1N seismic hazard level. Column reinforcement was not designed but were assumed to be adequate to satisfy the strong column weak beam requirements. Reinforcement at the base of the first story columns was designed for use in the nonlinear analysis of the building.

DESIGN BASE SHEAR

The design ground motions S_{DS} and S_{D1} for the site are:

S_{DS} = 1.386. and S_{D1} = 0.842g.

And the corresponding design base shear V = C_sW = 1087 kips.

The vertical distribution of forces derived from the base shear calculations using ASCE/SEI 7 are given in Appendix C1.

DESIGN OF SUPERSTRUCTURE ELEMENTS - BEAMS

Beam positive and negative reinforcement were designed to meet the demands for a special reinforced concrete moment frame with R = 8.0. Perimeter beam interior and end beam moment capacities and their corresponding Acceptance Ratios (AR) are given in the Tables C-1 and C-2 below. Building drifts were also checked and met the met the maximum allowable drift limits in ASCE/SEI 7-10.

Table C-1 Beam positive and negative moment capacities and DCRs for Interior Beams

Story	Bottom Reinf, bars	Top Reinf, bars	Depth inches	f'c ksi	M_{bot} Capacity kip-in	M_{top} Capacity kip-in	M_{biot} Demand kip-in	M_{top} Demand kip-in	AR Bot (+ve)	AR Top (-ve)
Roof	2#6	2#8	22	3	1002	1738	430	1292	0.48	0.83
7th	2#7	3#8	22.5	3	1385	2596	731	1954	0.59	0.84
6th	2#8	3#9	22.5	3	1797	3203	1084	2351	0.67	0.82

5th	2#8	3#9 & 1#8	22.5	3	2614	3914	1802	3037	0.77	0.86
4th	2#8	3#9 & 1#8	22.5	3	2614	3914	2109	3343	0.90	0.95
3rd	3#9	3#9 & 2#8	22.5	3	3225	4571	2470	3700	0.85	0.90
2nd	3#9	3#9 & 2#8	30	4	4674	6863	2592	4132	0.62	0.67

Table C-2 Beam positive and negative moment capacities and DCRs for End Beams

Story	Bottom Reinf, bars	Top Reinf. bars	Depth inches	f'_c Ksi	M_{bot} Capacity kip-in	M_{top} Capacity kip-in	M_{biot} Demand kip-in	M_{top} Demand kip-in	AR Bot (+ve)	AR Top (-ve)
Roof	2#7	3#7	22	3	1349	1977	1034	1366	0.85	0.77
7th	2#8	2#8	22.5	3	1798	2614	1563	2026	0.97	0.86
6th	3#8	3#9	22.5	3	2614	3225	2061	2481	0.88	0.85
5th	3#9	2#8 & 2#9	22.5	3	3225	3757	2631	3123	0.91	0.92
4th	3#9	4#9	22.5	3	3225	4124	2860	3429	0.99	0.92
3rd	2#8 & 2#9	3#9 & 2#8	22.5	3	3757	4605	3158	3812	0.93	0.92
2nd	2#8 & 2#9	3#9 & 2#8	30	4	5510	6897	4122	4452	0.83	0.72

DESIGN OF FOUNDATIONS – ISOLATED FOOTINGS

The footings were designed to meet the requirements of ASCE/SEI 7-10 and ACI 318-14 at the BSE 1N seismic hazard level for the worst-case loading from the baseline computer model with the foundations modeled as a fixed-base.

Soil Bearing

The footing acceptance ratio for soil bearing, for positive x-direction loading to the BSE-1N hazard level, assuming an allowable bearing capacity of 3.5 ksf with a 1/3 increase for seismic is shown in Table C-3. The governing load combination is the compression load combination $(1 + 0.14S_{DS})D + 0.5L + 0.7*0.75Q_E$. This assumes a 25% reduction in overturning demands for seismic loading as permitted by ASCE/SEI 7-10 section 12.13.4.

Table C-3 Footing acceptance ratio, soil bearing

Story	Column ID	P Comp (Kip)	M3 (kip ft)	B_f (ft)	L_f (ft)	e (ft)	q_{max} (ksf)	q allowable (ksf)	Acceptance Ratio
STORY1	C1	-112.4	86.0	10	10	0.77	1.6	4.65	0.35
STORY1	C5	-374.8	130.9	10	10	0.35	4.5	4.65	0.97
STORY1	C9	-349.5	129.4	10	10	0.37	4.3	4.65	0.92
STORY1	C13	-349.7	129.6	10	10	0.37	4.3	4.65	0.92
STORY1	C17	-349.8	129.6	10	10	0.37	4.3	4.65	0.92
STORY1	C21	-349.6	129.6	10	10	0.37	4.3	4.65	0.92
STORY1	C25	-351.4	129.4	10	10	0.37	4.3	4.65	0.92
STORY1	C29	-347.5	130.9	10	10	0.38	4.3	4.65	0.92
STORY1	C33	-270.8	86.0	10	10	0.32	3.2	4.65	0.69

LC = $(1 + .14S_{DS})D + 0.5L + 0.7*0.75Q_E$

q allowable = 4.65 ksf with 1/3 increase for seismic

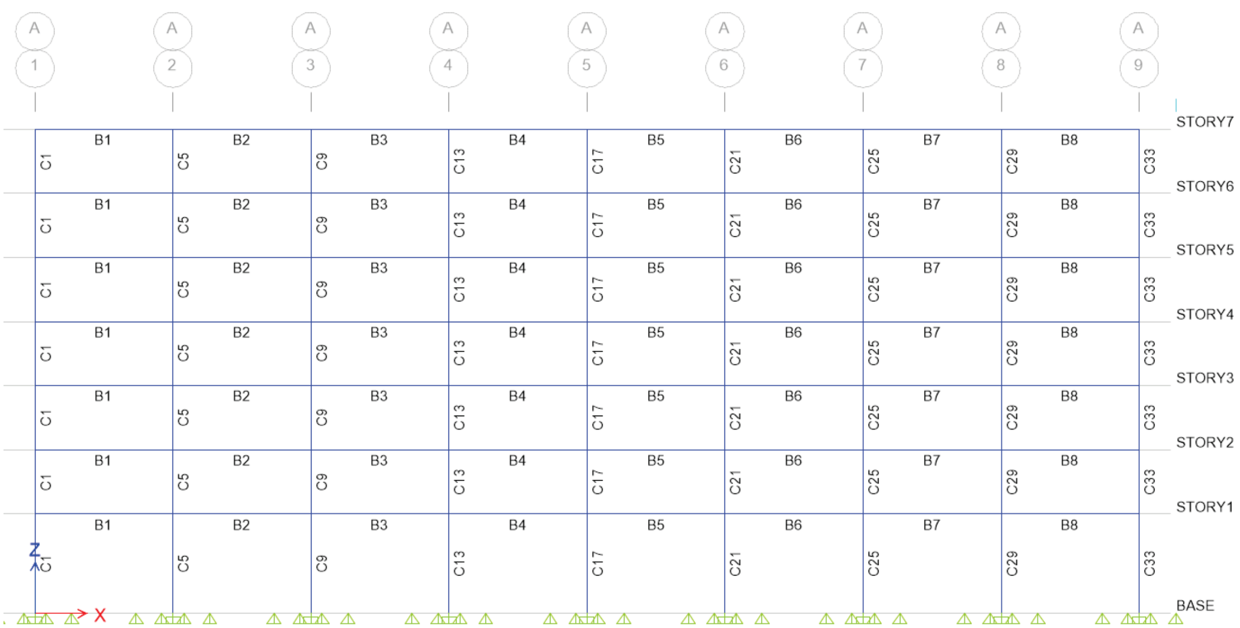


Figure C-2 Key plan of column and beam IDs along grid line A.

Structural Footing

The maximum axial load and moment, at the BSE-1N hazard level, on the footing was at first interior footing for the load combination $(1.2 + 0.2S_{DS})D + 0.5L + E$ and includes the 25% reduction in overturning as permitted by ASCE/SEI 7-10. The corresponding soil pressure distribution under the footing the applied axial load and moments is shown in Figure C-2.

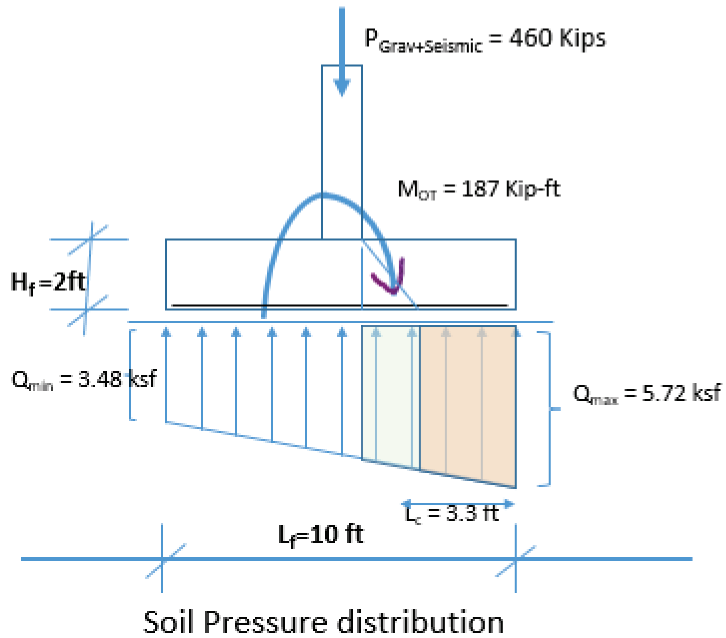


Figure C-3 Soil pressure distribution under the footing for the governing load combination

Moment at critical section

Moment demand on the 10 ft × 10 ft × 2 ft footing is calculated at the face of the 14 × 20 column for the soil distribution shown in Figure C-3.

Q_{min} at face of the column = 4.73 ksf

Dividing the soil pressure profile into a rectangle and a triangle, the moment at the face is the sum of the moments from each soil pressure block is calculated as:

Moment at column face

M_u = (4.73 ksf)(10 ft){5 ft –(14 in)/(2)/(12)}^{2}/2 + (5.72 ksf - 4.73 ksf)(10 ft){5 ft –(14 in)/(2)/(12)}^{2}/3

M_u =526 kip-ft

Use 10 #9 bars, Moment Capacity ϕM_n= 0.9(10 in^2)(60 ksi)(20 in – (1.47 in)/2)/(12 in) = 866 kip -ft

AR = 526/866 = 0.60 < 1.0 OK

Shear at critical section

Shear demand is calculated at a distance "*d*" the face of the 14 × 20 column.

Q_{min} at distance *d* from face of the column = 5.10 ksf

Dividing the soil pressure profile into a rectangle and a triangle, the shear at the critical section is the sum of the moments from each soil pressure block is calculated as shown below:

Shear demand at critical section:

V_u = (5.10 ksf)(10 ft){5 ft –(14 in)/(2)/(12) – (20 in)/12} + (5.72 ksf - 5.10 ksf)(10 ft){5 ft – (14 in)/(2)/12 – (20 in)/12}/2

V_u =149 kips

Shear capacity at the critical section, ϕV_n= 0.85(2(4000 psi)^0.5(10 in ×12 in)(20 in))/1000 lbs =258 kips

AR = 149/258 = 0.58 < 1.0 OK

Check Punching shear

b_o = 2((14 in + 2*(24 in))+(20 in + 2*(24 in))) = 260 in

Shear capacity = ϕV_c = (0.85)(4) $\sqrt{(f'_c)}$ b_o d = 0.85(1315) kips

AR = $V_u/\phi V_c$ = 460/1118 = 0.41 < 1.0 OK

C.2.7 Linear Static Procedures (ASCE/SEI 41-17)

The ground motions for the site at the BSE-1N, and BSE-2N seismic hazard levels were:

BSE-1N

S_{XS} = 1.386. and S_{X1} = 0.842g.

BSE-2N

S_{XS} = 2.079 and S_{X1} = 1.263g.

PSEUDO SEISMIC FORCE DEMANDS FOR LSP

The pseudo seismic force demands at the BSE-1N and BSE-2N hazard levels are given in Table C-4, Additional details and the vertical distribution of forces used for each model is given in Appendix C1.

Table C-4 Pseudo Seismic Force Demands for Each of the Models (kips)

Seismic Hazard Level	Model A Fixed Base	Model B Flexible base k_{sv} = 0.1 kci	Model C Flexible base (LB) k_{sv} = 0.05 kci	Model D Flexible base (UB) k_{sv} = 0.2 kci
BSE-1N	5348	5177	5056	5248
BSE-2N	8022	7765	7583	7872

C.2.8 Nonlinear Static Procedures (ASCE/SEI 41-17)

TARGET DISPLACEMENT

The target displacement δ_t used for the nonlinear analysis procedure calculated in accordance with ASCE/SEI 41-17 equation 7-28 at the BSE-2N seismic hazard level, for an assumed building period of 1.8 seconds is shown in Figure C-4 was 32 inches. Additional details are given in Appendix C2. The seismic hazard at BSE-2N was selected as that is maximum displacement required at the CP structural performance level.

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \qquad \text{(ASCE/SEI 41-17 Eq. 7-28)}$$

Target Displacement - Calculation

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g$$

C_0	1.44	Table 7-5
C_1	1	$T_e > 1$ s
C_2	1	$T_e \geq 0.7$ s
S_a	0.7	5% Damped spectrum, BSE-2N
T_e	1.80	Assumed

Target Displacment

Parameter	Modal Load Pattern
	inches
Roof Disp. δ_t=	31.97

Figure C-4 Target displacement calculation at the BSE-2N earthquake hazard level

HINGE PROPERTIES AND ASSIGNMENTS

Beam Moment Hinge Properties

Beam moment hinge properties are assigned based on the beam property tables given in Tables C-1 and C-2. The IO, LS and CP limits are taken from Table 10-7 of ASCE/SEI 41-17. The yield moment is calculate using the yield capacity f_y of the steel reinforcement of 60 ksi, and the ultimate is taken as 1.25 f_y achieved at the CP strain limit. The moment capacity is gradually decreased to a ultimate stain of 0.07 radians.

Beam Shear Hinge Properties

The superstructure shear reinforcing is assumed as conforming and meets the requirements of ACI 318-14 for a special moment resisting frame, therefore there should be no shear failures in the beams. For this reason, shear hinges were not modeled in this case study. A sample of the beam hinge property for a 6th floor interior beam is shown in Figure C-5.

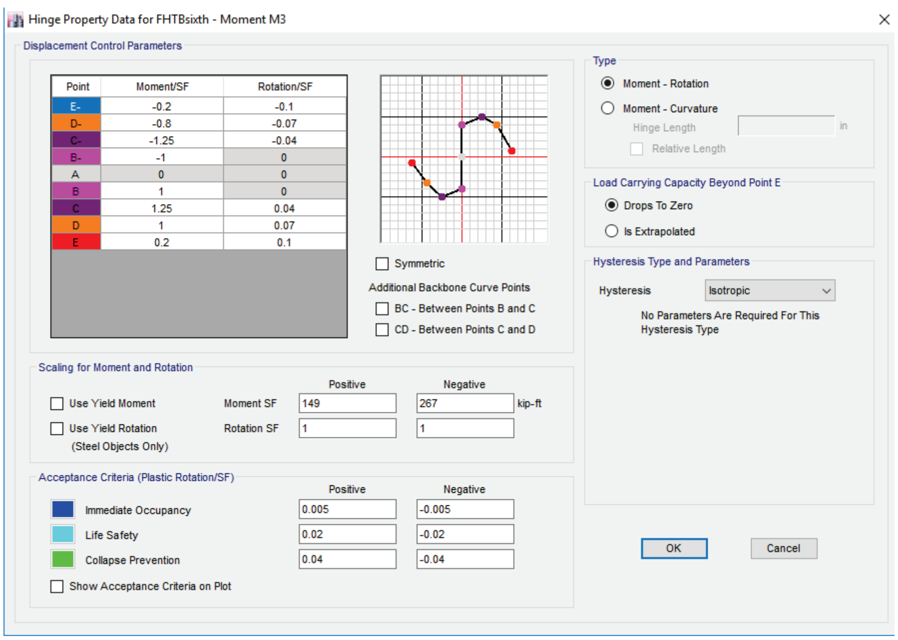


Figure C-5 Sample beam hinge property (6nd floor interior beam)

Column Moment Hinge Properties

Column hinge properties are derived from the beam moment capacities and the strong column weak beam criteria is satisfied. The actual capacities based on a steel reinforcement area and axial load was not done in this case study. Degradation of column moments is not considered in this analysis as the focus was to estimate the maximum demands that can be delivered to the foundations. A sample of the column hinge property for a 4th floor perimeter interior column is shown in Figure C-6. Since the columns are designed to satisfy the strong column weak beam requirement, only column yielding at the base is expected to occur, but the hinge properties for columns are assigned for completeness.

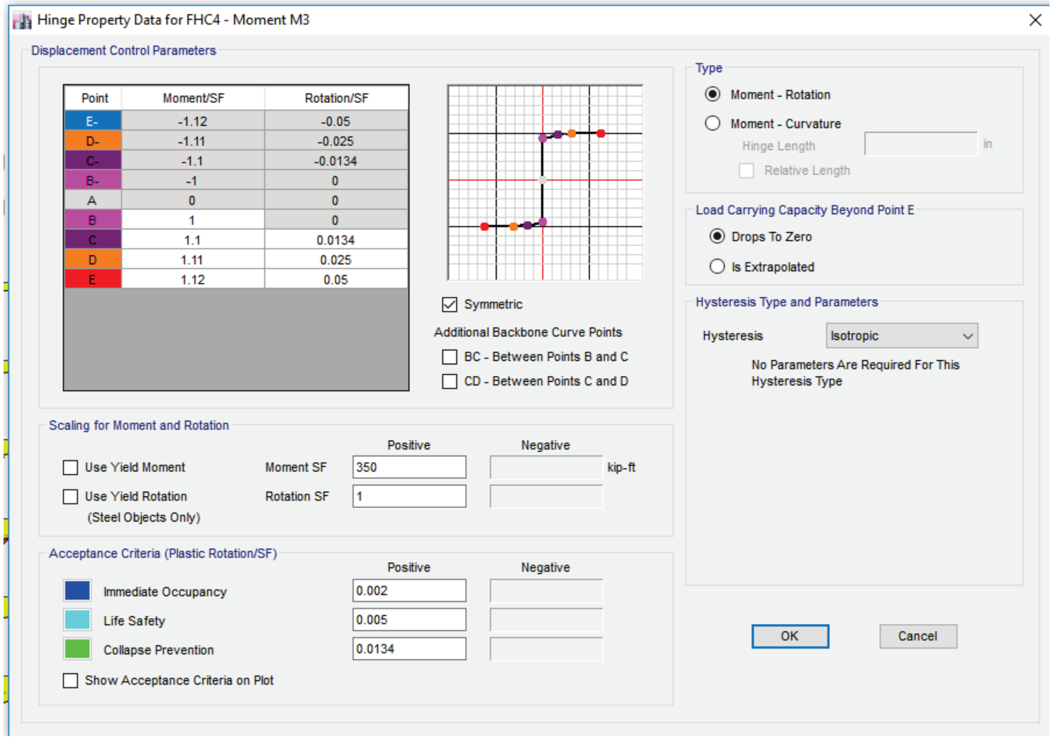


Figure C-6 Sample column hinge property (Fourth floor perimeter interior column)

C.3 Comparative Results from Parametric Study – Archetype 2

The superstructure and foundation demand and acceptance criteria were compared for the analysis cases run as described in Section C.2. Comparisons are shown for each of the parameters of interest tracked for both the superstructure and the foundations.

C.3.1 Comparisons of the Superstructure Demand Parameters

PSEUDO SEISMIC FORCE DEMANDS FOR LSP

A comparison of the pseudo seismic force demands at the BSE-1N and BSE-2N hazard levels are given in Table C-5,

Table C-5 Comparison of Pseudo Seismic Force Demands (kips)

Seismic Hazard Level	Model A Fixed Base	Model B Flexible base k_{sv} = 0.1 kci	Model C Flexible base (LB) k_{sv} = 0.05 kci	Model D Flexible base (UB) k_{sv} = 0.2 kci
BSE-1N	5348	5177	5056	5248
BSE-2N	8022	7765	7583	7872

Observations:

For this building and direction of loading (longitudinal) the variation in base shear considering the lower bound and upper bound soil stiffness values is less than four precent, and the difference between the fixed base model and the model using the upper bound stiffness is approximately two percent. Therefore, the impact on the pseudo seismic force demands between the various models with different foundation modeling base stiffnesses for this archetype building is minimal.

Conclusion:

For moment frame buildings where the LFRS of the superstructure is relatively flexible compared to the LFRS of other building types such as shear walls or braced frames, bounding on stiffness appears to have little impact on the overall demands to the structure for evaluations using LSP,

PUSHOVER CURVE COMPARISONS USING NSP:

The static pushover cure and the hinge pattern at the target displacement from the NSP analysis is shown in Figure C-7 and Figure C-8.

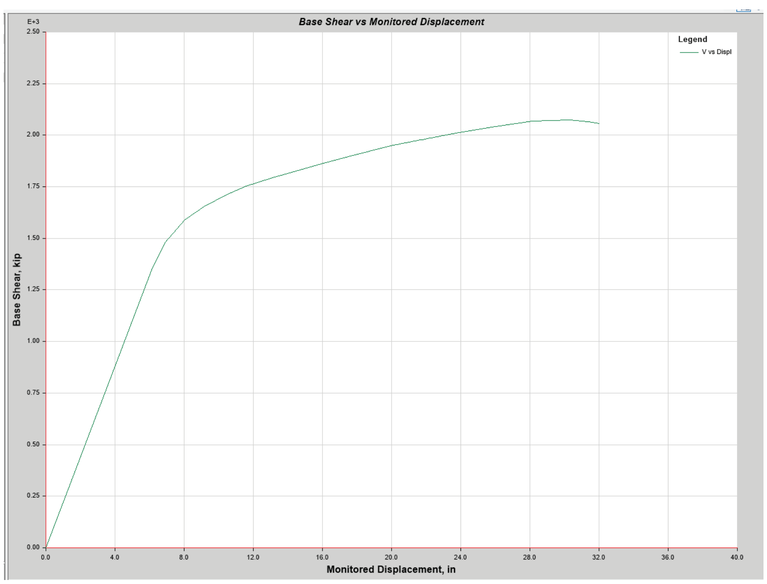
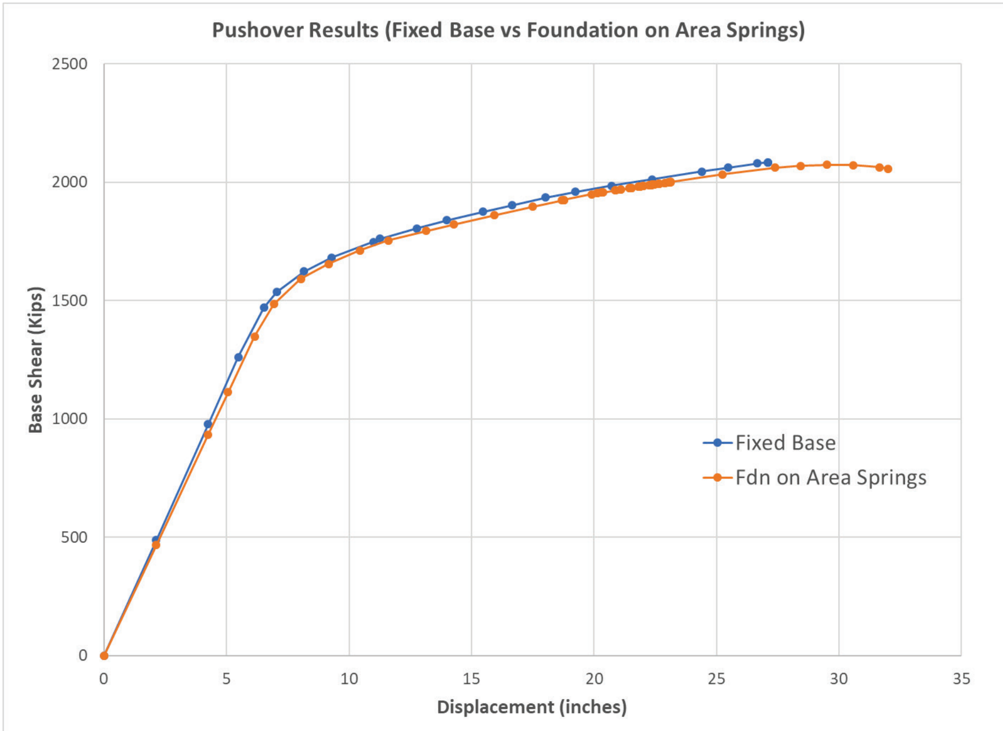


Figure C-7 Static pushover curve to the target displacement– BSE-2N of 32 inches



Target Displacement – BSE-2N = 32 inches

Figure C-8 Push over curve comparisons with foundations modeled as a fixed-base and flexible-base.

Observations:

There is very little difference in the shape of the pushover curve whether the building is modeled as fixed-base or a flexible-base. There was no convergence for the fixed-base analysis beyond 27 inches thus indicating excessive damage beyond the ductility capacity of superstructure elements. Modeling the building as flexible base with soil deformation capabilities permitted the building to be displaced to the desired target displacement. However, it should be noted that from the failure hinge pattern, the building did not satisfy its desired performance objective, regardless of whether the foundations were modeled as a fixed-base or a flexible-base.

Conclusions:

For ductile buildings designed with a response modification factor, R of 8.0 and where superstructure yielding is expected to occur prior to when excessive foundation deformations occur either by foundation yielding or bearing capacity failure, modeling the building as fixed-base or a flexible-base has minimal impact on the performance outcomes from the analysis.

BUILDING DISPLACEMENTS (DRIFTS) COMPARISON

A comparison of superstructure building displacements for force demands at the BSE-1N level is given below in Table C-6. The target displacement for the NSP corresponds to an assumed effective fundamental period T_e = 1.8 seconds. Had the effective period been chosen as 1.95 seconds, the buildings drifts from the fixed-base analysis and the nonlinear push would have been about the same. Drifts are slightly higher if the analysis is elastic, and uplift is prevented. However, if the uplift is permitted for the soils for the nonlinear case there is about a 25% increase in overall building displacements.

Table C-6 Drift Summary for Various Models – BSE 1N Demand

Foundation Fixity & Analysis Type	Spring no Tension, T_{eff}= =1.8 s k_{sv} =0.1kci (NSP)	Fixed-Base (LSP)	Spring takes Tension k_{sv} = 0.05kci, (LSP)	Springs take Tension k_{sv} = 0.1 kci (LSP)	Spring No Tension k_{sv} = 0.1 kci (LSP)
Period (sec)	1.626	1.574	1.665	1.626	1.626
Base shear (Kips)	1976	5348	5056	5177	5177
Story	Displacement (in)				
7	21.6	23.7	24.6	24.2	30.5
6	19.7	21.1	22.1	21.7	27.5
5	17.1	18	19	18.6	23.7
4	13.8	14.3	15.3	14.9	19.4
3	10.1	10.6	11.7	11.2	15.1
2	6.3	7.1	8.2	7.7	10.9
1	3.0	3.8	4.9	4.5	6.8

A comparison of superstructure building displacements for force demands at the BSE-2N level is given below in Table C-7. The superstructure displacements at each story for the various models are compared with the displacements from the NSP. The displacement demands at each story for the fixed-base and flexible-base where the soil resists tension, track well with the superstructure displacements from the nonlinear static procedure. The displacements where soil does not resist tension are at many stories over twice as high as the displacements from the NSP and is more pronounced at first floor level.

Table C-7 Drift Summary for Various Models – BSE-2N Seismic Hazard Level.

Foundation Fixity & Analysis Type	Spring No Tension, Teff = 1.8s k_{sv} =0.1kci (NSP)	Fixed-Base (LSP)	Spring take Tension k_{sv} = 0.05kci (LSP)	Springs take Tension k_{sv} = 0.1 kci (LSP)	Spring No Tension k_{sv}= 0.1 kci (LSP)
Period (sec)	1.626	1.574	1.665	1.626	1.626

Base shear (Kips)	2071	8022	7583	7765	7765
Story	Displacement (in)				
7	32.0	35.56	36.94	36.36	63.85
6	28.6	31.84	33.21	32.64	57.65
5	24.6	27.04	28.47	27.87	50.37
4	19.7	21.45	22.99	22.34	42.25
3	14.3	15.93	17.56	16.87	34.14
2	8.9	10.58	12.3	11.57	26.05
1	4.3	5.74	7.38	6.69	17.63

Conclusions/Recommendation:

As the pseudo seismic force demand increases, the analysis case where the building is modeled as elastic and the soils are modeled as nonlinear where soils do not resist tension gives the greatest departure in displacement demands from the NSP results. The displacements where the foundations are modeled as a fixed-base, or a flexible-base are consistent with the displacements from the nonlinear static pushover analysis. Permitting uplifting foundation springs with an elastic superstructure is therefore not recommended.

DEMANDS ON THE LATERAL FORCE RESISTING SYSTEM (LFRS) ELEMENTS OF THE SUPERSTRUCTURE

Superstructure Demand Comparisons for Soil Stiffness Bounding Provisions

Results from the flexible base model, using LSP for earthquake demands at the BSE-1N seismic hazard level, were compared with demands from the baseline model and the NSP. For this comparison the soil and superstructure were modeled as elastic i.e. soil resists tension. The results from the study for each element of the superstructure, are presented in Figure C-9 through Figure C-12 starting with the top story on the left and the bottom story on the right. The loading for each case was for demands applied in the positive x-direction, the longitudinal direction of the building. Observation of the column axial loads at the bottom story shown in Figure C-10, shows that the higher axial load demands from the column to foundation correspond to the fixed base analysis. Foundation demands from the columns are minimum when lower bound spring stiffness are used.

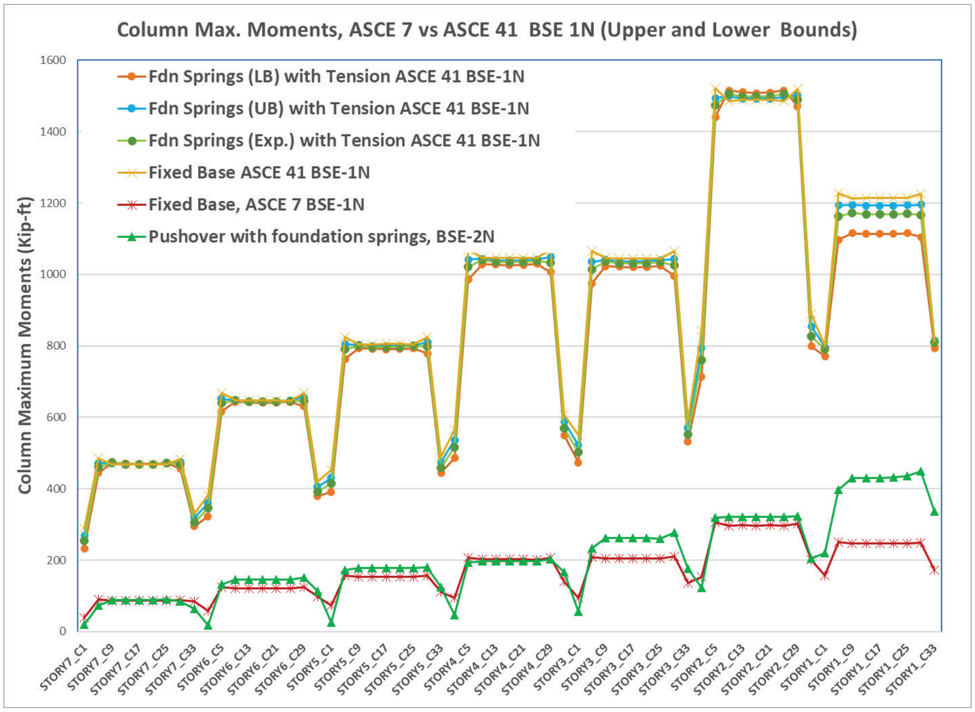


Figure C-9 **Column moments per story from left to right, starting with top story on the left to bottom story on the right.**

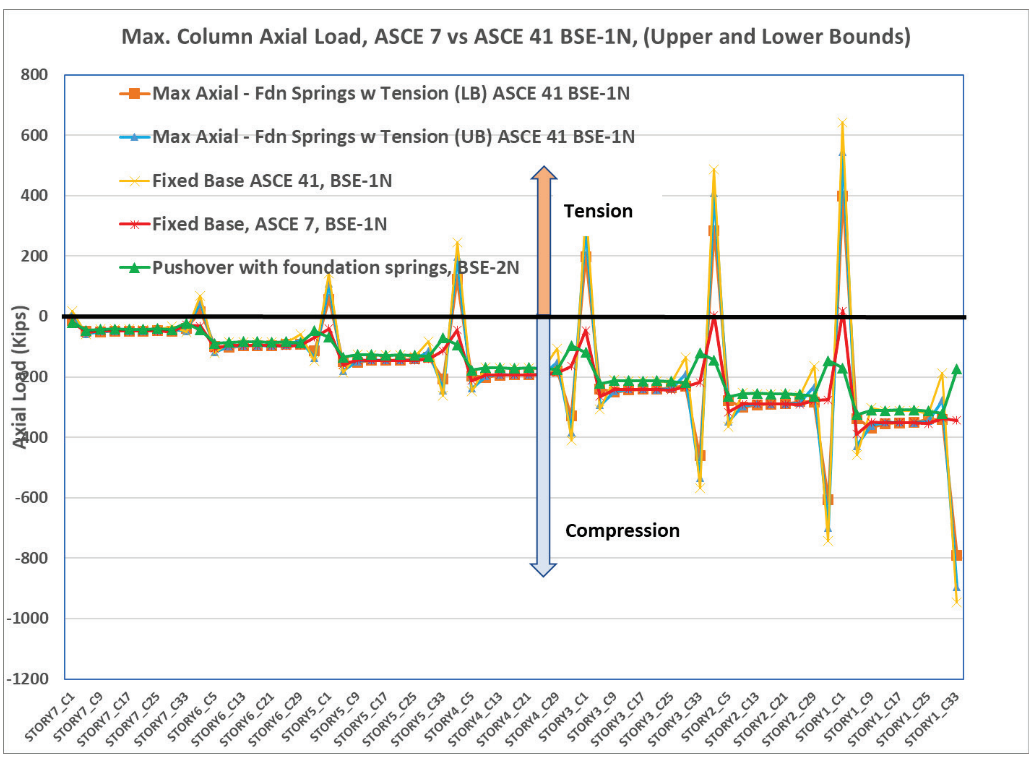


Figure C-10 **Column axial load per story from left to right, starting with top story on the left to bottom story on the right.**

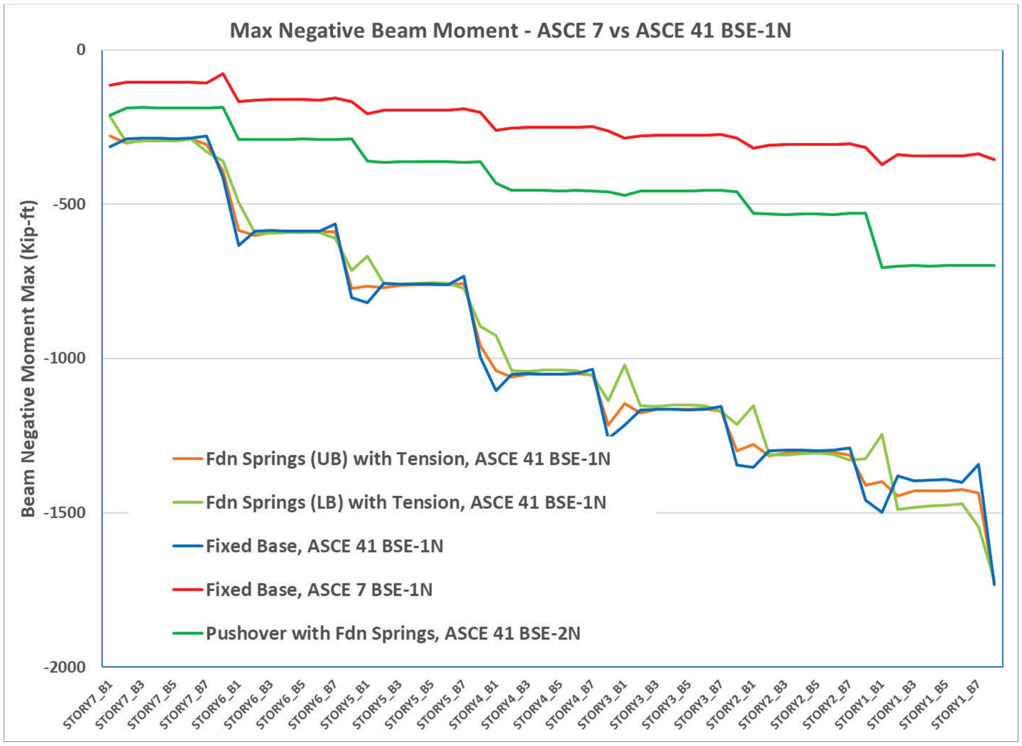


Figure C-11 Soil pressure distribution under the footing is a rectangle and a triangle.

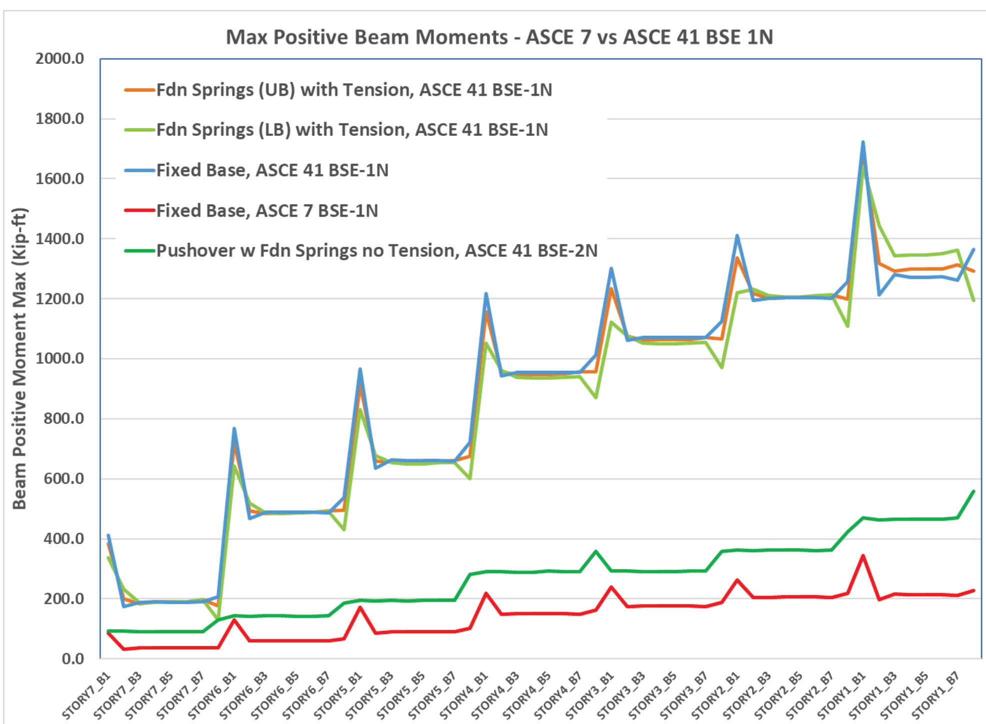


Figure C-12 Soil pressure distribution under the footing is a rectangle and a triangle.

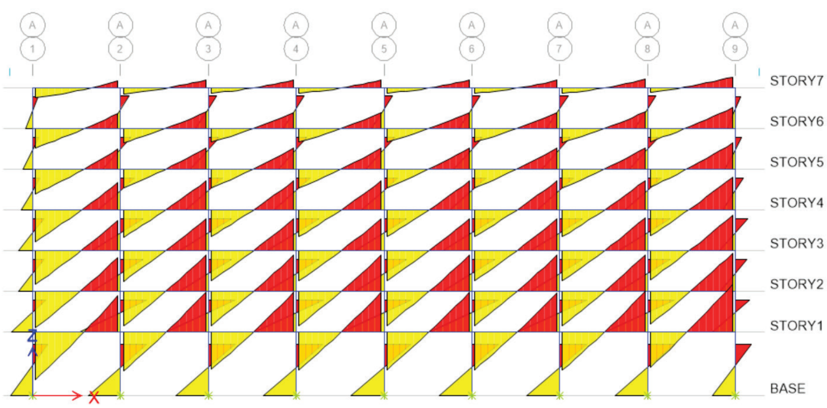
Observations and Conclusions:

The results show there is very little difference in the superstructure demands when upper bound and lower bound stiffness properties are used for the building modeled as a flexible base. Therefore, the results from the subsequent studies will only show comparisons where the flexible base models use expected properties for the soil.

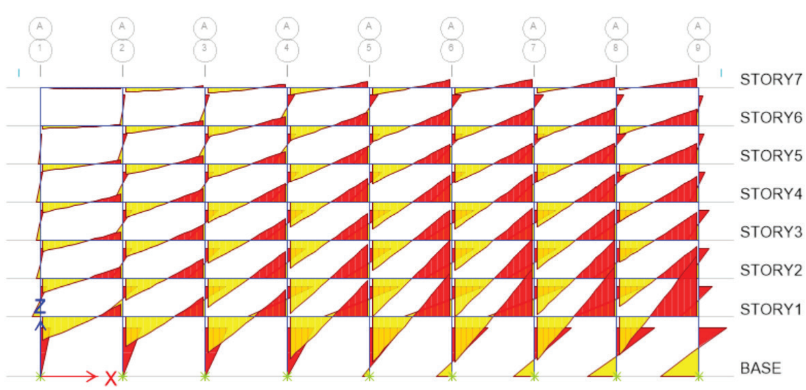
SUPERSTRUCTURE DEMAND COMPARISONS BETWEEN THE LSP AND NSP

Beams Moments

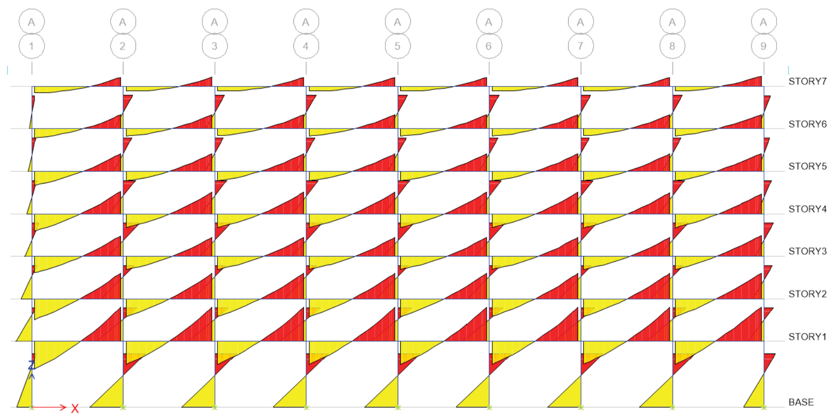
The superstructure moment pattern for X- direction loading for the tension load commination $0.9D + Q_E$, where soil takes tension and where soil does not resist tension, using LSP are compared with the moment pattern from the nonlinear static push case at the BSE-2N earthquake hazard level as shown in Figure C-13. The nonlinear static push shows lower demands for the beam positive moments as these are significantly less than the negative moment capacities. The beam moment demands from the LSP where soil resists tension, are fairly symmetric. This is not the case where foundation uplift is not restrained.



Soil Takes Tension



Soil No Tension



Soil No Tension – NSP

Figure C-13 Frame moment patterns when soil resists tension in the LSP and when soil does not resist tension for the NSP.

The beam moment demands in the structure from the baseline model, Case 1 are compared with the demands for Cases 3, 6, 8, 9 and 10. The demands are plotted per story from left to right and from floor 7 at the left to the first floor at the right in Figures C-14 and C-15 corresponding to beam negative or top moments and beam positive or bottom moments.

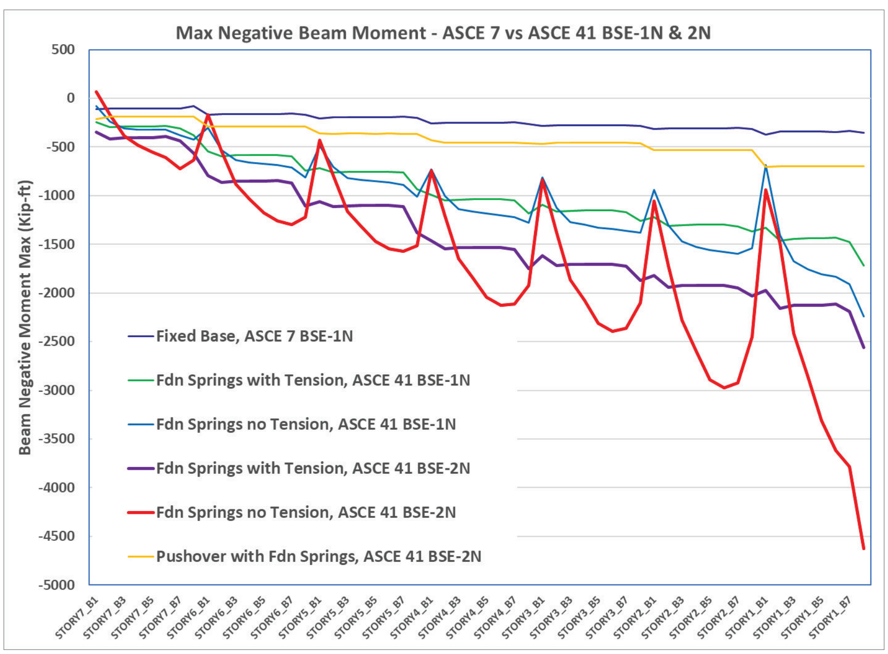


Figure C-14 Beam negative moments

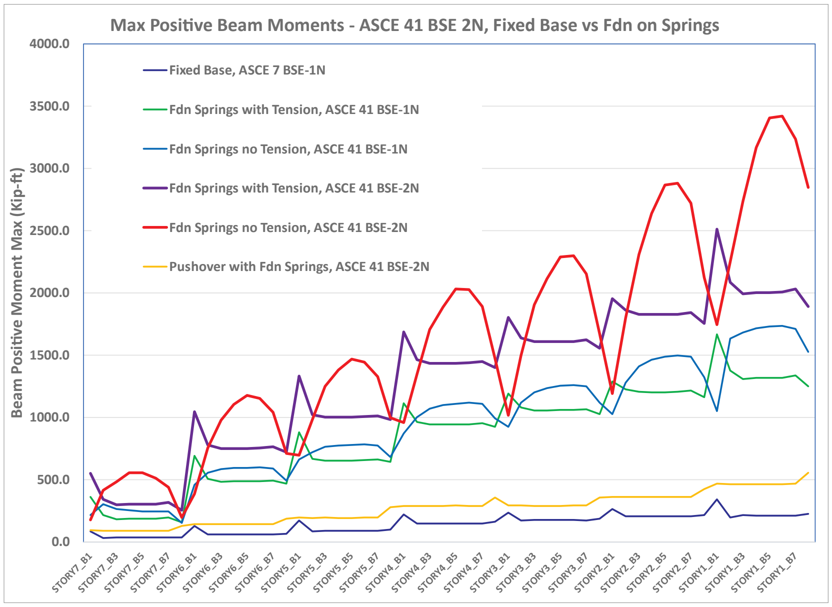


Figure C-15 Beam positive moments

Column Moments

The column moments are plotted similar to the beam moment, but for clarity, only for Cases1, 8, 9 and 10 as shown in Figure C-16,

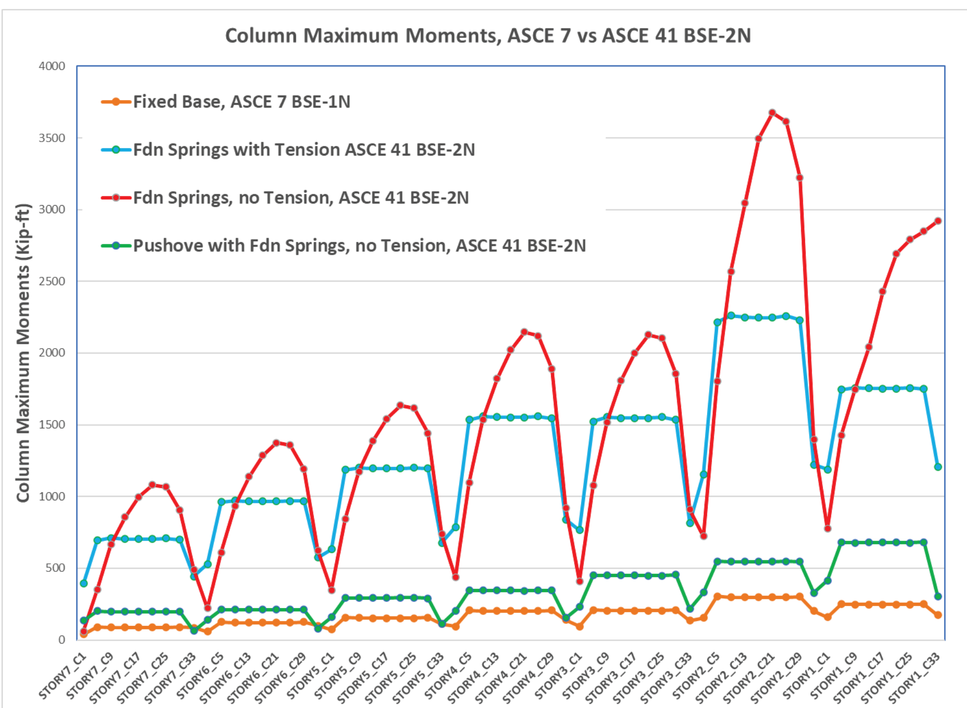


Figure C-16 Column moments

Column Axial Loads

The superstructure column axial loads for x- direction loading at the BSE-2N earthquake hazard level for the load combination where gravity and seismic are counteracting for the various analyses performed are shown in the following figures, Figure C-17 through Figure C-21. These include the fixed-base and flexible-base analysis where the soil springs resist tension, do not resist tension and for the nonlinear static procedure. From the results the column axial loads are the highest for the fixed-base analysis. It also shows a large net tension demand in the end columns which does not materialize in the nonlinear analysis model. The resulting column axial load pattern where the superstructure is elastic and the soil springs do not resist tension, shows a gravity load shift in the direction of overturning. Where the lateral force resisting system of the superstructure is flexible, such as in this example, this pattern is unrealistic.

Figure C-17 Column Axial Load, Load Combination (LC) - 0.9D + E (BSE-2N), Fixed-Base

Figure C-18 Column Axial Load, LC - 0.9D + E (BSE-2N), Soil resists Tension, k_{sv} = 0.05 kci,

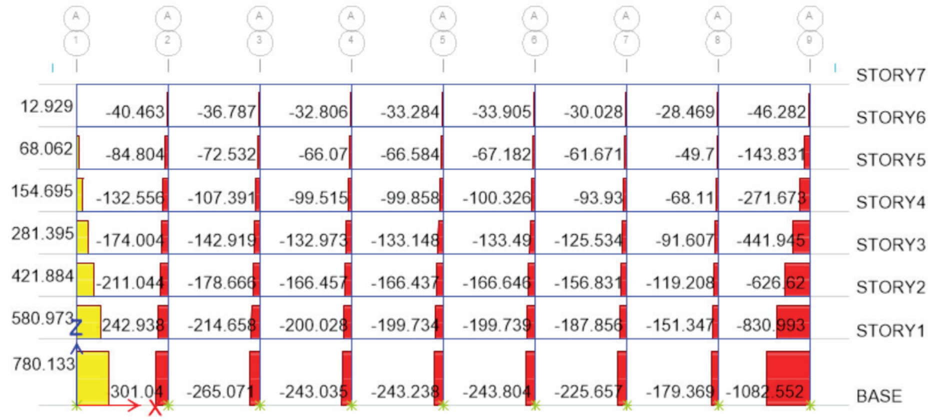


Figure C-19 Column Axial Load, LC – 0.9D + E (BSE-2N), Soil resists Tension, k_{sv} = 0.1 kci,

Figure C-20 Column Axial Load, LC - 0.9D + E (BSE-2N) Soil no Tension, k_{sv} = 0.1 kci,

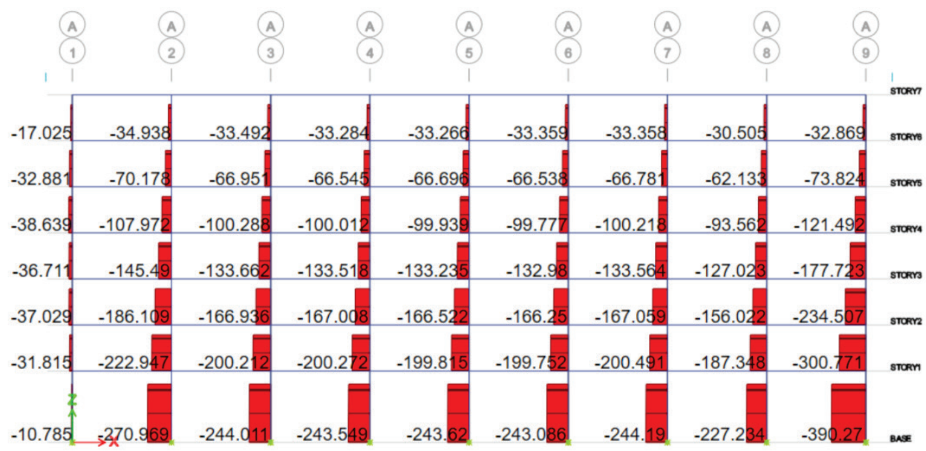


Figure C-21 Column Axial Load, LC – 0.9D + E (BSE-2N), NSP, k_{sv} = 0.1 kci,

A plot of the axial load pattern over the height of the building plotted from the top story on the left to the first story on the right (Figure C-22), shows the large spikes in axial load in the end columns.

These spikes do not occur for the axial loads from the NSP because of yielding in the superstructure elements.

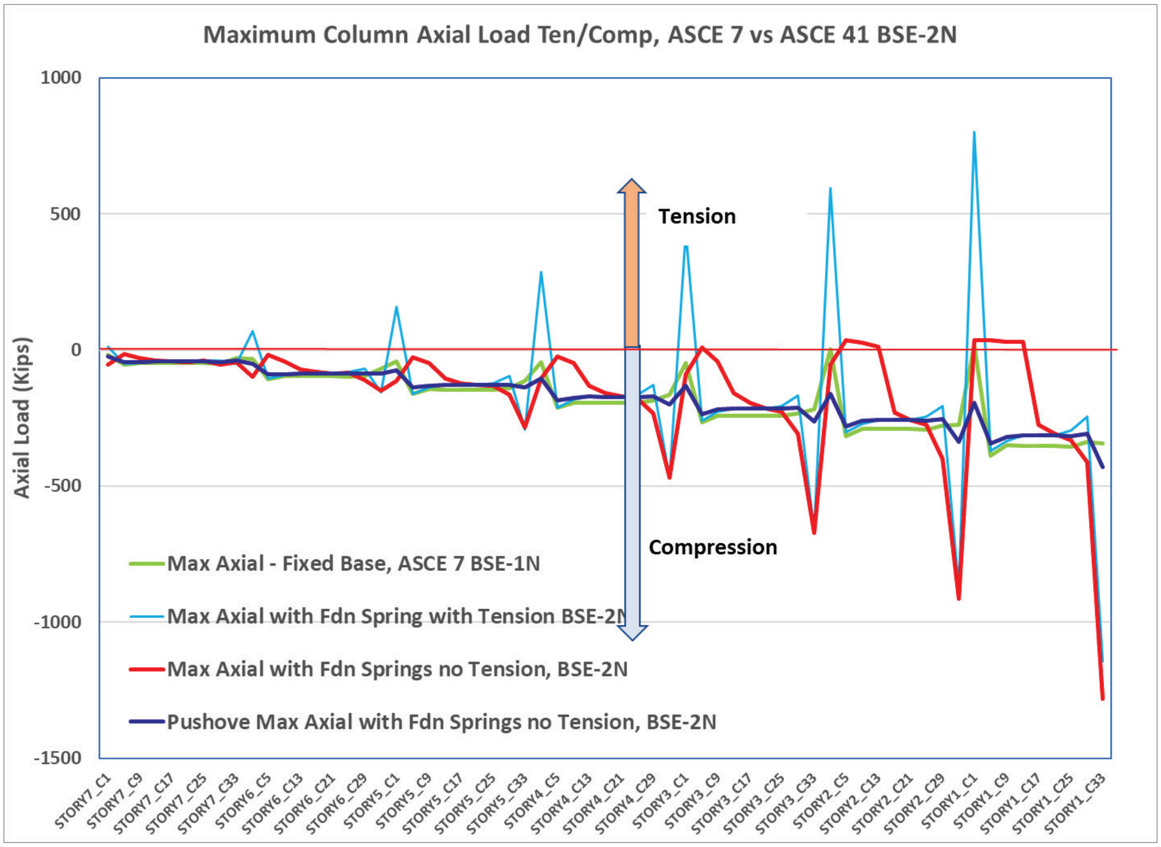


Figure C-22 Column axial loads

Observations and Conclusions:

The results from the various cases clearly show that for this archetype building, the results are consistent between the fixed-base, and the flexible-base models where the soil resist tension. When uplift is not restrained in the flexible base model for LSP, the demands are inconsistent and do not align with the demands from the NSP. Fixed base models give the highest overturning demands on the foundation, both tension and compression. The high column tension loads observed are inconsistent with the results from the NSP. Flexible base models result in lesser overturning seismic demands in the end columns and may be useful in justifying that the building meets the desired performance objective without performing a nonlinear analysis using NSP.

SUPERSTRUCTURE ACCEPTANCE RATIOS IN THE LFRS ELEMENTS

Comparison of Superstructure Acceptance Ratios: LSP - Beams

The Acceptance Ratios (AR) between the various cases run for beam negative and positive moments are given in Figure C-23 through Figure C-30. Results from the fixed-base or flexible-base analysis where soil takes tension seems to give the best approximate pattern with the baseline. Modeling the superstructure as elastic with nonlinear foundation compression only springs gives a different distribution of acceptance ratios with much higher maximums. Results when compared to acceptance ratios from the nonlinear static procedures shown in the next section confirm that modeling the superstructure as elastic with nonlinear compression only springs give incorrect acceptance ratios for the superstructure elements. The hinge patter from the NSP is shown in Figure C-31. Additional information on results from the NSP is given in Appendix C3.

Figure C-23 ARs Beam Negative Moment, ASCE 7: BSE-1N, - Fixed Base (Baseline)

Figure C-24 ARs Beam Negative Moment, ASCE 41: BSE-2N, - Fixed Base (CP)

Figure C-25 AR Beam Negative Moment, ASCE 41: BSE-2N - Soil Takes Tension (CP)

Figure C-26 AR Beam Negative Moment, ASCE 41: BSE-2N – Soil Compression only (CP)

Figure C-27 ARs Beam Positive Moment ASCE 7: BSE-1N – Baseline

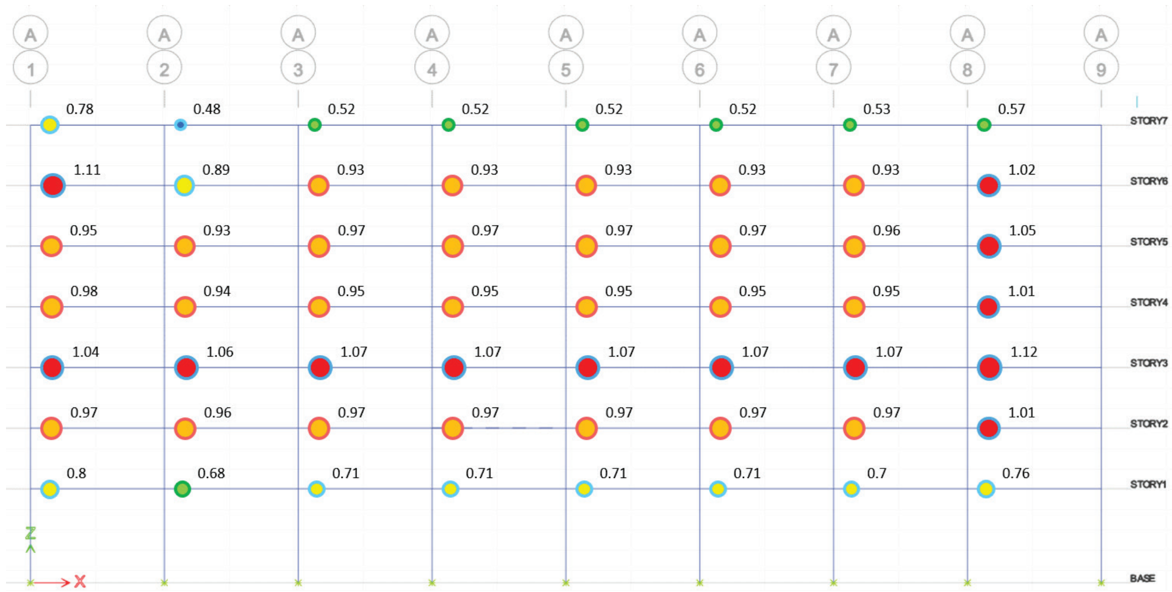


Figure C-28 ARs Beam Positive Moment ASCE 41: BSE-2N – Fixed-Base (CP)

Figure C-29 ARs Beam Positive Moment, ASCE 41: BSE-2N - Soil Takes Tension (CP)

Figure C-30 ARs Beam Positive Moment, ASCE 41: BSE-2N – Compression only (CP)

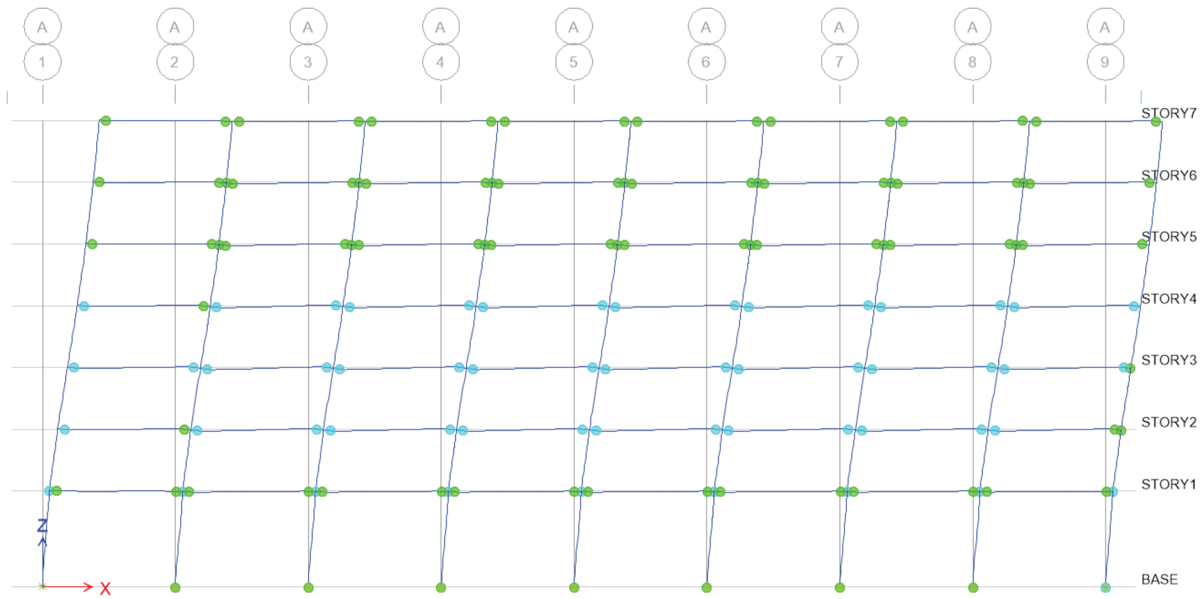


Figure C-31 Hinge pattern at Target Displacement – BSE-2N of 32 inches

Observations/Conclusions

From the results the beam acceptance ratios (AR) for the fixed base or flexible base condition where soils resist tension give reasonable results with the baseline. The AR pattern is also consistent with the results from the NSP, but the comparison is not that obvious as the results from the NSP are expressed in terms of performance levels, not as quantitative values.

C.3.2 Comparisons of the Foundation Demand Parameters

SOIL BEARING PRESSURE COMPARISONS AS A MEASURE OF FOUNDATION ACCEPTANCE

Soil Bearing Pressures - LSP

For the models with flexible-base foundations, the soil pressure distribution in the foundations for ASCE/SEI 41-17 demands at the BSE-1N and BSE-2N earthquake hazard levels considering the springs as elastic (both tension and compression) and nonlinear as compression only, are as shown in Figure C-32 through Figure C-35.

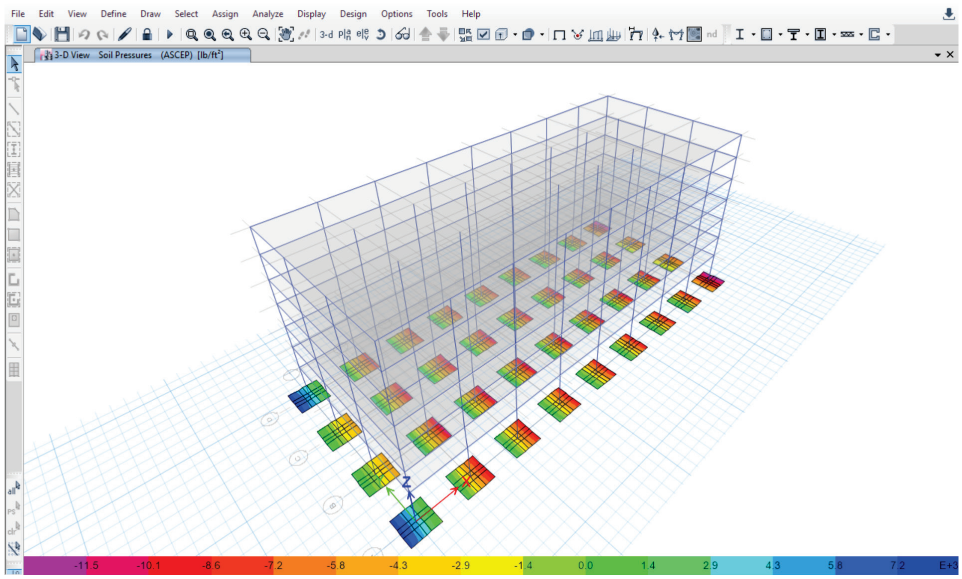


Figure C-32 Soil takes Tension Eq Hazard level BSE-1N Max pressure = 12.6 ksf

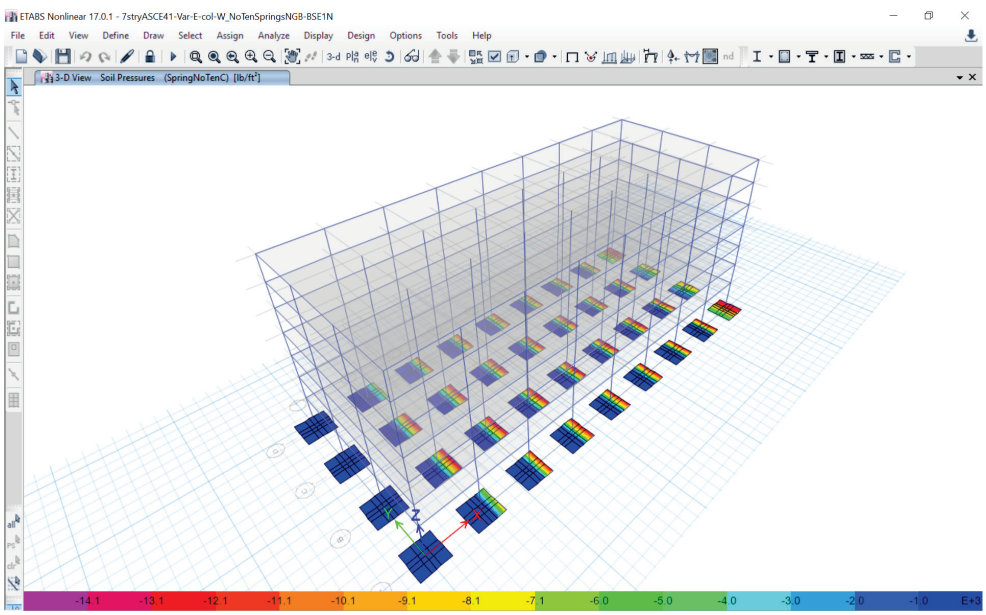


Figure C-33 Soil does not take Tension Eq Hazard Level BSE-1N Max pressure = 15.7 ksf

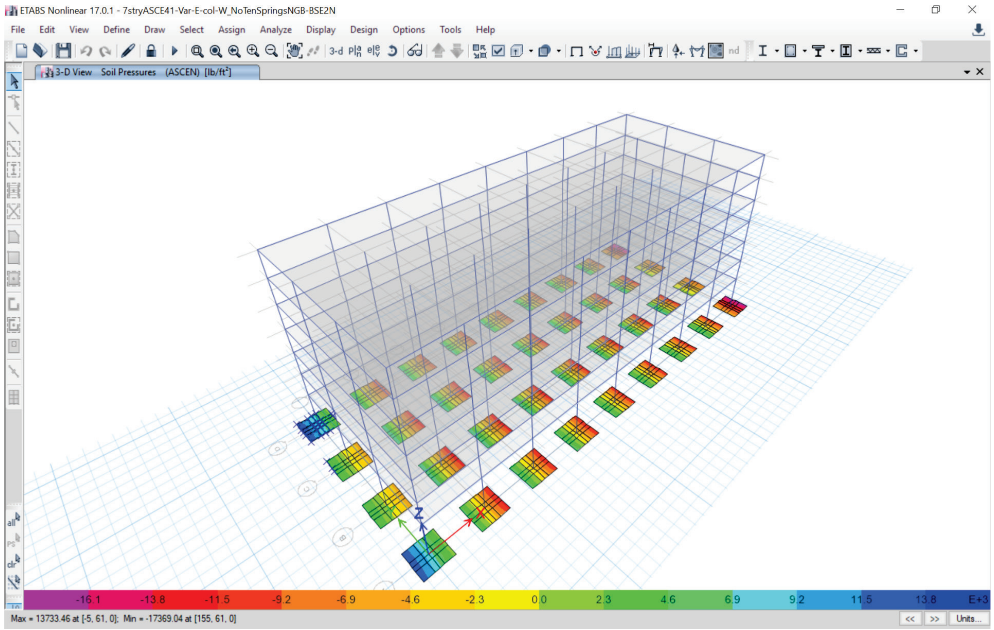


Figure C-34 Soil takes Tension Eq Hazard level BSE-2N Max pressure = 17.8 ksf.

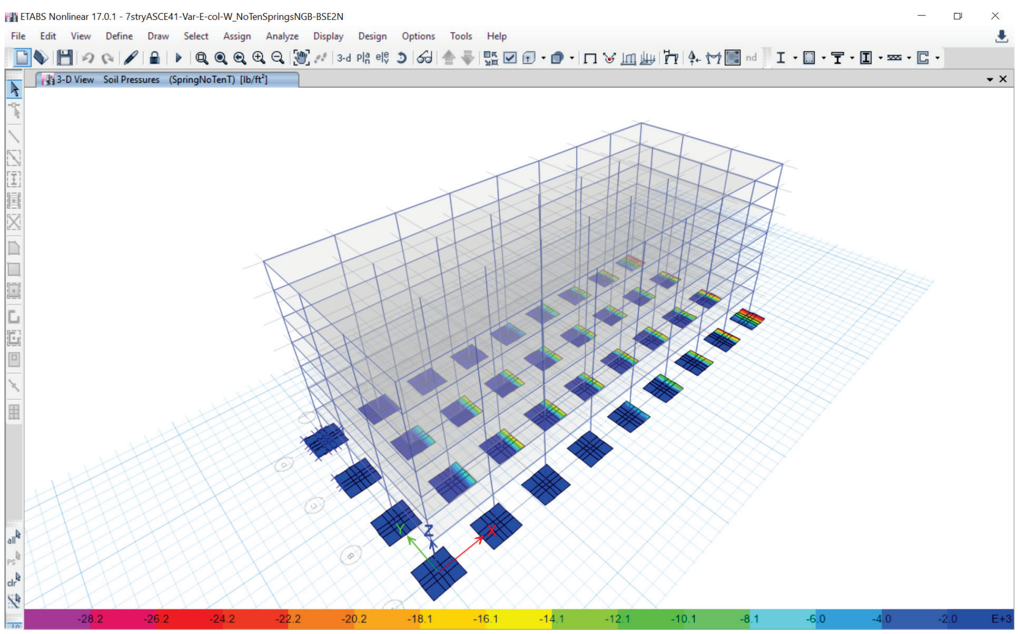


Figure C-35 Soil does not take Tension Eq Hazard Level BSE-2N Max pressure = 28.2 ksf

Observation of the soil pressures for the different analyses shows that when the superstructure is modeled as elastic, and the soil is modeled as nonlinear compression only springs, as the seismic overturning demand increases, there is a large uplift and shifting of the loads so that only few footings are in contact with the soil. Therefore, consistent with the observations from the superstructure demands, for linear analysis procedures it is not recommended to included foundation springs which act nonlinearly, where soils do not resist tension combined with an elastic analysis for the superstructure in the same computer model.

Soil Bearing Pressures – NSP

The soil bearing pressures when the superstructure is permitted to yield shows a very different soil bearing pressure profile but would be similar to the bearing pressure profile for the baseline demands using ASCE 7 assuming the model was also created using a flexible base judging from the axial load demands (Figure C-36). Results from the NSP show that the soil bearing Q_{max} = 9.24 < $3q_{allow}$ = 10.5 ksf, is satisfied at the expected strength level, and use of upper bound strengths are not required to satisfy the acceptance criteria for soil bearing.

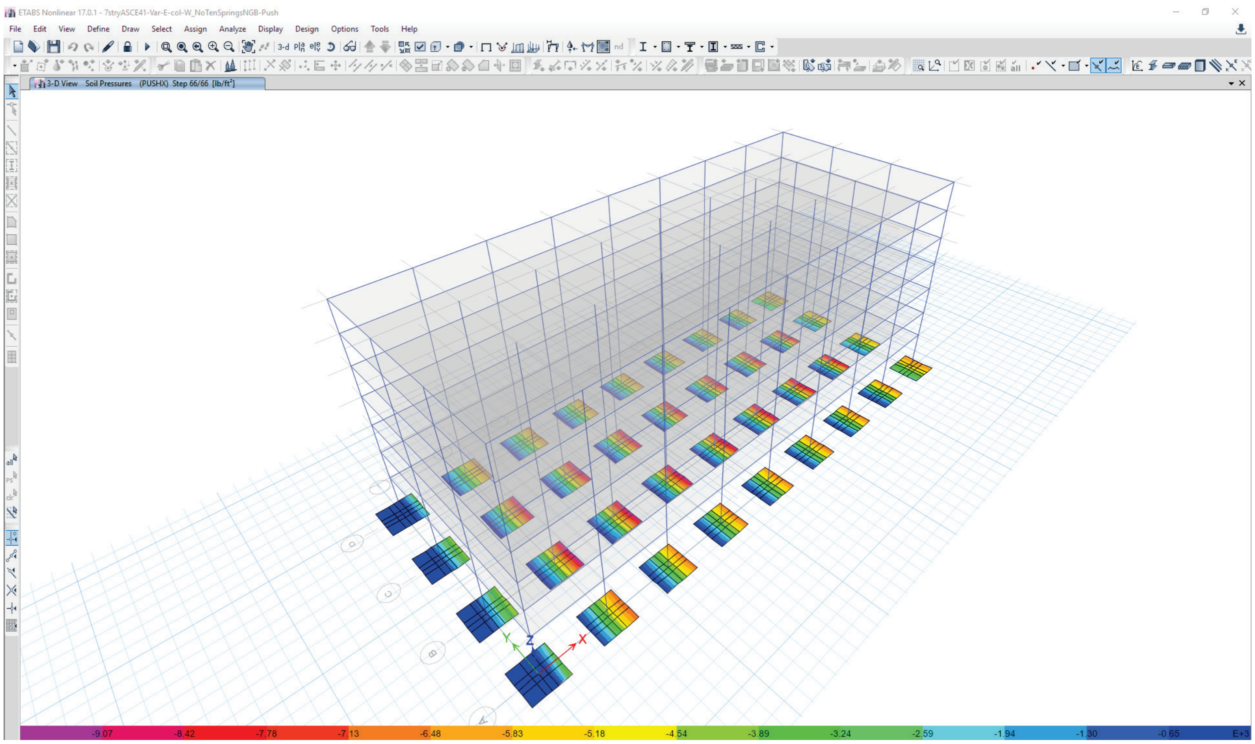


Figure C-36 Soil does not take Tension Eq Hazard Level BSE-2N NSP Max pressure = 9.24 ksf

Conclusion

When linear elastic procedures are used and the building is modeled as a flexible-base, where soils resist tension, the maximum soil bearing pressure may be compared with the use of upper bound soil strength as reasonable measure of acceptance of the footing for soil bearing. When nonlinear procedures are used, use of expected values of soil bearing for acceptance appear reasonable.

FOUNDATION ACCEPTANCE USING ASCE/SEI 7, AND ASCE/SEI 41 CRITERIA

Footing acceptance ratios from ASCE/SEI 7-10 (Table C-8) are contrasted with the acceptance ratios from ASCE/SEI 41 equation 8-10 using soil upper bound and lower bound capacities.

Table C-8 Acceptance Ratio, Bearing pressure – ASCE/SEI 7

Story	Column ID	P Comp (Kip)	M3 (kip ft)	B_f (ft)	L_f (ft)	e (ft)	q_{max} (ksf)	q allowable (ksf)	Acceptance Ratio
STORY1	C1	-112.4	86.0	10	10	0.77	1.6	4.65	0.35
STORY1	C5	-374.8	130.9	10	10	0.35	4.5	4.65	0.97
STORY1	C9	-349.5	129.4	10	10	0.37	4.3	4.65	0.92
STORY1	C13	-349.7	129.6	10	10	0.37	4.3	4.65	0.92
STORY1	C17	-349.8	129.6	10	10	0.37	4.3	4.65	0.92
STORY1	C21	-349.6	129.6	10	10	0.37	4.3	4.65	0.92
STORY1	C25	-351.4	129.4	10	10	0.37	4.3	4.65	0.92
STORY1	C29	-347.5	130.9	10	10	0.38	4.3	4.65	0.92
STORY1	C33	-270.8	86.0	10	10	0.32	3.2	4.65	0.69

LC = $(1 + .14S_{DS})D + 0.5L + 0.7*0.75Q_E$

q allowable = 4.65 ksf with 1/3 increase for seismic

Note: Overturning demands reduced by 25% per ASCE/SEI 7-10 section (12.13.4)

Requirements in ASCE/SEI 7-41 Section 8.4.2.3 state:

> For rectangular footings, the upper-bound moment capacity shall be determined using Eq. (8-10) with the expected values of P_{UD} and q using q_c multiplied by $(1 + C_v)$. The lower bound moment capacity shall be determined with the expected values of P_{UD} and q and using q_c divided by $(1 + C_v)$. The expected vertical load P_{UD} is taken as the maximum action that can be developed based on a limit-state analysis considering the expected strength of the components delivering force to the footing; alternatively, the expected vertical load is determined by dividing the seismic linear elastic load by the maximum demand-capacity ratio (DCR) of the components in the load path and summing with the gravity loads.

And are expressed mathematically as shown in Figure C-36 below.

$$M_{CE} = \frac{L_f P_{UD}}{2}\left(1 - \frac{q}{q_c(1 + C_v)}\right) \qquad (8-10)$$

$$M_{CE} = \frac{L_f P_{UD}}{2}\left(1 - \frac{q(1 + C_v)}{q_c}\right) \qquad (8-10)$$

$$P_{UD} = P_G \pm \frac{P_E}{DCR}$$

$$q = P_{UD}/(B_f L_f)$$

Figure C-36 Upper and Lower bound moment capacities using ASCE/SEI 41-17.

Footing acceptance ratios for the fixed base analysis (**Case 7**) and flexible-base analysis (**Case 8**) based on acceptance criteria in ASCE/SEI 41-17 are presented in Table C-9 through Table C-12.

Acceptance ratios for linear procedures using fixed base or flexible-base analysis are permitted to use upper bound values for soil strength. Acceptance ratios using lower bound strength are shown for comparison only. **Note**: Soil stiffness for the flexible base analysis should use lower bound stiffness properties. Expected stiffness values were used for this comparison, but it is expected that the overall results between the two will be similar, and the trend can be observed from differences from the fixed base and flexible base results.

If the lower bound soil strengths were required to be used, it would indicate the foundations would not meet the desired acceptance criteria, as the footing is unstable.

The acceptance ratio for column C1 is based on the seismic axial load being less than *m*-factor times the gravity load on the column since this column goes into net tension. Note the *m*-factors for uplift are twice the *m*-factors for overturning compression rocking action. It should also be noted that for the fixed-base analysis the first interior footing does not meet the acceptance criteria if the AR for axial load is taken as 1.0. This is because the seismic demands cause this column to go into uplift thus reducing the moment capacity even though there is still net compression on the footing, so the m-factors for uplift would not apply.

Sample Calculations:

1. Table C-9, Column C1, Fixed-Base, gravity and seismic loads are counteracting.

   $P_{UD} = P_G - P_E/DCR = 163 - 1161/2 = -418$ net tension, column is in uplift, therefore m-factors for uplift apply.

   M = 8.0 at CP.

   $AR = P_E/m(P_G) = 1161/(8)(163) = 0.89 < 1.0$ OK

2. Table C-1, Column C29, Fixed-Base, gravity and seismic loads are additive, upper bound.

   $P_{UD} = P_G - P_E/DCR = 278 - 202/1 = 76$ kips compression, column is in compression, therefore m-factors for compression apply.

   M = 4.0 at CP.

   $Q = P_{UD}/B_f L_f$ = (76 kips)/(10 ft)/(10 ft) = 0.76 ksf

   M_{CE} = (10 ft)(76 kips)/2(1- (0.76 ksf)/(10.5 ksf)/(1 + 1) = 367 kips (ASCE/SEI Eq. 8-10)

   $AR = M_{UD}/m\kappa M_{CE} = 1840/((4)(1)(376) = 1.25 > 1.0$ NG

Table C-9 Footing acceptance ratios (ASCE/SEI 41-17 Eq. 7-2) Fixed Base analysis at BSE-2N

Column ID	M_3	P_G = 0.9D	P_E	DCR	P_{UD}	B_f	L_f	q	q_c	C_v	M_{CE}UB	M_{CE}LB	m CP	Acceptance Ratio UB $Q_{UD}/(m\kappa Q_{CE})$	Acceptance Ratio LB $Q_{UD}/(m\kappa Q_{CE})$
C1	1208	-163	1161	2	-418	10	10	-4.18	10.5	1	-2507	-3755	8.0	0.89	0.89
C5	1840	-278	-202	1	480	10	10	4.80	10.5	1	1852	205	4.0	0.25	2.25
C9	1820	-271	14	1	256	10	10	2.56	10.5	1	1124	656	4.0	0.40	0.69
C13	1823	-270	-1	1	271	10	10	2.71	10.5	1	1179	656	4.0	0.39	0.70
C17	1822	-270	0	1	270	10	10	2.70	10.5	1	1177	656	4.0	0.39	0.69
C21	1823	-270	1	1	270	10	10	2.70	10.5	1	1175	656	4.0	0.39	0.69
C25	1820	-271	-14	1	285	10	10	2.85	10.5	1	1232	651	4.0	0.37	0.70
C29	1840	-278	202	1	76	10	10	0.76	10.5	1	367	326	4.0	1.25	1.41
C33	1208	-163	-1161	2	743	10	10	7.43	10.5	1	2401	-1545	4.0	0.13	Unstable

Note: DCR limited to 2C1C2 only for the end columns (ASCE/SEI 41-17, Section 8.4.2.3)

Table C-10 Footing acceptance ratios (ASCE/SEI 41-17 Eq. 7-1) Fixed Base analysis at BSE-2N

Column ID	M_3	P_G 1.1D+0.275L	P_E	DCR	P_{UD}	B_f	L_f	q	q_c	C_v	M_{CE}UB	M_{CE}LB	m CP	Acceptance Ratio UB $Q_{UD}/(m\kappa Q_{CE})$	Acceptance Ratio LB $Q_{UD}/(m\kappa Q_{CE})$
C1	1208	-205	1161	2	-376	10	10	-3.76	10.5	1	-2213	-3221	4.0	0.71	0.71
C5	1840	-355	-202	1	557	10	10	5.57	10.5	1	2047	-172	4.0	0.22	Unstable
C9	1820	-346	14	1	331	10	10	3.31	10.5	1	1395	611	4.0	0.33	0.74
C13	1823	-345	-1	1	346	10	10	3.46	10.5	1	1444	590	4.0	0.32	0.77
C17	1822	-345	0	1	345	10	10	3.45	10.5	1	1442	591	4.0	0.32	0.77
C21	1823	-345	1	1	344	10	10	3.44	10.5	1	1440	592	4.0	0.32	0.77
C25	1820	-346	-14	1	360	10	10	3.60	10.5	1	1492	565	4.0	0.30	0.80
C29	1840	-355	202	1	153	10	10	1.53	10.5	1	711	543	4.0	0.65	0.85
C33	1208	-205	-1161	2	786	10	10	7.86	10.5	1	2459	-1952	4.0	0.12	Unstable

Table C-11 Footing acceptance ratios (ASCE/SEI 41-17 Eq. 7-2) Flexible Base analysis at BSE-2N

Column ID	M_3	P_G 0.9D	P_E	DCR	P_{UD}	B_f	L_f	q	q_c	C_v	M_{CE}UB	M_{CE}LB	m $_{CP}$	Acceptance Ratio UB $Q_{UD}/(m\kappa Q_{CE})$	Acceptance Ratio LB $Q_{UD}/(m\kappa Q_{CE})$
C1	1198	-179	950	2	-296	10	10	-2.96	10.5	1	-1685	-2309	4.0	0.66	0.66
C5	1747	-267	-62	1	329	10	10	3.29	10.5	1	1388	614	4.0	0.31	0.71
C9	1757	-272	-20	1	292	10	10	2.92	10.5	1	1259	648	4.0	0.35	0.68
C13	1754	-270	0	1	270	10	10	2.70	10.5	1	1177	656	4.0	0.37	0.67
C17	1754	-270	0	1	270	10	10	2.70	10.5	1	1177	656	4.0	0.37	0.67
C21	1754	-270	0	1	271	10	10	2.71	10.5	1	1179	656	4.0	0.37	0.67
C25	1757	-272	20	1	252	10	10	2.52	10.5	1	1110	655	4.0	0.40	0.67
C29	1747	-267	62	1	205	10	10	2.05	10.5	1	925	625	4.0	0.47	0.70
C33	1198	-179	-951	2	654	10	10	6.54	10.5	1	2252	-807	4.0	0.13	Unstable

Table C-12 Footing acceptance ratios (ASCE/SEI 41-17 Eq. 7-1) Flexible Base analysis at BSE-2N

Column ID	M_3	P_G 1.1D+0.275L	P_E	DCR	P_{UD}	B_f	L_f	q	q_c	C_v	M_{CE}UB	M_{CE}LB	m $_{CP}$	Acceptance Ratio UB $Q_{UD}/(m\kappa Q_{CE})$	Acceptance Ratio LB $Q_{UD}/(m\kappa Q_{CE})$
C1	1198	-227	950	2	-248	10	10	-2.48	10.5	1	-1384	-1822	4.0	0.52	0.52
C5	1747	-341	-62	1	403	10	10	4.03	10.5	1	1629	468	4.0	0.27	0.93
C9	1757	-348	-20	1	368	10	10	3.68	10.5	1	1518	550	4.0	0.29	0.80
C13	1754	-345	0	1	345	10	10	3.45	10.5	1	1442	591	4.0	0.30	0.74
C17	1754	-345	0	1	345	10	10	3.45	10.5	1	1442	591	4.0	0.30	0.74
C21	1754	-345	0	1	346	10	10	3.46	10.5	1	1444	590	4.0	0.30	0.74
C25	1757	-348	20	1	328	10	10	3.28	10.5	1	1383	616	4.0	0.32	0.71
C29	1747	-341	62	1	279	10	10	2.79	10.5	1	1210	654	4.0	0.36	0.67
C33	1198	-227	-951	2	702	10	10	7.02	10.5	1	2337	-1186	4.0	0.13	Unstable

Conclusion/Recommendations

The acceptance ratios for soil bearing are very different when the results between ASCE/SEI 7 and ASCE/SEI 41 are compared. The footing acceptance ratio in ASCE/SEI 41-17 is governed by uplift at the end columns of the moment frame, while the highest acceptance ratios in ASCE/SEI 7 occur in the interior columns with high gravity and moment. The end column with the highest seismic axial compression load has the lowest AR when upper bound soil strengths are used, and the footing is unstable when lower bound soil bearing capacities values are used.

An alternate procedure is suggested in the commentary in ASCE/SEI 41-17 Section C8.4.2.3.2.1 for multiple isolated footings coupled by the superstructure above where the total area of the footing A_f is summed for all the footings, and the axial load P_{UD} is summed for all the axial loads. However, there is no further guidance on how this provision is to be applied when calculating the moment capacity or the acceptance criteria for the foundation.

Use of lower bound strength would result in too conservative results and is not recommended. Upper bound strength appears conservative but the nonlinear nature of the moment capacity equation, makes it difficult to predict the appropriate bearing values for consistent AR when compared with ASCE/SEI 7. Additional research justification is required as investigated later in this chapter.

ACCEPTANCE CRITERIA FOR THE STRUCTURAL FOOTING

Evaluate Footing for Building Modeled as a Fixed-Base – ASCE 41, BSE-2N hazard @ CP

The maximum axial load and moment, at the BSE-2N hazard level, for the building modeled as a fixed-base, with elastic soil springs, occurred at the footing supporting the corner column for the load combination 1.1(D + 0.25L) + Q_E. The DCR_{max} is capped per ASCE/SEI 41-17 Section 8.4.2.3.1 at $2C_1C_2$, or 2.0 since $C_1 = C_2 = 1.0$. The gravity moment M_G is ignored and assumed as zero, and moment due to seismic M_{OT} is divided by the m-factor of 4.0 for the Collapse Prevention performance level. The corresponding soil pressure distribution under the footing for the applied axial load and moments for the corner column from the fixed-base analysis is shown in Figure C-37.

Footing 10' x 10' x 2'

Column	b in	20
14x20	h in	14
Footing	B_f ft	10
10'x10'x2'	L_f ft	10
	H_{ftg} ft	2
Axial Load	P_G =	205
	P_E	1161
DCR_{Axial}	DCR_{max}	2
Moment	M_{UD} kip ft	1208
m-factor	mκ	4
	Q_{allowG} ksf	3.5
	f'$_c$ psi	4000
	f_y ksi	60
	#of bars	10
	Area per bar	1
	As	10
	As $_{min}$	8.0
	d	20.0

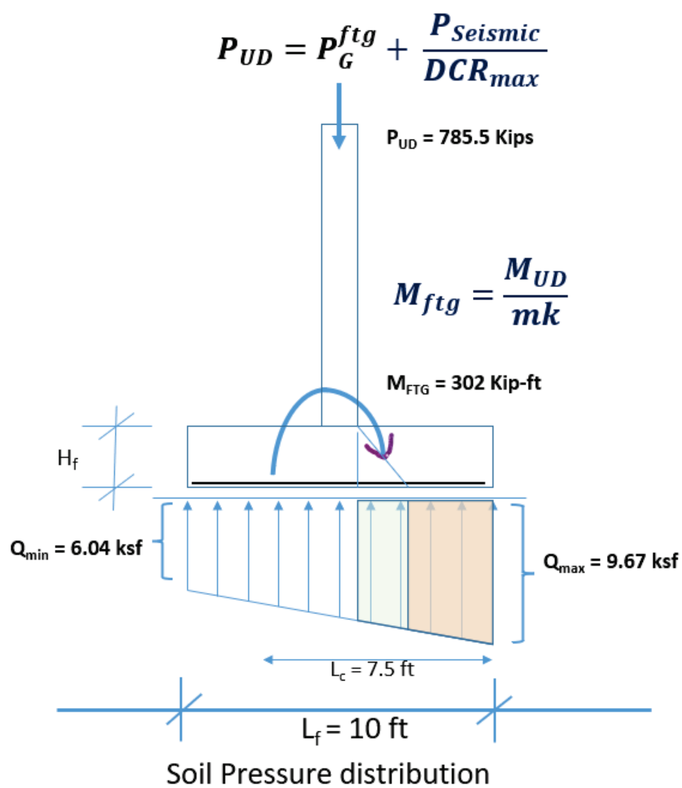


Figure C-37 Soil pressure distribution at the corner column for the fixed-base model.

Check Acceptance Ratio Moment at critical section

Moment demand is calculated at the face of the 14 × 20 column for the soil distribution shown in Figure C-37.

Q_{min} at face of the column = 8.07 ksf

Dividing the soil pressure profile into a rectangle and a triangle, the moment at the face is the sum of the moments from each soil pressure block is calculated as

Moment at column face

M_{UD} = (8.07 ksf)(10 ft){5 ft –(14 in)/(2)/(12)}^{2}/2 + (9.67 ksf – 8.07 ksf)(10 ft){5 ft –(14 in)/(2)/(12)}^{2}/3

= 978 kip-ft

AR = 978/963 = 1.02

Check Acceptance Ratio Shear at critical section

Shear demand is calculated at a distance "*d*" the face of the 14x20 column.

Q_{min} at distance *d* from face of the column = 8.67 ksf

Dividing the soil pressure profile into a rectangle and a triangle, the shear at the critical section is the sum of the moments from each soil pressure block is calculated as

Shear demand at critical section

V_{UD} = (8.67 ksf)(10 ft){5 ft –(14 in)/(2)/(12) – (20 in)/12} + (9.67 ksf – 8.67 ksf)(10 ft){5 ft –(14 in)/(2)/12 – (20 in)/12}/2

=279 kips

AR = 279/304 = 0.92

Evaluate Footing for Building Modeled as a Flexible-Base – ASCE 41, BSE-2N hazard @ CP

The maximum axial load and moment, at the BSE-2N hazard level, for the building modeled as a flexible-base, with elastic soil springs, occurred at the footing supporting the corner column for the load combination 1.1(D + 0.25L) + Q_E. The DCR_{max} is capped per ASCE/SEI 41-17 Section 8.4.2.3.1 at $2C_1C_2$, or 2.0 since $C_1 = C_2 = 1.0$. The gravity moment M_G is ignored and assumed as zero, and moment due to seismic M_{OT} is divided by the ductility *m*-factor of 4.0 for the Collapse Prevention performance level. The corresponding soil pressure distribution under the footing for the applied axial load and moments for the corner column from the flexible-base analysis is shown in Figure C-38.

Footing 10' x 10' x 2'

Column	b in	20
14x20	h in	14
Footing	B_f ft	10
10'x10'x2'	L_f ft	10
	H_{ftg} ft	2
Axial Load	P_G =	227
	P_E	951
DCR_{Axial}	DCR_{max}	2
Moment	M_{UD} kip ft	1198
m -factor	mκ	8.1
	Q_{allowG} ksf	3.5
	f'$_c$ psi	4000
	f_y ksi	60
	#of bars	10
	Area per bar	1
	As	10
	As $_{min}$	8.0
	d	20.0

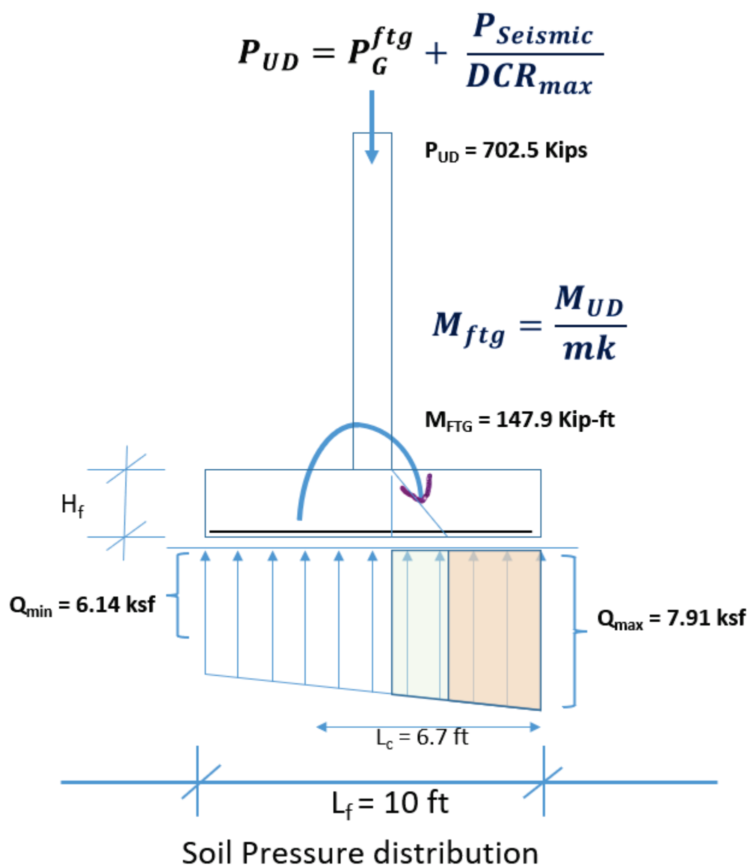


Figure C-38 Soil pressure distribution at the corner column for the flexible-base model.

Check Acceptance Ratio Moment at critical section

Moment demand is calculated at the face of the 14 × 20 column for the soil distribution shown in Figure C-38.

Q_{min} at face of the column = 7.13 ksf

Dividing the soil pressure profile into a rectangle and a triangle, the moment at the face is the sum of the moments from each soil pressure block is calculated as

Moment at column face

M_{UD} = (7.13 ksf)(10 ft){5 ft –(14 in)/(2)/(12)}^{2}/2 + (7.91 ksf – 7.13 ksf)(10 ft){5 ft –(14 in)/(2)/(12)}^{2}/3

= 746 kip-ft

AR = 746/963 = 0.77

Check Acceptance Ratio Shear at critical section

Shear demand is calculated at a distance "*d*" the face of the 14x20 column.

Q_{min} at distance *d* from face of the column = 7.42 ksf

Dividing the soil pressure profile into a rectangle and a triangle, the shear at the critical section is the sum of the moments from each soil pressure block is calculated as

Shear demand at critical section

V_{UD} = (7.42 ksf)(10 ft){5 ft –(14 in)/(2)/(12) – (20 in)/12} + (7.91 ksf – 7.42 ksf)(10 ft){5 ft –(14 in)/(2)/12 – (20 in)/12}/2

=211 kips

AR = 211/304 = 0.69

Summary

The maximum AR for the design of footing as a new building at the BSE-1N earthquake hazard using ASCE/SEI 7-10 was 0.60, and was 1.02 using ACSE/SEI 41-17, when the foundation is modeled as a fixed-base. This indicates that footings designed to the requirements of ASCE 7 for new buildings will not meet the performance objective of BPON at the collapse prevention level when the building is modeled as fixed-base. Modeling the building as a flexible-base results in lower acceptance ratios. The higher *m*-factors and the slightly reduced load demands because of the higher period of the building and load redistribution to other gravity and lateral force resisting elements with deformation of the footing due to settlement. But both methods using ASCE/SEI 41-17, modeling the building as fixed or flexible resulted in higher acceptance ratios when evaluating the strength of the structural footing.

Takeaways

- Actual demands (DCR not capped) should be used when computing moment capacity of the footing and acceptance criteria "m" adjusted accordingly
- A well-designed new concrete moment frame building in ASCE/SEI 7, could show noncompliance when evaluated using ASCE/SEI 41-17 when evaluated for a performance objective of BPON.
- Use of lower bound soil capacity for C_v = 1 (half of expected strength) may be too conservative, propose to change to 0.5 or eliminated.
- Axial tension in column may be limited to maximum weight of the footing including the adjacent floor slab.

- Where seismic demands subtract from column axial load, the acceptance criteria for tension should be applied regardless is the column goes into tension.

Conclusion/Recommendations

Footings evaluated with demands and *m*-factors from a fixed-base analysis using ASCE/SEI 41-17 are more conservative than similar designs using ASCE/SEI 7-10. Evaluating the building as a flexible base with associated *m*-factors results in a more favorable outcome but the results are still conservative when compared with ASCE/SEI 7-10. Evaluating the footings as force-controlled is also likely to produce conservative results if conservative estimates are made for the maximum force delivered to the foundation as this does not account for redistribution of forces with foundation displacement. Use of *m*-factors from the material chapters based on the action on the footing is a preferred alternative,

C.3.3 Overall Summary/Conclusion

Comparing the results from the various analysis using the linear static procedure (LSP) with the results from the nonlinear static procedure (NSP) shows that when the elements of the superstructure are ductile relative to the foundation system, combining results from a linear superstructure with a nonlinear foundation leads to results inconsistent with what engineering judgement would predict. For this reason, for LSP, modeling only the foundations as nonlinear is not recommended. Results from the LSP with all elements modeled as elastic and the NSP gave reasonable correlation with baseline ASCE/SEI 7 analysis model.

The evaluation of the footing structural component using linear analysis procedures (LSP) fixed-base of flexible-base, shows that modeling new buildings using the requirements in ASCE/SEI 41-17 result in more conservative designs than using the prescriptive methods of ASCE 7-10 when fixed base *m*-factors are used.

C.4 Investigation of Alternate Foundation Acceptance Criteria

C.4.1 Summary of foundation acceptance for each archetype building modeled as a fixed-base

Since the lateral resisting systems selected for the two case study buildings were different, it gave a good opportunity for a comparative foundation evaluation check between the outcomes using ASCE/SEI 7-10 and ASCE/SEI 41-17. Summary comparisons of the acceptance ratios for the two archetype buildings. Archetype Building 1 given in appendix B and this building, Archetype Building 2, for foundations modeled as a fixed-base.

C.4.1.1 ARCHETYPE BUILDING 1, FOUNDATION ACCEPTANCE COMPARISON BETWEEN ASCE/SEI 7-10 AND ASCE/SEI 41-17 FOR SOIL BEARING

For the case study building, Archetype Building 1, with details and calculations shown in Appendix B, the existing foundation was not adequate to support overturning forces due to lateral loading on the new concrete shear walls. New concrete foundations were added between gridlines 2 and 5 as shown in Figure C-39.

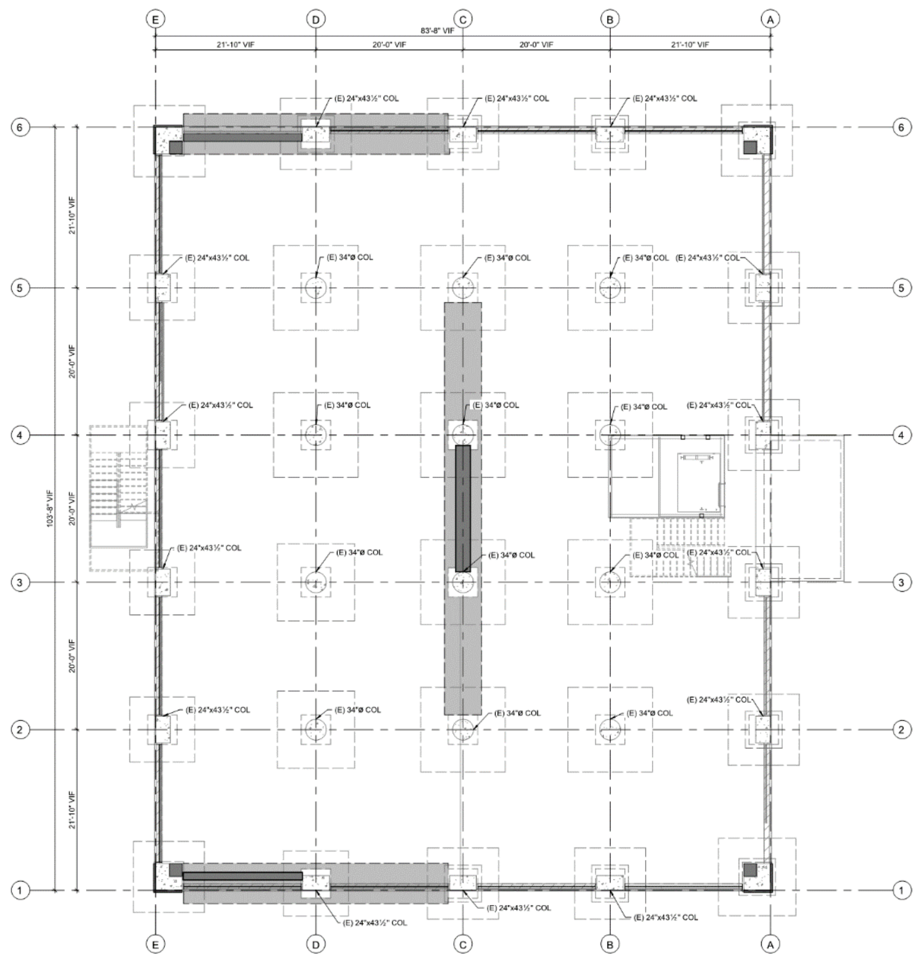


Figure C-39 Foundation Plan: With Proposed Foundation Retrofit

The retrofit foundation was designed based on ASCE/SEI 7-10 provisions and the same loading conditions for the new superstructure. This foundation was evaluated for comparison using ASCE/SEI 41-17 for the same hazard level loading conditions associated acceptance criteria for the building modeled as a fixed-base. Therefore, no change to the analysis was required because within the analysis model the structure from the original as the foundation was assumed as fixed.

Footing retrofit geometry

The retrofit footing connected the existing pad footing between gridlines 2 and 5 together to create one continuous footing with the new footing retrofit plan layout is shown in Figure C-40, with geometric properties in Table C-13. To simplify the analysis, the retrofit footing is approximated as a rectangular footing with an average footing width to account for the variations in footing width along its length.

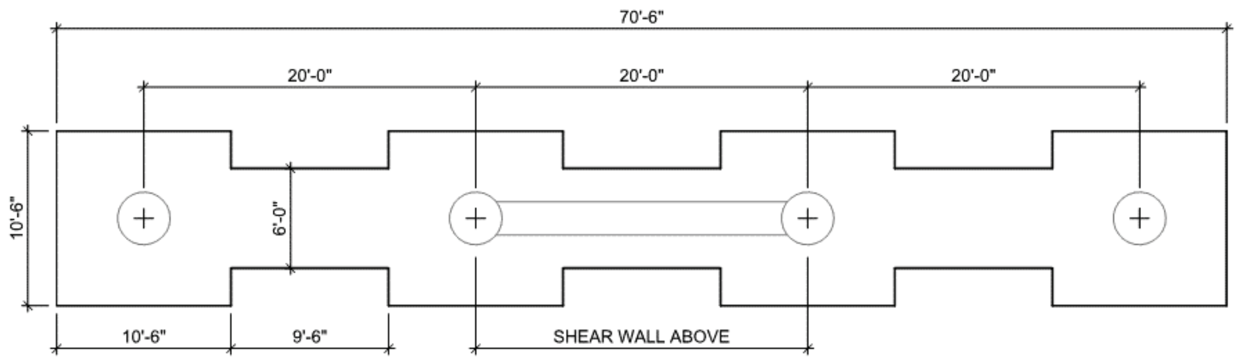


Figure C-40 Retrofit Footing Plan Layout with Dimensions

Table C-13 Retrofit Footing Geometric Properties

Footing Area (Af)	612 ft^2
Average Footing Width (B)	8.7 ft

Retrofit footing acceptance using ASCE/SEI 7-10

For the moment demand on the new retrofit footing, the acceptance ratio was 0.85 for loading details shown in Figure C-41.

ASCE 7-10 Footing design, bearing pressure			
M =	13177.5	k-ft, ASD (ETABS) with 25% reduction	
P, total =	1304	kips (sum of dead load at all 4 cols)	
footing width, B =	8.7	ft	
footing length, L =	70.5	ft	
L/ 6 =	11.8	ft	
M/P = e =	10.1	ft	e> L/6
e' =	25.1	ft	
q max =	3.98	ksf	<4.66 ksf okay
DCR =	0.85		

Figure C-41 Footing acceptance for soil bearing using ASCE/SEI 7-10

Retrofit footing acceptance using ASCE/SEI 41-17

For the same axial load on the footing, but for moment demands from the ASCE/SEI 41-17 evaluation at the CP level, the acceptance ratio for soil bearing was 1.2 as shown in Figure C-42.

ASCE 41-17 with footing, check overturning assuming foundation is rigid compared to soil		
PUD =	1304	kips
Af =	612	ft2
q =	2.13	ksf
qc =	21.0	ksf, upper-bound in accordance with 8.4.2.3.2
Lf =	70.5	ft
MCE =	41302	k-ft
Mbase =	198580	k-ft (ETABS)
required m =	**4.8**	
allowable m =	4	Section 8.4.2.3.2.1
Acceptance ratio =	1.20	

ASCE 41-17 with footing, check overturning stability (uplift) assuming foundation is rigid compared to soil		
Q_E = ΣMu EQ =	198580	k-ft
ΣM DL =	39120	k-ft
Q_G = 0.9DL =	35208	k-ft
required m =	**5.6**	(Q_E/Q_G)
allowable m =	8	Section 8.4.2.3.2.1
Acceptance ratio =	0.71	

ASCE 41-17 with footing, Overturning Effects for Linear Procedures		
Restoring Moment, M_{ST} =	39120	k-ft
M_{OT} = ΣMu EQ =	198580	k-ft
C_1C_2 =	1.1	
$0.9M_{ST}$ =	35208	k-ft
M_{OT}/C_1C_2=	180527	k-ft
Required μ_{OT}=	**5.1**	
μ_{OT}=	10	
Acceptance ratio =	0.51	

Figure C-42 Footing acceptance for soil bearing using ASCE/SEI 7-10

Summary of retrofit footing acceptance between ASCE/SEI 7-10 and ASCE/SEI 41-17

A comparison of the footing acceptance for soil bearing overturning action presented in Figure C-43 shows the ASCE/SEI results are more conversative than if the footing were designed as a new building using ASCE/SEI 7-10.

	ASCE 41-17			ASCE 7-10	
	Section	CP m-factor	Acceptance Ratio	Section	DCR
LSP, Bearing Pressure =	8.4.2.3.2.1	4	1.20	12.13.4	0.85
LSP, Uplift =	8.4.2.3.2.1	8	0.71	12.13.4	0.56
LSP, Overall Overturning =	7.2.8.1	10	0.51	12.8.5	0.75
Outcome =			NG		OK

Figure C-43 Comparison of Acceptance Ratios between ASCE/SEI 7-10 and ASCE/SEI 41-17

C.4.1.2 COMPARISON OF FOOTING SOIL BEARING ACCEPTANCE BETWEEN ASCE/SEI 7-10 AND ASCE/SEI 41-17 FOR ARCHETYPE BUILDING 2

As shown and described earlier, the footing acceptance for soil bearing between ASCE/SEI 7-10 and ASCE/SEI 41-17 are very different for the building modeled as a fixed-base as shown by looking at the last column in Table C-14 and Table C-15. The acceptance for foundation compression and uplift are switched between the two methods. Note: only one direction of loading was considered, so the AR would be maximum from both directions on each footing. However, the results clearly show a disconnect between the two methods.

Table C-14 Acceptance Ratio, Bearing Pressure – ASCE/SEI 7-10

Story	Column ID	P Comp (Kip)	M3 (kip ft)	B_f (ft)	L_f (ft)	e (ft)	q_{max} (ksf)	q allowable (ksf)	Acceptance Ratio
STORY1	C1	-112.4	86.0	10	10	0.77	1.6	4.65	0.35
STORY1	C5	-374.8	130.9	10	10	0.35	4.5	4.65	0.97
STORY1	C9	-349.5	129.4	10	10	0.37	4.3	4.65	0.92
STORY1	C13	-349.7	129.6	10	10	0.37	4.3	4.65	0.92
STORY1	C17	-349.8	129.6	10	10	0.37	4.3	4.65	0.92
STORY1	C21	-349.6	129.6	10	10	0.37	4.3	4.65	0.92
STORY1	C25	-351.4	129.4	10	10	0.37	4.3	4.65	0.92
STORY1	C29	-347.5	130.9	10	10	0.38	4.3	4.65	0.92
STORY1	C33	-270.8	86.0	10	10	0.32	3.2	4.65	0.69

LC = $(1 + .14S_{DS})D + 0.5L + 0.7*0.75Q_E$

q allowable = 4.65 ksf with 1/3 increase for seismic

Table C-15 Acceptance Ratio, Bearing Pressure – ASCE/SEI 41-17

Column ID	M_{UD}	DCR	P_{UD}	B_f	L_f	q	q_c	C_v	$M_{CE}UB$	m_{CP}	Acceptance Ratio UB $Q_{UD}/(m\kappa Q_{CE})$
C1	1208	2	-418	10	10	-4.18	10.5	1	-2507	8.0	0.89
C5	1840	1	480	10	10	4.80	10.5	1	1852	4.0	0.25
C9	1820	1	256	10	10	2.56	10.5	1	1124	4.0	0.40
C13	1823	1	271	10	10	2.71	10.5	1	1179	4.0	0.39
C17	1822	1	270	10	10	2.70	10.5	1	1177	4.0	0.39
C21	1823	1	270	10	10	2.70	10.5	1	1175	4.0	0.39
C25	1820	1	285	10	10	2.85	10.5	1	1232	4.0	0.37
C29	1840	1	76	10	10	0.76	10.5	1	367	4.0	1.25
C33	1208	2	743	10	10	7.43	10.5	1	2401	4.0	0.13

C.4.1.3 COMPARISON SUMMARY BOTH ARCHETYPE BUILDINGS BETWEEN ASCE/SEI 7-10 AND ASCE/SEI 41-17

For Archetype Building 1, the maximum acceptance ratio for ASCE/SEI 7-10 was 0.85 compared to 1.20 using ASCE/SEI 41-17. While for Archetype Building 2 the majority of interior columns give an acceptance ratio of 0.40 using ASCE/SEI 41 compared to 0.92 using ASCE/SEI 7-10. In addition, the acceptance ratios for end bay columns using ASCE/SEI 41-17 are very different from ASCE/SEI 7-10. It can be argued that the fixed based results are in reasonable agreement between the two standards for the cantilevered shear wall example, it is difficult to make the same case for the moment frame building. Therefore, a search for alternate methods to establish and clarify the acceptance criteria for foundations using ASCE/SEI 41 was explored.

C.4.2 Issues Considered

- Soil bearing capacity and stiffness is different for gravity and dynamic loads.
- Gravity demands/acceptance criteria should not be reduced by ductility or m-factor.
- Cannot combine elastic pseudo seismic force and compare with nonlinear capacity equations based on real loads and then amplify by *m*-factor.

C.4.3 Essence of the Proposal

A new proposal was postulated based on the assumptions below:

- Apply pseudo force reduction by DCR on moment demand similar to axial load demand reduction in equation 8-10 of ASCE/SEI 41-17 also referred to as equation 8-10.
- Apply *m*-factor reduction only to seismic actions.
- Reformulate the acceptance criteria based on first principles.

Considering the issues above, a new acceptance ratio expressed in terms of maximum bearing pressure was proposed as given below:

$$Acceptance\ Ratio = \frac{\boldsymbol{P_{D+L}}}{\boldsymbol{q_{allowG}A_G}} + \frac{\boldsymbol{P_{Seis}}}{\boldsymbol{mq_{ultimate}A_g DCR_{supA}}} + \frac{\frac{\boldsymbol{M_{Seis}L}}{\boldsymbol{2}}}{\boldsymbol{mq_{ultimate}I_g DCR_{supM}}} \qquad Eq. C-1$$

Where:

DCR_{supA} = Reduction factor for pseudo force axial load action on the footing

DCR_{supM} = Reduction factor for pseudo force moment action on the footing

q_{allow} = allowable soil bearing capacity including the 1/3 increase for seismic

$q_{ultimate}$ = allowable soil bearing capacity including the 1/3 increase for seismic

P_{D+L} = Axial load on the footing from the superstructure and need not include the weight of the footing

Using the new formula, a comparison of the ARs between ASCE/SEI 41-17 and ASCE/SEI 7-10 for different performance objectives and for two different footing sizes is shown in the Table C-16 and Table C-17 below.

Table C-16 Comparison of ARs using the proposed formulation and ASCE/SEI 7 for a 10 × 10 footing

Footing size 10' x 10'

	Performance Level	Eq. Hazard	Max DCR Axial	Max DCR Moment	Interior Footing	Max DCR Axial	Max DCR Moment	End Bay Footing
ASCE 7-10	Risk Cat II, I = 1.0	BSE-1N	R= 8		0.93			0.69
	Risk Cat III, I = 1.25	BSE-1N			0.97			0.76
	Risk Cat IV, I = 1.5	BSE-1N			1.01			0.83
ASCE 41			Acceptance Criteria using proposed formulation					
	IO; m = 2	BSE-1N	1	1	1.03	1	1	0.83
	LS; m = 3	BSE-2N	1	1	1.03	2	1	0.69
	LS; m = 3	BSE-1N	1	1	0.93	1	1	0.67
	CP; m = 4	BSE-2N	1	1	0.95	2	1	0.61
BPON	Risk Category II				0.95			0.67
	Risk Category IV				1.03			0.83

Table C-17 Comparison of ARs using the proposed formulation and ASCE/SEI 7 for a 8 × 8 footing

Footing size 8' x 8'

	Performance Level	Eq. Hazard	Max DCR Axial	Max DCR Moment	Interior Footing	Max DCR Axial	Max DCR Moment	End Bay Footing
ASCE 7-10	Risk Cat II, I = 1.0	BSE-1N	R= 8		1.51			1.13
	Risk Cat III, I = 1.25	BSE-1N			1.60			1.25
	Risk Cat IV, I = 1.5	BSE-1N			1.68			1.37
ASCE 41			Acceptance Criteria using proposed formulation					
	IO; m = 2	BSE-1N	1	1	1.72	1	1	1.36
	LS; m = 3	BSE-2N	1	1	1.72	2	1	1.14
	LS; m = 3	BSE-1N	1	1	1.52	1	1	1.10
	CP; m = 4	BSE-2N	1	1	1.57	2	1	1.00
BPON	Risk Category II				1.57			1.10
	Risk Category IV				1.72			1.36

From the results it is clear that the proposed formulation for acceptance criteria aligns well for the moment frame example, or Archetype Building 2 for a range of footing sizes. However, there were questions as to the applicability for other types of lateral force resisting systems like:

- How do the results compare with existing equation 8-10?

- New formulation does not consider stability.
- New formulation is not consistent with the philosophy of ASCE/SEI 41.

To address these concerns, at the same time accounting for foundation uplift stability, alternate methods were researched, and new proposals brought forward.

Starting with the general acceptance criteria based on maximum soil pressure rather than a foundation overturning capacity, and where the demands to the foundations were reduced by "m" or "DCR" prior to the check, the following cases were considered.

CASE 1: FOOTING WITH SEISMIC DEMANDS REDUCED BY *m*-FACTOR WHERE FOOTING IS IN COMPLETE CONTACT WITH THE SOIL.

This condition is similar to the procedure used when designing footings with expected force demands including axial load and moment where the footing remains completely in contact with the soil (Figure C-44).

From elastic theory, the maximum soil pressure is determined as a superposition of the normal load on the soil from axial load and moment as P/A + My/I. If the pseudo force demands are converted to expected forces on the soil, by dividing only the seismic demands by m and DCR of the superstructure, the soil pressure can be written as:

$$Q_{max} = \frac{1}{A_g}\left(P_{D+L} + \frac{P_{Seis}}{mDCR_{supA}} + \frac{6M_{Seis}}{mDCR_{supM}L_f}\right) \qquad Eq. C-2$$

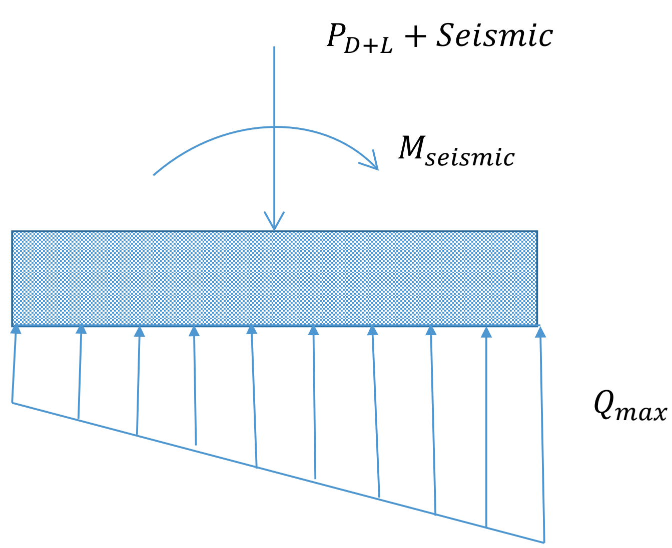


Figure C-44 Case 1, entire footing remains in contact with the soil

Making the following substitutions,

$$P_{equivalent} = P_{D+L} + \frac{P_{Seis}}{mDCR_{supA}} \qquad Eq. C - 3$$

$$M_{equivalent} = \frac{M_{Seis}}{mDCR_{supM}} \qquad Eq. C - 4$$

$$e_{equivalent} = \frac{M_{equvialnt}}{P_{equvialnt}} \qquad Eq. C - 5$$

And where:

$\boldsymbol{e_{equivalent}} \leq \frac{\boldsymbol{L_f}}{\boldsymbol{6}}$; where L_f is the length of the footing in the direction of rocking.

Q_{max} can be written in general form as

$$Q_{max} = \frac{P_{equivalent}}{A_g}\left(1 + \frac{6e_{equivalent}}{L_f}\right) \qquad Eq. C - 6$$

Starting with the new expression for soil pressure, the acceptance ratio can be written as:

$$Acceptance\ Ratio = \frac{P_{D+L}}{q_{allowG}A_G} + \frac{P_{Seis}}{mq_{ultimate}A_g DCR_{supA}} + \frac{M_{Seis}L/2}{mq_{ultimate}I_g DCR_{supM}}$$

Since $q_{ultimate} = 3q_{allowG}$, and making the substitution

$$P_{equivalent\_ac} = P_{D+L} + \frac{P_{Seis}}{3mDCR_{supA}} \qquad Eq. C - 7$$

$$M_{equivalent\_ac} = \frac{\boldsymbol{M_{Seis}}}{\boldsymbol{3mDCR_{supM}}} \qquad Eq. C - 8$$

and

$$\boldsymbol{e_{equivalent\_ac}} = \frac{\boldsymbol{M_{equivalent\_ac}}}{\boldsymbol{P_{equivalent\_ac}}} \qquad Eq. C - 9$$

Therefore when $\boldsymbol{e_{equivalent}} < \frac{\boldsymbol{L_f}}{\boldsymbol{6}}$, the acceptance ratio can be written as:

$$\text{Acceptance Ratio} = \frac{\boldsymbol{P_{equivalent\_ac}}}{\boldsymbol{A_g q_{allowG}}}\left(\boldsymbol{1} + \frac{\boldsymbol{6e_{equivalent\_ac}}}{\boldsymbol{L_f}}\right) \qquad Eq. C - 10$$

CASE 2: FOOTING WITH SEISMIC DEMANDS REDUCED BY *m*-FACTOR WITH FOOTING IN PARTIAL UPLIFT

This condition occurs when the maximum pressure from the pseudo force moment divided by *m*-factor and DCR puts the footing in partial uplift. The maximum soil pressure for this case is not a simple superposition of forces based on elastic theory and has to be established from statics.

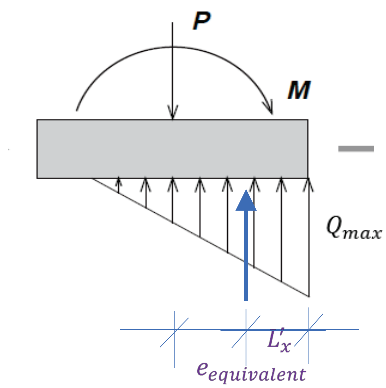


Figure C-45 Case 2, partial uplift of the footing and Q_{max} < $q_{ultimate}$.

Therefore when,

$$e_{equivalent} > \frac{L_f}{6};$$

Maximum soil pressure is given as:

$$Q_{max} = \frac{2P_{equivalent}}{3B_f L'_f} \qquad Eq. C-11$$

Where

$$L'_f = \frac{L_f}{2} - e_{equivalent}$$

Or,

$$Q_{max} = \frac{2P_{equivalent}}{3B_f\left(\frac{L_f}{2} - e_{equivalent}\right)} \qquad Eq. C-12$$

Making the substitution similar to Case 1, for $P_{equivalent\_ac}$ and $e_{equivalent\_ac}$ the AR can be written as:

$$\text{Acceptance Ratio} = \frac{2P_{equivalent\_ac}}{3B_f q_{allowG}\left(\frac{L_f}{2} - e_{equivalent\_ac}\right)} \qquad \text{Eq. C-13}$$

CASE 3 – SOIL PRESSURE BASED ON A RECTANGULAR DISTRIBUTION

This case is the same as the method used in the determination of the ultimate moment capacity of the foundation M_{CE}. Where the soil force deformation behavior is represented by an elastic perfectly plastic backbone curve, when the bearing capacity q_c is reached:

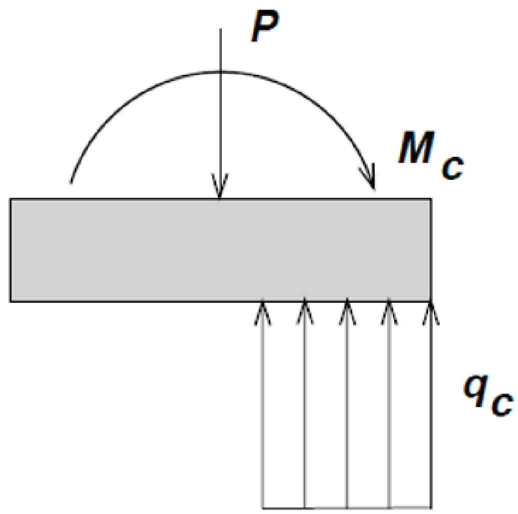


Figure C-46 Case 3, soil pressure calculated using a rectangular distribution.

The foundation ultimate moment capacity M_{CE} given in equation 8-10 of ASCE/SEI 41-17, can be rewritten in the terms of soil bearing capacity q_c as follows:

$$M_{CE} = \frac{L_f P_{UD}}{2}\left(1 - \frac{q}{q_c}\right)$$

$$\frac{M_{CE}}{P_{UD}} = \frac{L_f}{2}\left(\frac{q_c - q}{q_c}\right)$$

$$q_c e_{CE} = \frac{L_f}{2}(q_c - q)$$

$$q_c e_{CE} - \frac{L_f}{2} q_c = -\frac{L_f}{2} q$$

$$q_c\left(\frac{L_f}{2} - e_{CE}\right) = \frac{L_f}{2}\frac{P_{UD}}{B_f L_f}$$

$$q_c = \frac{P_{UD}}{2B_f\left(\frac{L_f}{2} - e_{CE}\right)} \qquad Eq. C-14$$

Substituting for P_{UD} and e_{CE}, this equation can be in terms of Q_{max} as:

$$Q_{max} = \frac{P_{equivalent}}{2B_f\left(\frac{L_f}{2} - e_{equivalent}\right)} \qquad Eq. C-15$$

Where:

$$\boldsymbol{P_{equivalent}} = \boldsymbol{P_{D+L}} + \frac{\boldsymbol{P_{Seis}}}{\boldsymbol{mDCR_{supA}}};$$

$$\boldsymbol{M_{equivalent}} = \frac{\boldsymbol{M_{Seis}}}{\boldsymbol{mDCR_{supM}}};$$

and

$$\boldsymbol{e_{equivalent}} = \frac{\boldsymbol{M_{equvialnt}}}{\boldsymbol{P_{equvialnt}}};$$

Therefore, in the limit when $Q_{max} = q_c$, the foundation overturning acceptance criteria is reached, and no additional check is required.

A summary of the maximum soil pressure for the three cases are given below.

$$\text{Case 1: } \boldsymbol{Q_{max}} = \frac{\boldsymbol{P_{equivalent}}}{\boldsymbol{A_g}}\left(\mathbf{1} + \frac{\boldsymbol{6e_{equivalent}}}{\boldsymbol{L_f}}\right); \ \mathbf{e} \leq \boldsymbol{L_f}/\mathbf{6}$$

$$\text{Case 2: } \boldsymbol{Q_{max}} = \frac{\boldsymbol{2P_{equivalent}}}{\boldsymbol{3L_y}\left(\frac{\boldsymbol{L_f}}{\mathbf{2}} - \boldsymbol{e_{equivalent}}\right)} \ \text{ - Triangular, e > } L_f/6$$

$$\text{Case 3: } \boldsymbol{Q_{max}} = \frac{\boldsymbol{P_{equivalent}}}{\boldsymbol{2L_y}\left(\frac{\boldsymbol{L_f}}{\mathbf{2}} - \boldsymbol{e_{equivalent}}\right)} \ \text{ - Rectangular}$$

$$\text{Moment Capacity: } \boldsymbol{M_{CE}} = \frac{\boldsymbol{L_f P_{UD}}}{\mathbf{2}}\left(\mathbf{1} - \frac{\boldsymbol{q}}{\boldsymbol{q_c}}\right) \ \text{ - Eq 8-10}$$

A summary of the equations used for the proposed acceptance ratio is given below.

$$P_{equivalent} = P_{D+L} + \frac{P_{Seis}}{3mDCR_{supA}}$$

$$M_{equivalent} = \frac{M_{Seis}}{3mDCR_{supM}}$$

$$e_{equivalent\_AC} \leq \frac{L_f}{6}$$

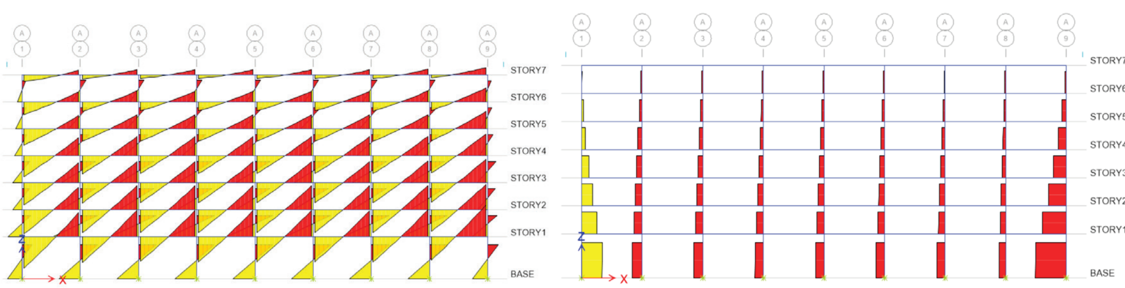
$$\text{Acceptance Ratio} = \frac{P_{equivalent\_AC}}{A_g Q_{allowG}}\left(1 + \frac{6e_{equivalent\_AC}}{L_f}\right)$$

$$e_{equivalent\_AC} > \frac{L_x}{6}$$

$$\text{Acceptance Ratio} = \frac{2P_{equivalent\_AC}}{3B_f Q_{allowG}\left(\frac{L_x}{2} - e_{equivalent\_AC}\right)}$$

C.4.3 Acceptance Criteria Check – Archetype Building 2

Taking the demands from the moment frame example for the interior and the end bay columns, with moment and axial load patterns shown in Figure C-47, the results are plotted (Figure C-48 and Figure C-49) in terms of soil pressure for the three cases and compared with that obtained from equation 8-10, when expected strengths for the soil bearing capacity are used, i.e. q_c = 3 x q_{allow}.



Moment Diagram

Axial Force Distribution

Figure C-47Moment and axial force distribution in elements of the LFRS

Soil Pressure with increasing Seismic Moment, Interior Bay, ($P_{seismic}$ = 0)

B =	10	**ft**
D =	10	**ft**
P_G =	350	**kips**
P_{seism}	0	**kips**
P_{seism}_inc	0	**kips**
DCR_{Axial}	1	
m_{Axail} - Factor	4	**CP**
M_{seis}	0	**kip-ft**
M_{seism}_inc	400	**kip-ft**
DCR_{mom}	1	
m_{mom} - Factor	4	**CP**
Q_{allowG}	4.5	**ksf**

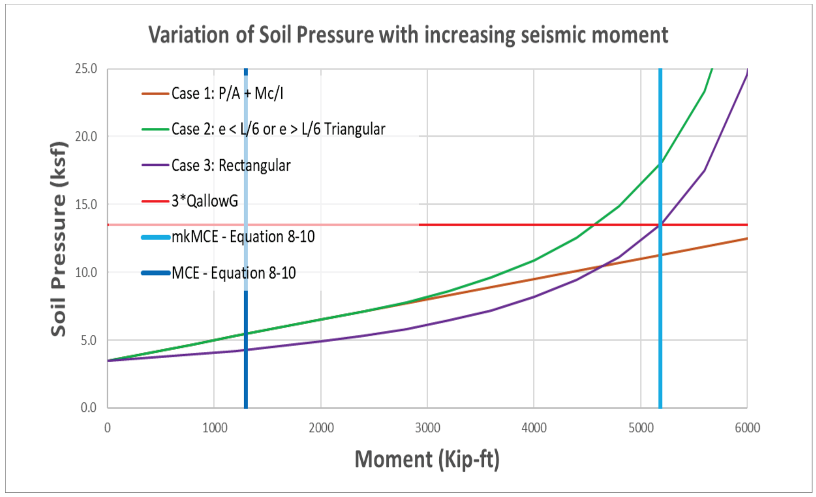


Figure C-48 Soil pressure variation with seismic overturning moment.

Proposed Acceptance Ratio with increasing Seismic Moment, Interior Bay, ($P_{seismic}$ = 0)

B =	10	**ft**
D =	10	**ft**
P_G =	350	**kips**
P_{seism}	0	**kips**
P_{seism}_inc	0	**kips**
DCR_{Axial}	1	
m_{Axail} - Factor	4	**CP**
M_{seis}	0	**kip-ft**
M_{seism}_inc	400	**kip-ft**
DCR_{mom}	1	
m_{mom} - Factor	4	**CP**
Q_{allowG}	4.5	**ksf**

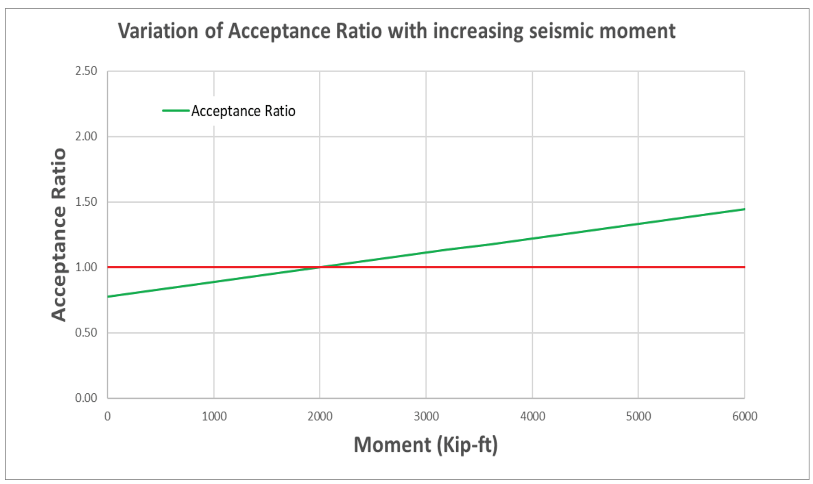


Figure C-49 Variation in acceptance ratio with increasing seismic overturning moment.

Observation of the results from the two scenarios shows that acceptance ratio for an interior bay footing based on soil pressure is approximately 5200 kip-ft and that using the new formulation is approximately 2000 kip-ft. These results indicate that there is a lot more reserve capacity in the foundation with respect to overturning resistance when soil bearing is used for the acceptance criteria.

A similar comparison is made for footings under the end bay columns where seismic demand adds to the gravity load as shown in Figure C-50 and Figure C-51.

Soil Pressure with increasing Seismic Moment, End Bay, ($P_{seismic} \neq 0$)

B =	10	ft
D =	10	ft
P_G =	250	kips
P_{seism}	0	kips
P_{seism_inc}	75	kips
DCR_{Axial}	2	
m_{Axail} - Factor	4	CP
M_{seis}	0	kip-ft
M_{seism_inc}	100	kip-ft
DCR_{mom}	1	
m_{mom} - Factor	4	CP
Q_{allowG}	4.5	ksf

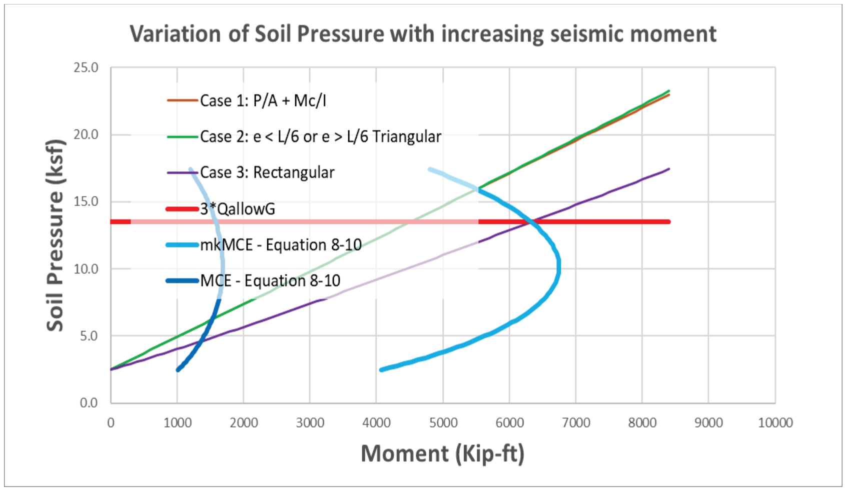


Figure C-50 Soil pressure variation with seismic overturning moment, end bay.

Proposed Acceptance Ratio with increasing Seismic Moment, End Bay, ($P_{seismic} \neq 0$)

B =	10	ft
D =	10	ft
P_G =	250	kips
P_{seism}	0	kips
P_{seism_inc}	75	kips
DCR_{Axial}	2	
m_{Axail} - Factor	4	CP
M_{seis}	0	kip-ft
M_{seism_inc}	100	kip-ft
DCR_{mom}	1	
m_{mom} - Factor	4	CP
Q_{allowG}	4.5	ksf

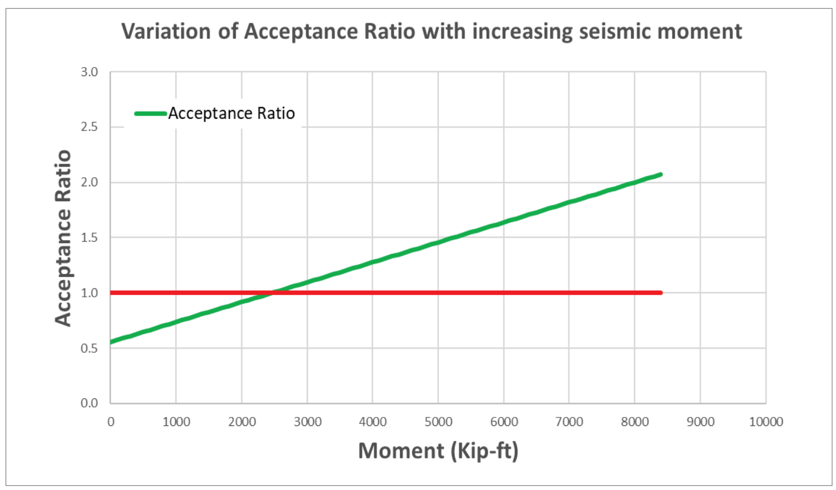


Figure C-51 Variation in acceptance ratio with increasing seismic overturning moment, end bay.

Again, the results from the two scenarios for the end bays where seismic axial load increases with overturning moment, acceptance ratio based on soil pressure is approximately 6400 kip-ft and with the new formulation is approximately 2500 kip-ft. The new proposal is conservative compared to the existing formulation and there is a lot more reserve capacity in the foundation with respect to overturning resistance for soil bearing.

Figure C-52 shows the soil pressure under footings in the end bay columns where seismic demand subtract from gravity. This is contrasted with the new acceptance ratio in Figure C-53.

Soil Pressure with increasing Seismic Moment, End Bay, ($P_{seismic} < 0$)

B =	9	ft
D =	9	ft
P_G =	250	kips
P_{seism}	0	kips
P_{seism_inc}	-50	kips
DCR_{Axial}	2	
m_{Axail} - Factor	4	CP
M_{seis}	0	kip-ft
M_{seism_inc}	100	kip-ft
DCR_{mom}	2	
m_{mom} - Factor	4	CP
Q_{allowG}	4.5	ksf

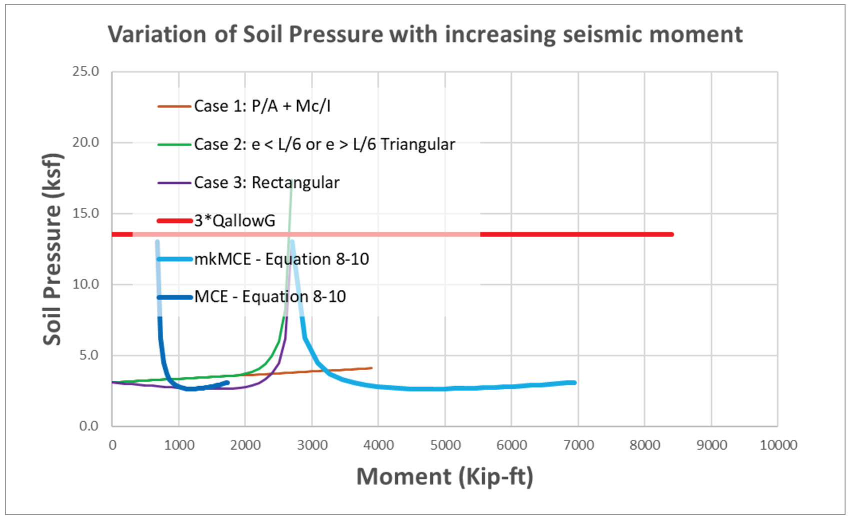


Figure C-52 Soil pressure variation with seismic overturning moment, end bay, seismic demand subtracts from gravity.

Proposed Acceptance Ratio with increasing Seismic Moment, End Bay,

($P_{seismic}$ < 0)

B =	9	ft
D =	9	ft
P_G =	250	kips
P_{seism}	0	kips
P_{seism_inc}	-50	kips
DCR_{Axial}	2	
m_{Axail} - Factor	4	CP
M_{seis}	0	kip-ft
M_{seism_inc}	100	kip-ft
DCR_{mom}	2	
m_{mom} - Factor	4	CP
Q_{allowG}	4.5	ksf

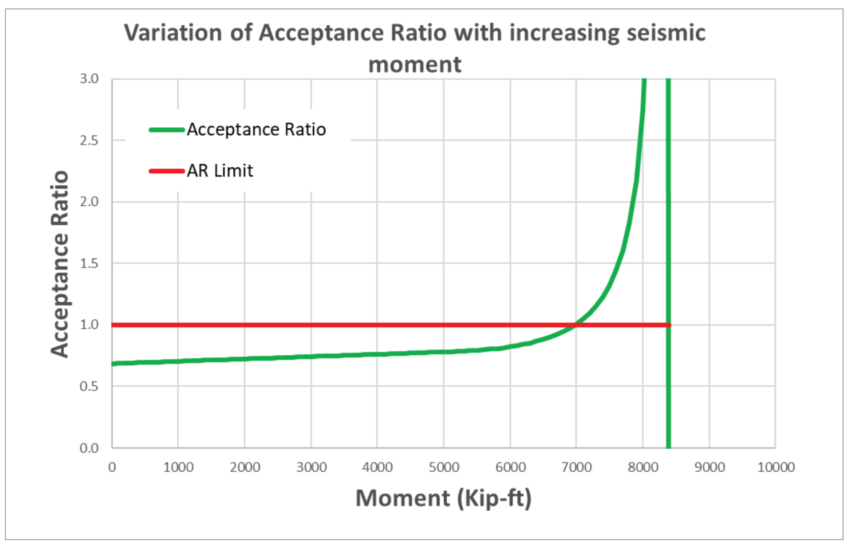


Figure C-53 Soil pressure variation with seismic overturning moment, end bay, seismic demand subtracts from gravity.

The acceptance ratio for this case has a significantly higher moment capacity acceptance criteria compared to the moment capacity when instability is reached. Since this is the end bay of a multi-bay moment frame, the soil pressure is not really a good measure of the capacity of the foundation as the superstructure transfers the load to the adjacent footing. Therefore, it is proposed that the acceptance criteria involving soil pressure of foundation stability is not required for multi-bay systems when seismic axial demand subtracts from gravity.

C.4.4 Acceptance Criteria Check - Archetype 1 (Shear Wall Example)

A similar acceptance criteria check was done on the foundation in Archetype 1 (Figure C-54), evaluated as fixed base is shown in Figure C-55 and Figure C-56.

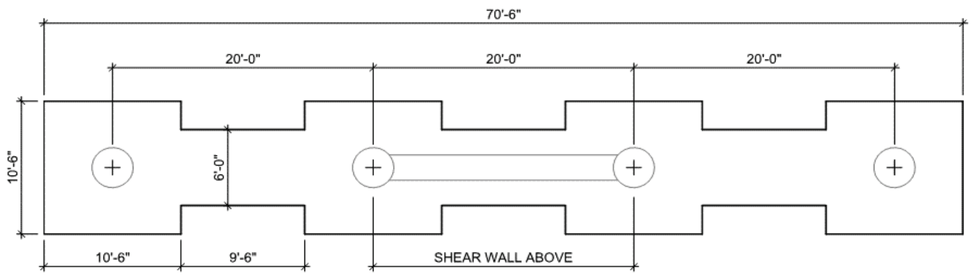


Figure C-54 Footing plan under new shear wall, Archetype 1

Soil Pressure with increasing Seismic Moment, Shear wall Footing, ($P_{seismic}$ = 0)

B =	8.7	**ft**
D =	70	**ft**
P_G =	1400	**kips**
P_{seism}	0	**kips**
P_{seism}_inc	0	**kips**
DCR_{Axial}	2	
m_{Axail} - Factor	4	**CP**
M_{seis}	0	**kip-ft**
M_{seism}_inc	12000	**kip-ft**
DCR_{mom}	1	
m_{mom} - Factor	4	**CP**
Q_{allowG}	4.5	**ksf**

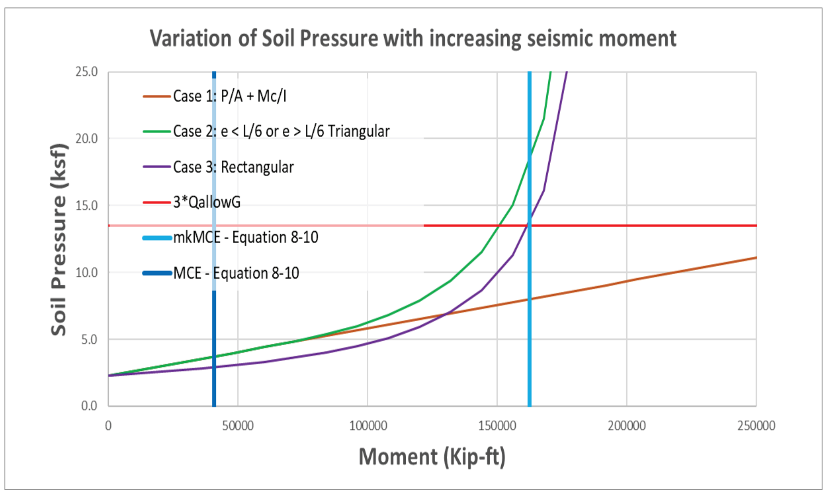


Figure C-55 Soil pressure variation with seismic overturning moment.

Proposed Acceptance Ratio with increasing seismic moment, shear wall footing ($P_{seismic}$ = 0)

B =	8.7	ft
D =	70	ft
P_G =	1400	kips
P_{seism}	0	kips
P_{seism}_inc	0	kips
DCR_{Axial}	2	
m_{Axail} - Factor	4	CP
M_{seis}	0	kip-ft
M_{seism}_inc	12000	kip-ft
DCR_{mom}	1	
m_{mom} - Factor	4	CP
Q_{allowG}	4.5	ksf

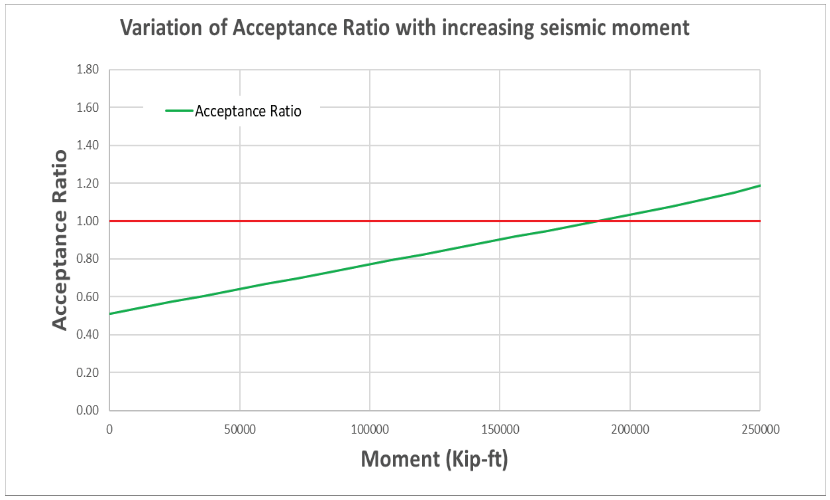


Figure C-56 Variation of acceptance ratio with seismic overturning moment.

The moment capacity corresponding to the acceptance ratio of 1.0 for the cantilever shear wall footing using the proposed formulation is now greater than that compared with the soil pressure ratio using $q_c = 3 \times q_{allowG}$.

TAKEAWAYS

Archetype Building 1 and Archetype Building 2 show different Acceptance Ratio patterns, Why? Possible reasons could be:

- Footing demands are from multiple point loads in the Archetype Building 1 example.
- Footing is assumed to have a rigid body rotation.
- High overturning demand.
- Different superstructure failure mechanisms are at play for Archetype Building 1 and Archetype Building 2. (Figure C-57 and Figure C-58)

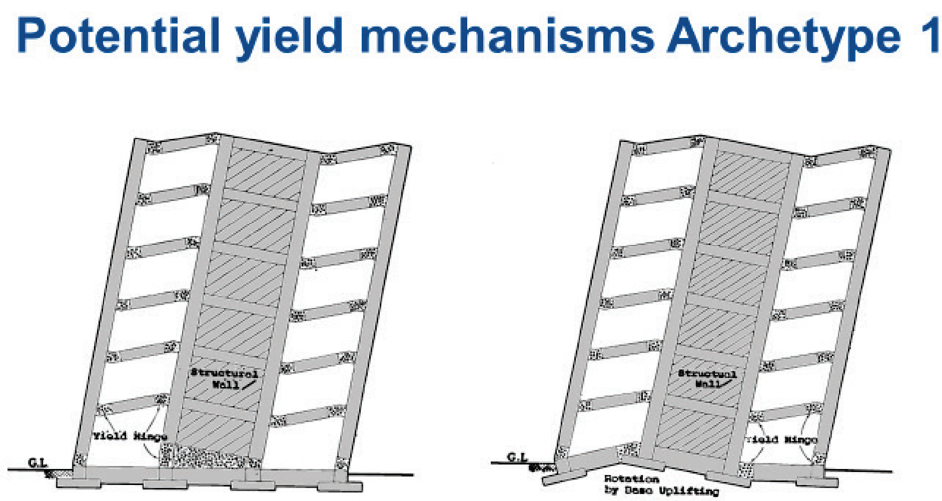


Figure C-57 Soil pressure variation with seismic overturning moment.

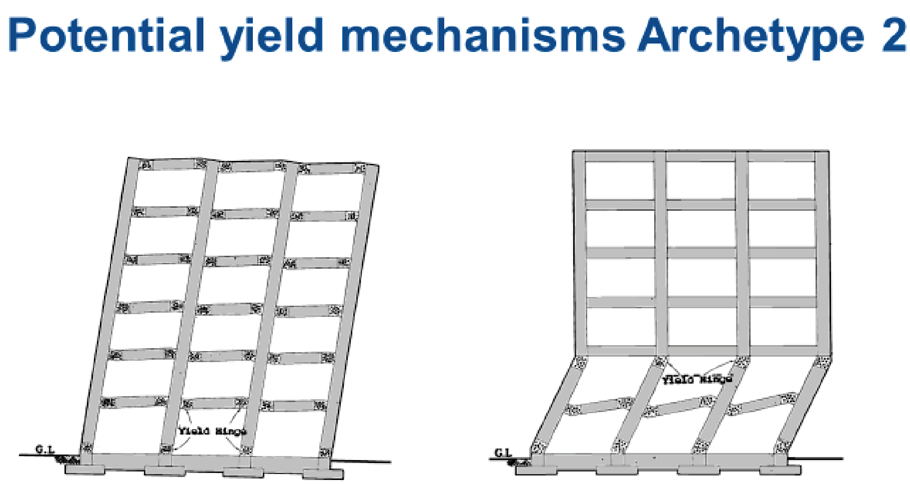


Figure C-58 Soil pressure variation with seismic overturning moment.

CONCLUSIONS

Given the fact that the newly proposed acceptance criteria may not be a true indication of the resistance capacity of the foundation, additional checks were discontinued, and instead the acceptance criteria based on the soil pressure approach was pursued. A new proposal was put forward, Proposal A, and compared with a revised version of the standard in Proposal B. Details are presented in the following sections.

C.4.5 Proposal A – Divide the pseudo force demands by "m" or DCR before foundation check

This method required the best estimate of the seismic demands to the foundation to address stability and soil bearing failure. To achieve this a new approach was investigated where the seismic demands on the foundation were divided by the *m*-factor or a DCR whichever is greater, but not both, for the soil bearing acceptance criteria. At the same time an evaluation of the footings based on the reduced soil pressures was also proposed. In order to meet the acceptance criteria for the desired performance objective, the acceptance criteria for both the soil bearing, and foundation structural component must be satisfied.

The tenets of this proposal are shown in the flow chart in Figure C-59.

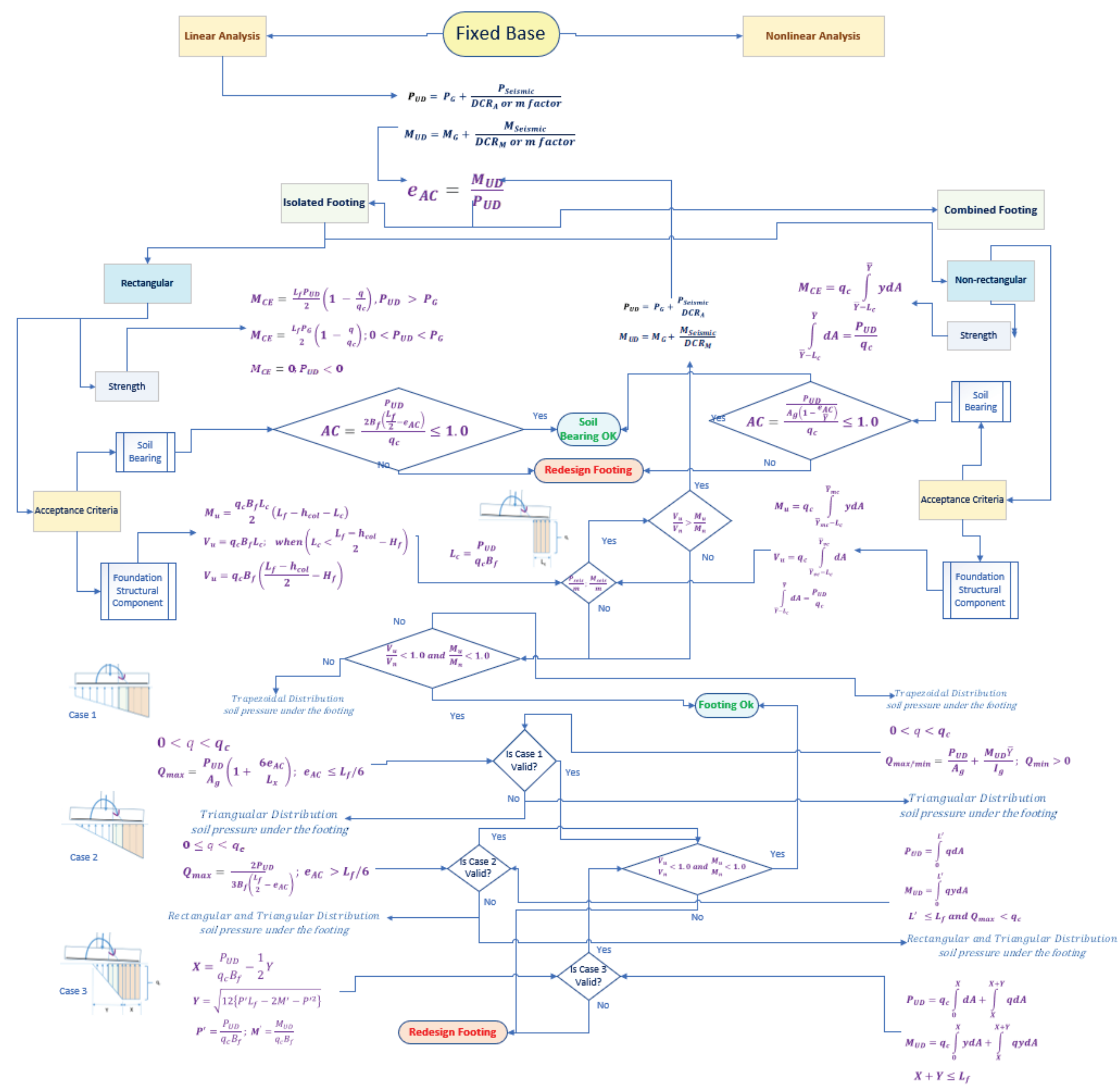


Figure C-59 Flow chart of Proposed acceptance criteria methodology for soil bearing and the structural footing

The step-by-step procedure to be followed if this proposal were adopted as outlined in the flow chart above is described in the following sections. While the acceptance criteria procedure is described for isolated spread footings, it would be equally applicable to combined footings and mat foundations.

ISOLATED SPREAD FOOTINGS

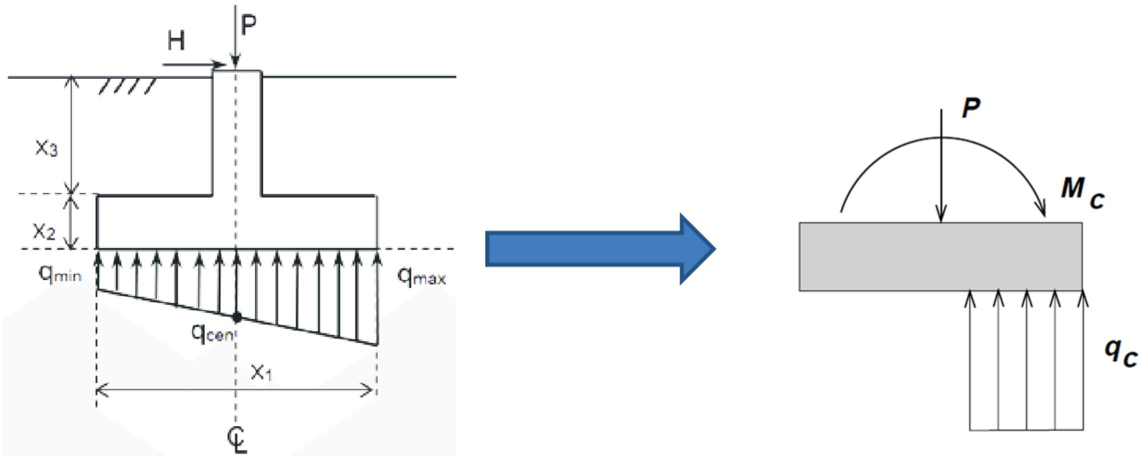


Figure C-60 Proposed method for evaluating soil bearing and footing acceptance based on anticipated soil pressure distribution under the footing.

PROPOSED ACCEPTANCE CRITERIA – SOIL BEARING

The moment capacity of the footing given by Equation 8-10 of ASCE/SEI 41-17 is nonlinear and goes to zero either when P_{UD} or the instantaneous axial load on the foundation is small or goes into tension, or the axial load is large compared with the bearing capacity of the soil.

$$M_{CE} = \frac{L_f P_{UD}}{2}\left(1 - \frac{q}{q_c}\right) \quad \text{ASCE/SEI Eq. (8-10)}$$

Where, $q = \frac{P_{UD}}{B_f L_f}$ is the vertical bearing pressure on the soil.

For this reason, when the seismic demands are not the actual demands on the footing, the results can be erroneous. When the axial seismic demand on the foundation subtracts from gravity, and the column is not yet under tension there is little to no reserve moment capacity in the foundation, but this is a transient pseudo force load and basing the acceptance criteria on this condition would show many end bay columns of moment frames or braced frames would not pass this test. Therefore, where seismic axial demand subtracts from gravity, it is recommended that the moment capacity be based on the gravity load on the footing instead.

$M_{CE} = \frac{L_f P_G}{2}\left(1 - \frac{q}{q_c}\right)$ When seismic demand subtracts from gravity.

Since the seismic axial switches between compression and tension, and the soil bearing m-factors are a function of the A_c/A_f ratio, where the *m*-factor is higher for small A_c/A_f ratios, it is further recommended to check the foundation only when seismic axial load adds to gravity. When the pseudo force demand puts the column in tension the formulations using the division of the seismic demands by the *m*-factors are already considered in the acceptance criteria and the moment capacity is taken as zero and not considered as the acceptance criteria for the footing. $M_{CE} = 0$ When pseudo axial demand on the foundation is negative.

Therefore, from above, foundation acceptance criteria should only be considered when seismic axial demand adds to gravity, and the following procedures are proposed:

Condition 1: When superstructure yielding governs the response:

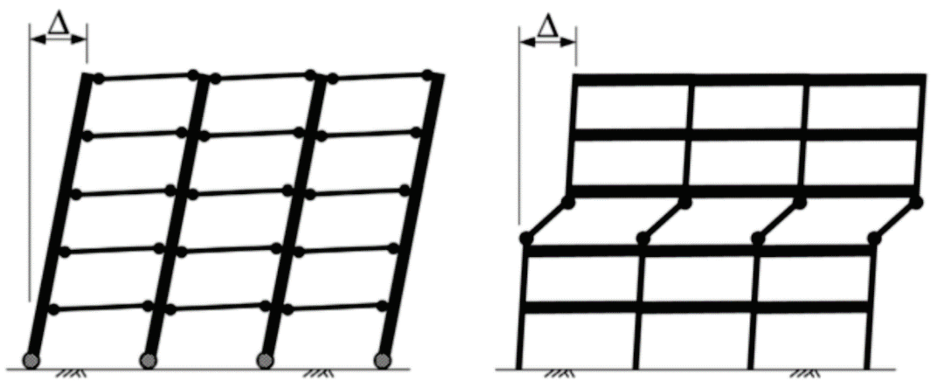


Figure C-61 Superstructure yield mechanisms limiting demands on the foundation

$$P_{UD} = P_{D+L} + \frac{P_{Seis}}{DCR_A}$$

$$M_{UD} = \frac{M_{Seis}}{DCR_M}$$

Where, DCR_A and DCR_M are the maximum DCRs affecting the moment or the axial load on the foundation from the superstructure (Figure C-61). This may be limited by $2C_1C_2$.

Condition 2: When soil yielding governs response:

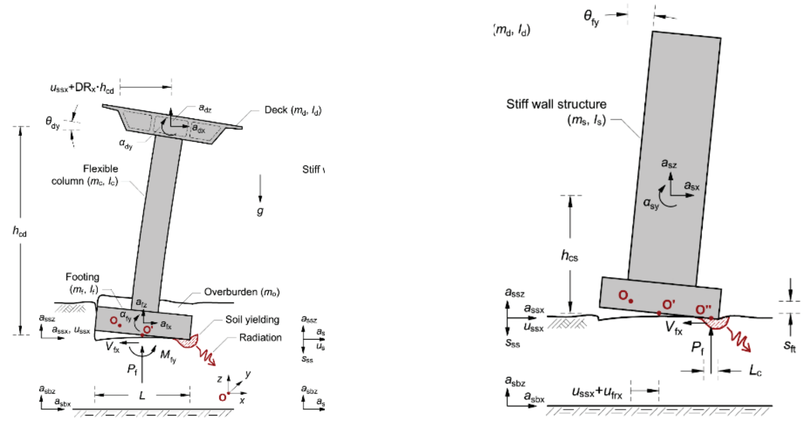


Figure C-62 Soil yielding governs the mechanisms limiting demands on the foundation

$$P_{UD} = P_{D+L} + \frac{P_{Seis}}{m}$$

$$M_{UD} = \frac{M_{Seis}}{m}$$

And defining e_{AC} as:

$$e_{AC} = \frac{M_{UD}}{P_{UD}}$$

The acceptance criteria for soil bearing can be written as:

$$AC = \frac{\frac{P_{UD}}{2B_f\left(\frac{L_f}{2} - e_{AC}\right)}}{q_c} \leq 1.0 \qquad \text{Eq. C} - 16$$

Where the numerator is written in terms of a rectangular soil pressure bulb at the end of the footing that just balances the applied moment for a prescribed axial load. See derivation for Case 3 acceptance criteria where soil pressure distribution under the footing is rectangular and where Q_{max} = q_c.

PROPOSED PROCEDURE FOR - EVALUATION OF THE FOUNDATION STRUCTURAL COMPONENT

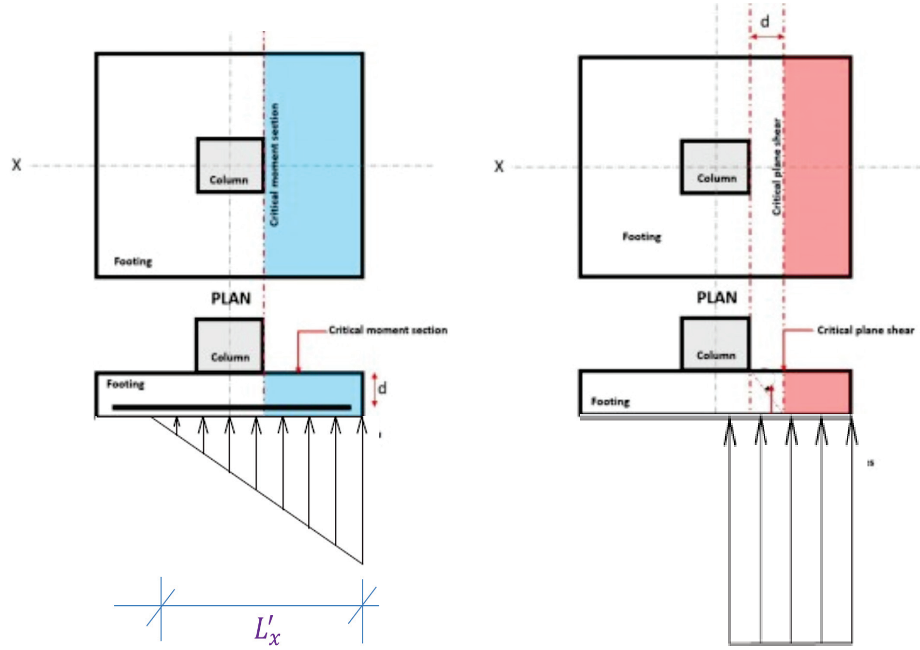


Figure C-63 Evaluation of the structural footing at each critical section for moment and shear

Foundation components are to be evaluated at each critical section using an upward soil pressure distribution under the footing (Figure C-63). This distribution varies for gravity and gravity plus seismic loads. Traditional designs evaluate demands on the footing as a superposition of forces from the axial load and moment on the footing. If the maximum soil pressure that can be resisted by the footing is q_c before excessive settlement occurs, and if the soil pressure block under the footing is rectangular over an area supporting the axial load on the footing taken from the end of the footing towards the neutral axis, this pressure distribution will generate the maximum moment or shear at the critical section. Alternatively, when superstructure yielding governs the demand on the footing, the pressure distribution can be triangular based on the axial loads and moments divided by the *m*-factor or DCR for superstructure and provided $Q_{max} < q_c$, where Q_{max} is the maximum soil pressure at the edge of the footing. The procedure to evaluate footings of rectangular geometry where the applied moment is parallel to the axis of bending of the footing, is described in the next section.

Evaluation of Rectangular Footings:

For rectangular footings, the strength demand at the critical section can be determined using an upward uniform rectangular soil pressure distribution where $q = q_c$ is applied over the critical contact area for a distance $L_c = P_{UD}/q_cB_f$ from the end of the footing towards the neutral axis as shown in Figure C-64 below.

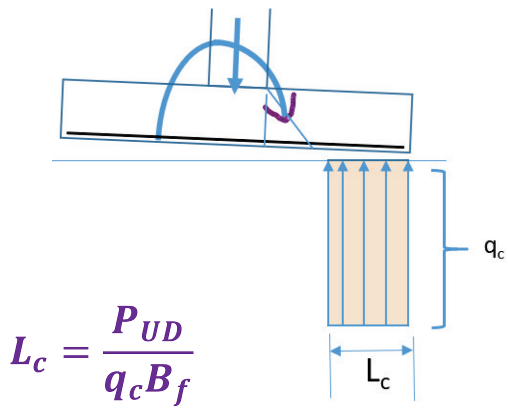


Figure C-64 Soil pressure distribution for evaluation of the structural footing

Alternatively, if the footing design fails this check, the soil bearing pressure $q < q_c$ across the width of the footing and distributed along the length of the footing resulting in the lowest strength demand at the critical section from one of the three cases below corresponding to the soil pressure distribution under the footing as shown in Figure C-65, is permitted when all the necessary conditions for that case is satisfied.

The demands (axial load and moments on the footing) are permitted to be divided by the governing *m*-factor or DCR of the superstructure to account for superstructure yielding prior to the check.

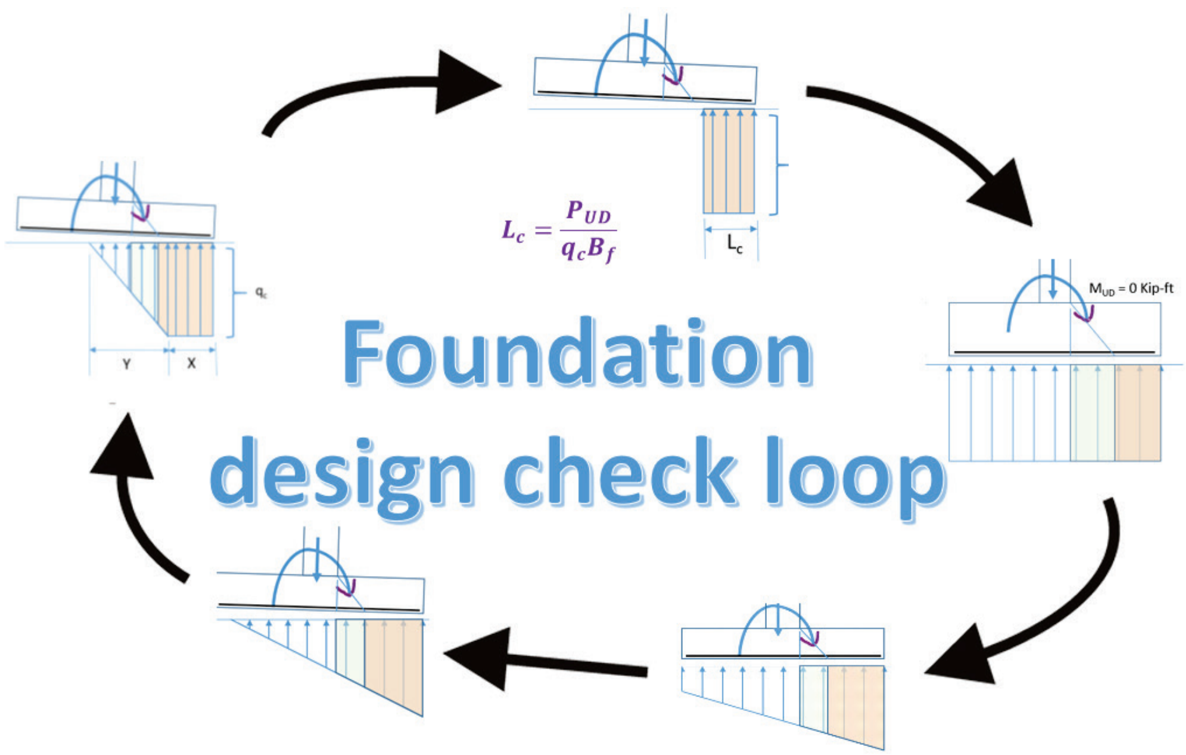


Figure C-65 Alternative soil pressure distribution for evaluation of the structural footing

Case 1: (Uniform or Trapezoidal distribution of soil pressure)

This condition as shown in Figure C-66 is applicable when the soil pressure, q, distributed along the length from Q_{max} to Q_{min} determined from Equation C-17 satisfies the requirement that no portion of the soil is in tension, $Q_{min} > 0$ and the $Q_{max} < q_c$, such that $0 \le Q_{min} < q < q_c$, where:

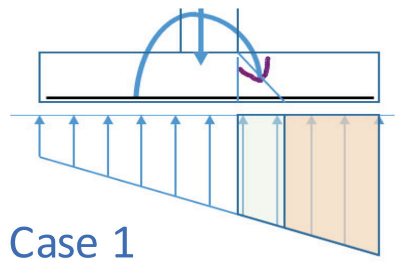


Figure C-66 Trapezoidal soil pressure distribution

$$Q_{max/min} = \frac{P_{UD}}{A_g}\left(1 \pm \frac{6e_{AC}}{L_f}\right); when\ e_{AC} \le L_f/6 \qquad \text{C-17}$$

Case 2: (Triangular distribution of soil pressure)

This condition as shown in Figure C-67 is applicable when the soil pressure, q, linearly distributed along the length goes from Q_{max} determined in Equation C-18, to 0, and satisfies the requirement that $Q_{max} < q_c$. such that $0 \le q < q_c$, where:

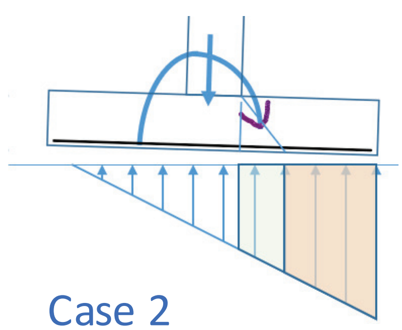


Figure C-67 Triangular soil pressure

$$Q_{max} = \frac{2P_{UD}}{3B_f\left(\frac{L_f}{2} - e_{AC}\right)};\ when\ \frac{L_f}{6} \le e_{AC} \le \frac{L_f}{2} \qquad \text{C-18}$$

$$Q_{min} = 0\ at\ L' = 3\left(\frac{L_f}{2} - e_{AC}\right) \le L_f$$

Case 3: (Rectangular and triangular distribution of soil pressure)

This condition as shown in Figure C-68 may be used if the conditions in this section are met when the soil pressure distribution of the seismic demands are not satisfied using either Case 1 or Case 2.

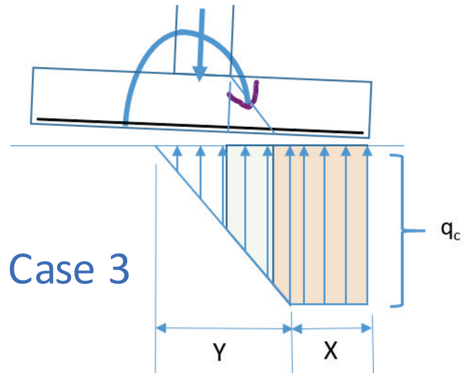


Figure C-68 Rectangular and triangular soil pressure distribution

A rectangular distribution of soil pressure with $q = q_c$ shall be applied over an area for a distance X from footing end towards the neutral axis followed by a triangular distribution over a distance Y with $q_c \geq q \geq 0$, where:

$$X = \frac{P_{UD}}{q_c B_f} - \frac{1}{2}Y \qquad \text{Eq. C} - 19$$

$$Y = \sqrt{12\{P'L_f - 2M' - P'^2\}} > 0 \qquad \text{Eq. C} - 20$$

And

$$X + Y < L_f \qquad \text{Eq. C} - 21$$

Where:

$$P' = \frac{P_{UD}}{q_c B_f}$$

and

$$M' = \frac{M_{UD}}{q_c B_f}$$

EXAMPLE: - FOUNDATION DESIGN CHECK

An example of the design demands at the critical section of an isolated footing for moment and shear where the soil pressure resistance under the footing varies from a pure axial case to where the overturning moments cause tipping over of the footing is shown in Figure C-69 through Figure C-72 below along with a verification check of the results for Archetype Building 1 for the design using the ASCE/SEI 7 in Figure C-73.

Observation of the results show that when overturning demand is resisted by purely an axial load, the ratio of design demands at the critical section of the footing for moment and shear can be less than one half of the demands when the axial load is completely resisted by a rectangular soil pressure distribution at the end of the footing. This ratio approaches 1.0 as the overturning moment approaches the tipping over moment or $(M_{UD}/m) = M_{CE}$.

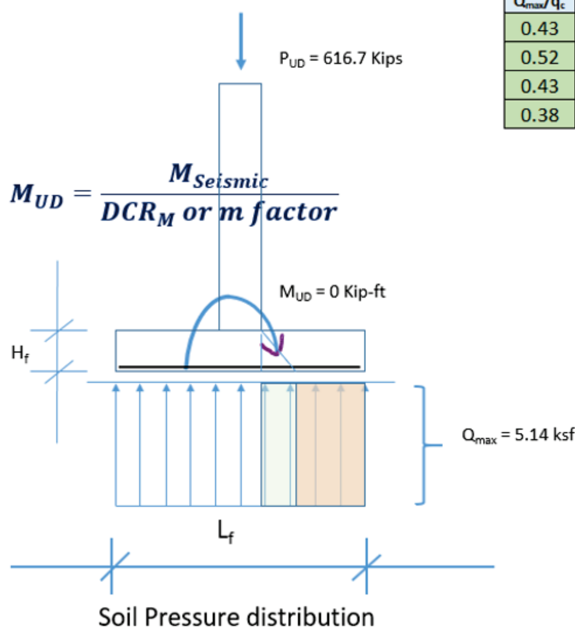
Foundation Design – Soil Pressure Uniform (Pure Axial Load)

Footing 12' x 10' x 2'

Column	b in	18
24x18	h in	24
Footing	B_f ft	10
12'x10'x2'	L_f ft	12
	H_{ftg} ft	2
Axial Load	P_G =	350
	P_{seism}	800
DCR_A	DCR_A	3
Moment	M_{seis} kip ft	0
DCR_M	DCR_m	1
	Q_{allowG} ksf	4
	f'c psi	3000
	f_y ksi	60
	#of bars	14
	Area per bar	1
	As	14
	As_{min}	8.0
	d	20.0

$$P_{UD} = P_G + \frac{P_{Seismic}}{DCR_A \ or \ m \ factor}$$

$$M_{UD} = \frac{M_{Seismic}}{DCR_M \ or \ m \ factor}$$



Summary Results Table

Soil Pressure Ratio Q_{max}/q_c	Fdn. Moment Demand / Moment Capacity	Fdn. Shear Demand / Shear Capacity	Fdn. Moment DCR/Fdn. Shear DCR	m Factor	Perfomance Ojective
0.43	0.49	0.65	0.76	Eq. Using DCR_A & DCR_M	
0.52	0.60	0.79	0.76	2	IO
0.43	0.49	0.65	0.76	3	LS
0.38	0.44	0.58	0.76	4	CP

$$e_{AC} = \frac{M_{UD}}{P_{UD}}$$

$$Q_{max} = \frac{P_{UD}}{2B_f\left(\frac{L_f}{2} - e_{AC}\right)}$$

Rectangular Distribution
soil pressure under the footing

Transition Zone

$$Q_{max} = \frac{P_{UD}}{A_g}\left(1 + \frac{6e_{AC}}{L_x}\right); \ e_{AC} \le L_f/6$$

$$Q_{max} = \frac{2P_{UD}}{3B_f\left(\frac{L_f}{2} - e_{AC}\right)}; \ e_{AC} > L_f/6$$

Triangualar Distribution
soil pressure under the footing

Footing Design Demands @ Critical Section			
	Maximum $Q_{max} = q_c$	Actual Demand	Ratio
Moment k-f	1500	642	0.43
Shear kips	400	171	0.43

Figure C-69 Moment and Shear Demand Ratios for Footings Under Pure Axial Load

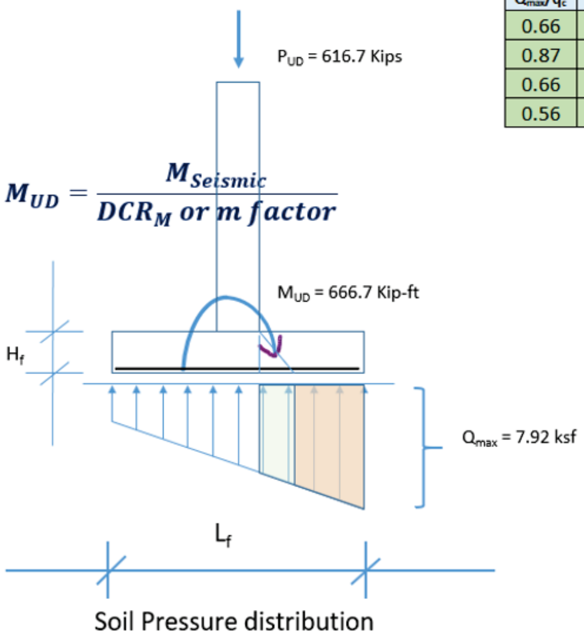
Foundation Design – Soil Pressure Trapezoidal

Footing 12' x 10' x 2'

Column	b in	18
24x18	h in	24
Footing	B_f ft	10
12'x10'x2'	L_f ft	12
	H_{ftg} ft	2
Axial Load	P_G =	350
	P_{seism}	800
DCR_A	DCR_A	3
Moment	M_{seis} kip ft	2000
DCR_M	DCR_m	3
	Q_{allowG} ksf	4
	f'_c psi	3000
	f_y ksi	60
	#of bars	14
	Area per bar	1
	As	14
	As min	8.0
	d	20.0

$$P_{UD} = P_G + \frac{P_{Seismic}}{DCR_A \; or \; m \; factor}$$

$$M_{UD} = \frac{M_{Seismic}}{DCR_M \; or \; m \; factor}$$



Summary Results Table

Soil Pressure Ratio Q_{max}/q_c	Fdn. Moment Demand / Moment Capacity	Fdn. Shear Demand / Shear Capacity	Fdn. Moment DCR/Fdn. Shear DCR	m Factor	Perfomance Ojective
0.66	0.68	0.91	0.76	Eq. Using DCR_A & DCR_M	
0.87	0.89	1.17	0.76	2	IO
0.66	0.68	0.91	0.76	3	LS
0.56	0.58	0.77	0.76	4	CP

$$e_{AC} = \frac{M_{UD}}{P_{UD}}$$

$$Q_{max} = \frac{P_{UD}}{2B_f\left(\frac{L_f}{2} - e_{AC}\right)}$$

Rectangular Distribution
soil pressure under footing

Transition Zone

$$Q_{max} = \frac{P_{UD}}{A_g}\left(1 + \frac{6e_{AC}}{L_x}\right); \; e_{AC} \le L_f/6$$

$$Q_{max} = \frac{2P_{UD}}{3B_f\left(\frac{L_f}{2} - e_{AC}\right)}; \; e_{AC} > L_f/6$$

Triangualar Distribution
Under the footing

Footing Design Demands @ Critical	Maximum $Q_{max} = q_c$	Actual Demand	Ratio
Moment k-f	1500	893	0.60
Shear kips	400	238	0.60

Figure C-70 Moment and Shear Demand Ratios for Footings with Axial Load and Low Moment where no Gapping Occurs

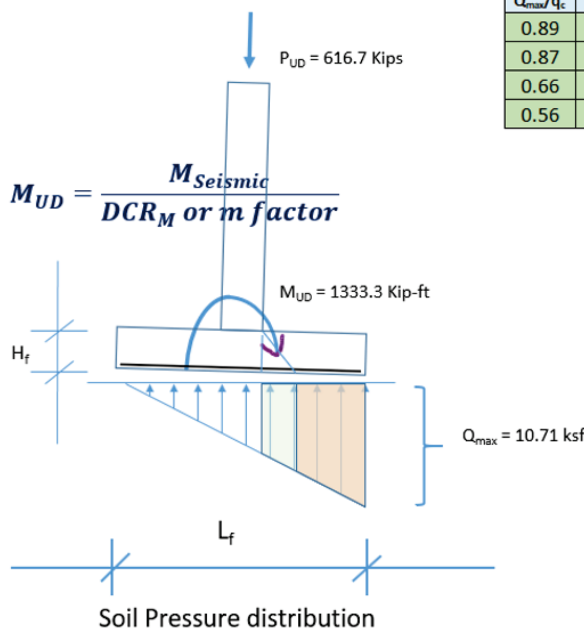
Foundation Design – Soil Pressure Triangular

Footing 12' x 10' x 2'

Column	b in	18
24x18	h in	24
Footing	B_f ft	10
12'x10'x2'	L_f ft	12
	H_{ftg} ft	2
Axial Load	P_G =	350
	P_{seism}	800
DCR_A	DCR_A	3
Moment	M_{seis} kip ft	2000
DCR_M	DCR_m	1.5
	Q_{allowG} ksf	4
	f'c psi	3000
	f_y ksi	60
	#of bars	14
	Area per bar	1
	As	14
	As min	8.0
	d	20.0

$$P_{UD} = P_G + \frac{P_{Seismic}}{DCR_A \text{ or } m \text{ factor}}$$

$$M_{UD} = \frac{M_{Seismic}}{DCR_M \text{ or } m \text{ factor}}$$



Summary Results Table

Soil Pressure Ratio Q_{max}/q_c	Fdn. Moment Demand / Moment Capacity	Fdn. Shear Demand / Shear Capacity	Fdn. Moment DCR/Fdn. Shear DCR	m Factor	Perfomance Ojective
0.89	0.88	1.16	0.76	Eq. Using DCR_A & DCR_M	
0.87	0.89	1.17	0.76	2	IO
0.66	0.68	0.91	0.76	3	LS
0.56	0.58	0.77	0.76	4	CP

$$e_{AC} = \frac{M_{UD}}{P_{UD}}$$

$$Q_{max} = \frac{P_{UD}}{2B_f\left(\frac{L_f}{2} - e_{AC}\right)}$$

Rectangular Distribution
soil pressure under the footing

Transition Zone

$$Q_{max} = \frac{P_{UD}}{A_g}\left(1 + \frac{6e_{AC}}{L_x}\right);\ e_{AC} \le L_f/6$$

$$Q_{max} = \frac{2P_{UD}}{3B_f\left(\frac{L_f}{2} - e_{AC}\right)};\ e_{AC} > L_f/6$$

Triangualar Distribution
soil pressure under the footing

Footing Design Demands @ Critical			
	Maximum $Q_{max} = q_c$	Actual Demand	Ratio
Moment k-f	1500	1145	0.76
Shear kips	400	305	0.76

Figure C-71 Moment and Shear Demand Ratios for Footings with Axial Load and Moment Producing Gapping

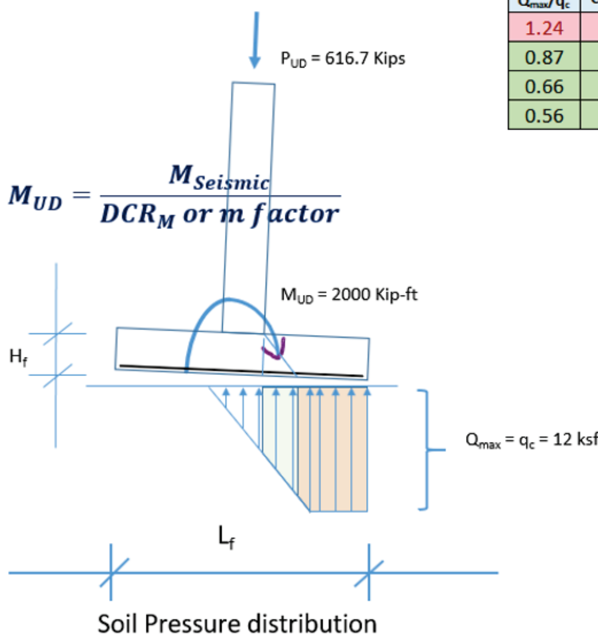
Foundation Design – Soil Pressure in Transition Zone

Footing 12' x 10' x 2'

Column	b in	18
24x18	h in	24
Footing	B_f ft	10
12'x10'x2'	L_f ft	12
	H_{ftg} ft	2
Axial Load	P_G =	350
	P_{seism}	800
DCR_A	DCR_A	3
Moment	M_{seis} kip ft	2000
DCR_M	DCR_m	1
	Q_{allowG} ksf	4
	f'_c psi	3000
	f_y ksi	60
	#of bars	14
	Area per bar	1
	As	14
	As_{min}	8.0
	d	20.0

$$P_{UD} = P_G + \frac{P_{Seismic}}{DCR_A \text{ or } m\ factor}$$

$$M_{UD} = \frac{M_{Seismic}}{DCR_M \text{ or } m\ factor}$$



Summary Results Table

Soil Pressure Ratio Q_{max}/q_c	Fdn. Moment Demand / Moment Capacity	Fdn. Shear Demand / Shear Capacity	Fdn. Moment DCR/Fdn. Shear DCR	m Factor	Perfomance Ojective
1.24	1.11	1.50	0.74	Eq. Using DCR_A & DCR_M	
0.87	0.89	1.17	0.76	2	IO
0.66	0.68	0.91	0.76	3	LS
0.56	0.58	0.77	0.76	4	CP

$$e_{AC} = \frac{M_{UD}}{P_{UD}}$$

$$Q_{max} = \frac{P_{UD}}{2B_f\left(\frac{L_f}{2} - e_{AC}\right)}$$

Rectangular Distribution
soil pressure under the footing

Transition Zone

$$Q_{max} = \frac{P_{UD}}{A_g}\left(1 + \frac{6e_{AC}}{L_x}\right);\ e_{AC} \le L_f/6$$

$$Q_{max} = \frac{2P_{UD}}{3B_f\left(\frac{L_f}{2} - e_{AC}\right)};\ e_{AC} > L_f/6$$

Triangualar Distribution
soil pressure under the footing

Footing Design Demands @ Critical			
	Maximum $Q_{max} = q_c$	Actual Demand	Ratio
Moment k-ft	1500	1452	0.97
Shear kips	400	396	0.99

Figure C-72 Moment and Shear Demand Ratios for Footings with Axial Load and High Moment with Soil Yielding

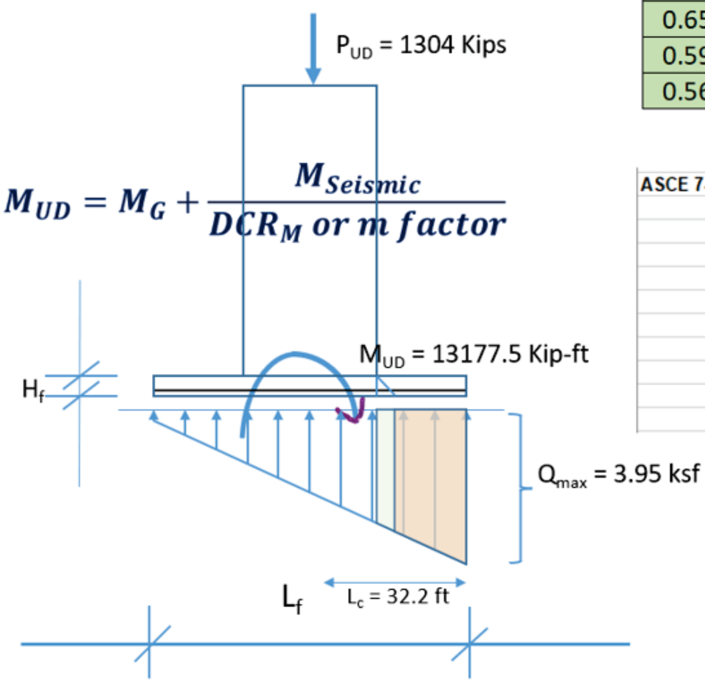
Results for Archetype Building 1 – ASCE/SEI 7-10

Footing 70.5' x 8.7' x 4.5'

$$P_{UD} = P_G + \frac{P_{Seismic}}{DCR_A \; or \; m \; factor}$$

Column	b in	12
360x12	h in	360
Footing	B_f ft	8.7
70.5'x8.7'x4.	L_f ft	70.5
	H_{ftg} ft	**4.5**
Axial Load	P_G =	1304
	P_{seism}	0
DCR_A	DCR_A	1
Moment	M_{seis} kip ft	13177.5
DCR_M	DCR_m	1
	Q_{allowG} ksf	1.55
	f'_c psi	3000
	f_y ksi	60
	#of bars	28
	Area per bar	1
	As	28
	As $_{min}$	17.4
	d	50.0

$$M_{UD} = M_G + \frac{M_{Seismic}}{DCR_M \; or \; m \; factor}$$



Soil Pressure distribution

Summary - Foundation Acceptance Criteria

Soil Pressure Ratio Q_{max}/q_c	Footing M_u/M_n	Footing V_u/V_n	Ratio (M_u/M_n) / (V_u/V_n)	m Factor	Perfomance Level
0.85	0.98	0.87	1.13	Eq. Using DCR_A & DCR_M	
0.65	0.78	0.69	1.12	2	IO
0.59	0.71	0.64	1.12	3	LS
0.56	0.68	0.61	1.12	4	CP

ASCE 7-10 Footing design, bearing pressure

M =	13177.5	k-ft, ASD (ETABS) with 25% reduction	
P, total =	1304	kips (sum of dead load at all 4 cols)	
footing width, B =	8.7	ft	
footing length, L =	70.5	ft	
L/ 6 =	11.8	ft	
M/P = e =	10.1	ft	e> L/6
e' =	25.1	ft	
q max =	3.98	ksf	<4.66 ksf okay
DCR =	0.85		

Footing Design Demands @ Critical Section

	Maximum $Q_{max} = q_c$	Actual Demand	Ratio
Moment k-ft	8295	6429	0.78
Shear kips	651	495	0.76

Figure C-73 Verification Check of Footing Design Demands for Archetype 1 for the Two Methods

C.4.6 Proposal B – Keep the General Philosophy for Acceptance Criteria but Revise for Usability and Original Intent

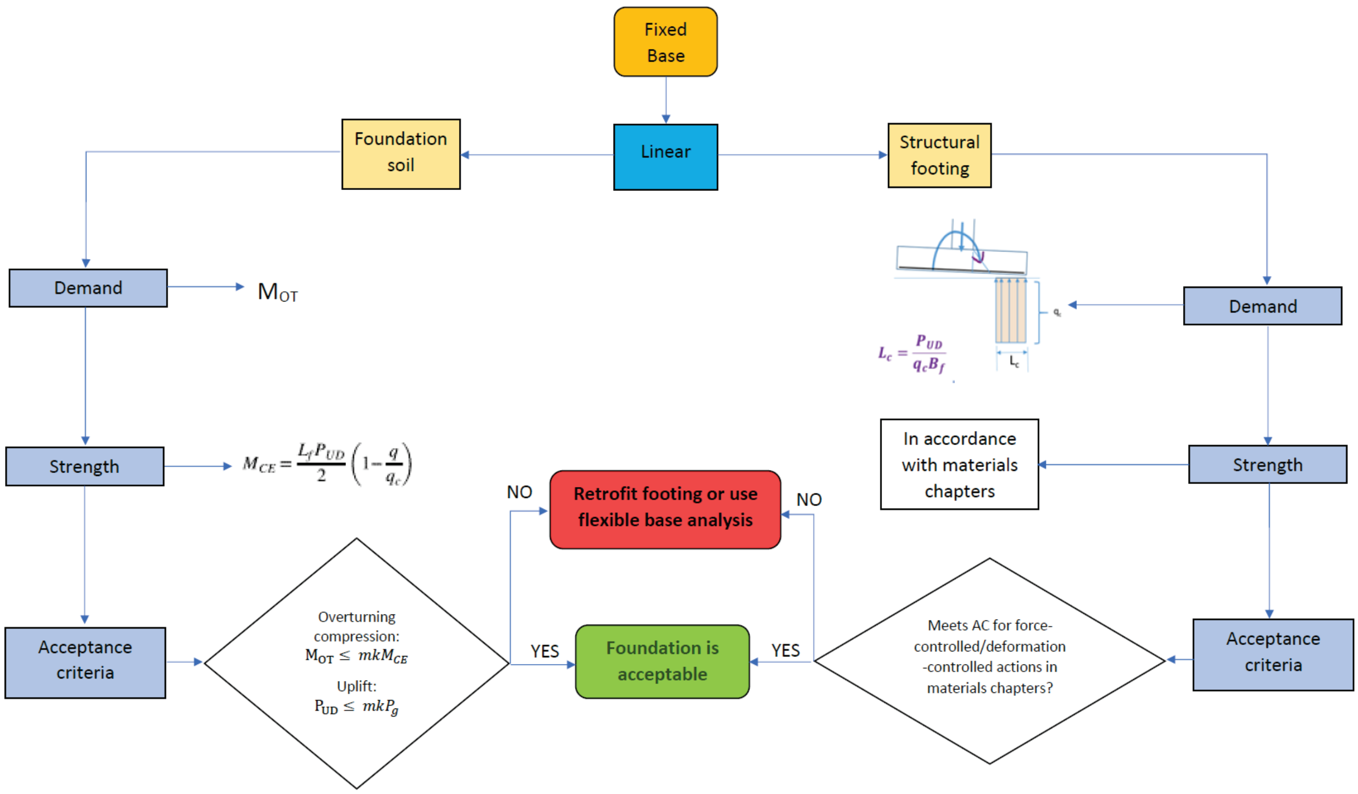


Figure C-74 Flowchart for Proposal B

This proposal expands the current check in ASCE/SEI 41 to explicitly check the foundation structural element. See flowchart shown in Figure C-74. Other aspects of the foundation evaluation using ASCE/SEI 41 remain unchanged except when the seismic overturning and gravity load on the foundation is predominantly an axial load with a small moment.

C.4.7 Comparison of Outcomes from Proposal A and Proposal B

To obtain consensus in adopting Proposal A, it was necessary to quantify the differences between the new procedure formulated in Proposal A and a clarified version of the existing method in Proposal B. To achieve this, two options were delineated, called Option 1 and Option 2. The methodology and acceptance criteria for Option 1, conforms with the methodology in ASCE/SEI 41 where the element capacity is multiplied by the *m*-factor in the acceptance criteria check. In Option 2 the pseudo force demand on the element is divided by "m" in the acceptance criteria check. The methodologies used for the two options is given below:

OPTION 1

The acceptance criteria for overturning action for Option 1 is based on the following:

- Upper bound value for soil bearing capacity q_c is retained, or q_c = 2(3q_{allow})
- Foundation overturning capacity is calculated as:

$$M_{CE} = \frac{L_f P_{UD}}{2}\left(1 - \frac{q}{q_c}\right) \qquad \text{ASCE/SEI 41-17 (Eq 8-10)}$$

$where$:

$$P_{UD} = P_G \pm \frac{P_E}{DCR}$$ (DCR is as defined in Eq. 7-16 of ASCE 41-17, and is limited to $2C_1C_2$)

and

$$q = \frac{P_{UD}}{B_f L_f}$$

Acceptance Criteria for Overturning Action: - Soil Bearing

Overturning moment demand on the foundation M_{OT} is less than *m*-factor times knowledge factor times the capacity, or

$$M_{OT} \leq m\kappa M_{CE}$$

When overturning results in compression on entire footing area, the acceptance criteria is given as:

$$P_{UD} \leq m\kappa q_c A_f$$

When overturning results in an axial upward force P_{UD}, acceptance criteria is given as:

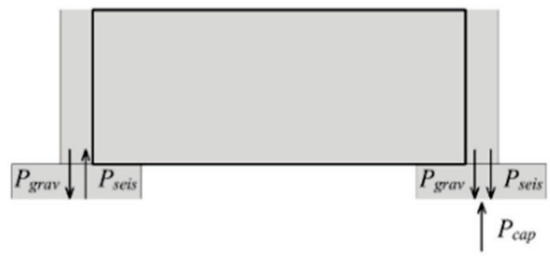
$$P_{UD} \leq m\kappa P_g$$

Acceptance Criteria for Overturning Action: - Foundation Structural Component

Foundation design check is based on rectangular soil pressure distribution where q = q_c is applied over the critical contact area for a distance $L_c = P_{UD}/q_cB_f$ from the end of the footing towards the neutral axis as shown in Figure C-64.

The applicable *m*-factors when the resulting axial load on the footing from gravity and seismic overturning results in compression or uplift is given in the Table C-18 below.

Table C-18 m-factors for axial uplift and compression



Overturning Action	Performance Level		
	IO	LS	CP
Compression	2	3	4
Uplift	4	6	8

OPTION 2

The acceptance criteria for overturning action for Option 2 is based on the following:

Expected values of q_c are used, or $q_c = 3q_{allow}$

Pseudo force demands for compression load combinations are converted to an equivalent pressure block:

Convert pseudo force demands to expected forces and then to an equivalent soil pressure block defined as q_{UD}.

$$P_{UD} = P_G + \frac{P_E}{DCR_A} ;$$

No limit on DCR can equal "m" or DCR_A = m, for coupled column axial actions

$$M_{UD} = M_G + \frac{M_E}{DCR_m} \ (a) \ or \ M_{UD} = M_G + \frac{M_E}{m} \ (b)$$

Since M_E is divided by "m" or DCR_M, no additional *m*-factor reduction is permitted, and

$$q_{UD} = \frac{P_{UD}}{2B_f\left(\frac{L_f}{2} - e_{AC}\right)}$$

Where,

$$e_{AC} = \frac{M_{UD}}{P_{UD}}$$

m-factors are the same as used for Option 1.

Acceptance Criteria for Overturning Action:- Soil Bearing

Equivalent soil pressure block demand is less than soil bearing capacity and is only applied when seismic axial load adds to gravity.

$$q_{UD} \leq \kappa q_C$$

When overturning results in an axial upward force P_{UD}:

$$P_{UD} \leq m\kappa P_g$$

Acceptance Criteria for Overturning Action:- Foundation Structural Component

Foundation design check is based on a soil pressure distribution beneath the footing determined as described earlier, and where the pseudo force demands are reduced by the *m*-factor.

BENEFITS OF EACH OPTION

Option 1

This has already been accepted by ASCE/SEI 41.

m-factors have already been established based on test results for rocking behavior.

Option 2

Applicable to all footing types and methods.

Do not need to use upper bound for overturning and expected values when checking the footing.

If the same value of q_c is used for soil capacity, results converge at the same acceptance criteria limit whether using Option 1 or Option 2:

$M_{OT} < m\kappa M_{CE}$ - Option 1

$q_{UD}/\kappa q_c < 1$ - Option2

DRAWBACKS OF EACH OPTION

Option 1

Use of upper bound values for soil bearing q_c gives unconservative results for soil bearing in some cases (Archetype Building 2).

Process needs to be tweaked for footings under different LFRS.

Option 2

May need to recalibrate (increase) the *m*-factors to results from Archetype Buildings 1 and 2 and other case studies because expected bearing capacity is used instead of upper bound.

COMPARISON OF OPTION 1 AND OPTION 2

To decide between the two options, a spreadsheet was created to show the similarities and differences between the options. In addition, an alternate procedure similar to Option 2 was proposed, and the outcomes from a different moment frame example than Archetype 2 was also used to compare the results from the two options.

In Option 1, the provisions in the standard are applied closely as written, and changes made where the standard does not give specific guidance. In Option 2, the rules for the acceptance criteria are modified such that the seismic demands are divided by the *m*-factor or a DCR prior to performing the acceptance criteria check.

Example 1: Analysis of Results from Spreadsheet Model

A 10 feet × 12 feet footing example was used to quantify the differences between the options. Three cases are presented in Figure C-75 through Figure C-78. The moment demands and *m*-factor was varied between the cases. The axial load is divided by DCR in both cases, so the axial load used in the check is the same for both Options.

For case 1, the *m*-factor was set equal to DCR_M. For this case, the acceptance criteria using Option 2 was higher than Option 1. For the second case the *m*-factor and DCR_M are different, and a division by "m" after reveals that the results between the two methods are close, but slightly different 0.35 vs 0.39 for the same q_c. For the case where $M_{OT} = M_{CE}$ for the same value of q_c, the acceptance ratio is close to 1.0, and both methods converge.

Comparison of Options

- **Case 1 (m = DCR_M)**

M_{OT} = 3000 kip-ft

m = 4.0

DCR_A = 2.0

DCR_M = 4.0

Footing	B_f ft	10
12'x10'x2'	L_f ft	12
	H_{ftg} ft	2
Axial Load	P_G =	350
	P_{seism}	800
DCR_A	DCR_A	2
Moment	M_{seis} kip ft	3000
DCR_M	DCR_m	4
	Q_{allowG} ksf	4
Option 1	m	4
Option 2	m	-

Equations used

$$P_{UD} = P_G + P_{seism}/DCR_A$$

$$M_{UD} = M_{OT}/DCR_M$$

$$e_{AC} = \frac{M_{UD}}{P_{UD}}$$

$$e_{EC} = \frac{1}{2}\left(L_f - \frac{P_{UD}}{B_f q_c}\right)$$

$$q_{UD} = \frac{P_{UD}}{2B_f\left(\frac{L_f}{2} - e_{AC}\right)}$$

$$M_{CE} = \frac{L_f P_{UD}}{2}\left(1 - \frac{q}{q_c}\right)$$

														Acceptance Ratio	
	P_{UD}	M_{OT}	DCR_A	DCR_M	e_{AC}	e_{CE}	q	q_{allow}	q_c	M_{CE}	m	κ	$m\kappa M_{CE}$	$M_{OT}/m\kappa M_{CE}$	q/q_c
Option 1 (Upper Bound	750	3000	2	-	-	-	6.25	4	24	3328	4	1	13313	0.23	0.26
Option 1 (Expected)	750	3000	2	-	-	-	6.25	4	12	2156	4	1	8625	0.35	0.52
	P_{UD}	M_{UD}	DCR_A	DCR_M	e_{AC}	e_{CE}	q_{UD}	q_{allow}	q_c	M_{CE}	m	κ	κM_{CE}	$M_{UD}/\kappa M_{CE}$	q_{UD}/q_c
Option 2	750	750	2	4	1.00	2.88	7.50	4	12	2156	-	1	2156	0.35	0.63

Figure C-75 m = DCR_M

Comparison of Options

- **Case 2 (m = 4*DCR_M)**

M_{OT} = 3000 kip-ft

m = 4.0

DCR_A = 2.0

DCR_M = 1.0

Footing	B_f ft	10
12'x10'x2'	L_f ft	12
	H_{ftg} ft	2
Axial Load	P_G =	350
	P_{seism}	800
DCR_A	DCR_A	2
Moment	M_{seis} kip ft	3000
DCR_M	DCR_m	1
	Q_{allowG} ksf	4
Option 1	m	4
Option 2	m	-

Equations used

$$P_{UD} = P_G + P_{seism}/DCR_A$$

$$M_{UD} = M_{OT}/DCR_M$$

$$e_{AC} = \frac{M_{UD}}{P_{UD}}$$

$$e_{EC} = \frac{1}{2}\left(L_f - \frac{P_{UD}}{B_f q_c}\right)$$

$$q_{UD} = \frac{P_{UD}}{2B_f\left(\frac{L_f}{2} - e_{AC}\right)}$$

$$M_{CE} = \frac{L_f P_{UD}}{2}\left(1 - \frac{q}{q_c}\right)$$

														Acceptance Ratio	
	P_{UD}	M_{OT}	DCR_A	DCR_M	e_{AC}	e_{CE}	q	q_{allow}	q_c	M_{CE}	m	κ	$m\kappa M_{CE}$	$M_{OT}/m\kappa M_{CE}$	q/q_c
Option 1 (Upper Bound	750	3000	2	-	-	-	6.25	4	24	3328	4	1	13313	0.23	0.26
Option 1 (Expected)	750	3000	2	-	-	-	6.25	4	12	2156	4	1	8625	0.35	0.52
	P_{UD}	M_{UD}	DCR_A	DCR_M	e_{AC}	e_{CE}	q_{UD}	q_{allow}	q_c	M_{CE}	m	κ	κM_{CE}	$M_{UD}/\kappa M_{CE}$	q_{UD}/q_c
Option 2	750	3000	2	1	4.00	2.88	18.75	4	12	2156	-	1	2156	1.39	1.56

q_{UD}/mq_c = 0.39

Figure C-76 m = 4 x DCR_M

Comparison of Options

- **Case 3 ($m = DCR_M$)**

  M_{OT} = 8500 kip-ft

  m = 4.0

  DCR_A = 2.0

  DCR_M = 4.0

Footing 12'x10'x2'	B_f ft	10
	L_f ft	12
	H_{ftg} ft	2
Axial Load	P_G =	350
	P_{seism}	800
DCR_A	DCR_A	2
Moment	M_{seis} kip ft	8500
DCR_M	DCR_m	4
	Q_{allowG} ksf	4
Option 1	m	4
Option 2	m	-

Equations used

$$P_{UD} = P_G + P_{seism}/DCR_A$$

$$M_{UD} = M_{OT}/DCR_M$$

$$e_{AC} = \frac{M_{UD}}{P_{UD}}$$

$$e_{EC} = \frac{1}{2}\left(L_f - \frac{P_{UD}}{B_f q_c}\right)$$

$$q_{UD} = \frac{P_{UD}}{2B_f\left(\frac{L_f}{2} - e_{AC}\right)}$$

$$M_{CE} = \frac{L_f P_{UD}}{2}\left(1 - \frac{q}{q_c}\right)$$

														Acceptance Ratio	
	P_{UD}	M_{OT}	DCR_A	DCR_M	e_{AC}	e_{CE}	q	q_{allow}	q_c	M_{CE}	m	κ	$m\kappa M_{CE}$	M_{OT}/mkM_{CE}	q/q_C
Option 1 (Upper Bound	750	8500	2	-	-	-	6.25	4	24	3328	4	1	13313	0.64	0.26
Option 1 (Expected)	750	8500	2	-	-	-	6.25	4	12	2156	4	1	8625	0.99	0.52
	P_{UD}	M_{UD}	DCR_A	DCR_M	e_{AC}	e_{CE}	q_{UD}	q_{allow}	q_c	M_{CE}	m	κ	κM_{CE}	M_{UD}/kM_{CE}	q_{UD}/q_C
Option 2	750	2125	2	4	2.83	2.88	11.84	4	12	2156	-	1	2156	0.99	0.99

Figure C-77 $m = DCR_M$, and $M_{OT} = M_{CE}$ when $q_c = 3 \times q_{allow}$

A comparison of the acceptance ratio for the two options for the same axial load and soil bearing capacity q_c is shown in Figure C-78. From the figure the acceptance ratio for Option 2, is higher than Option 1 for ratios less than 1 but has a shallower slope and increases exponentially beyond the point where the ratio is greater than 1.0.

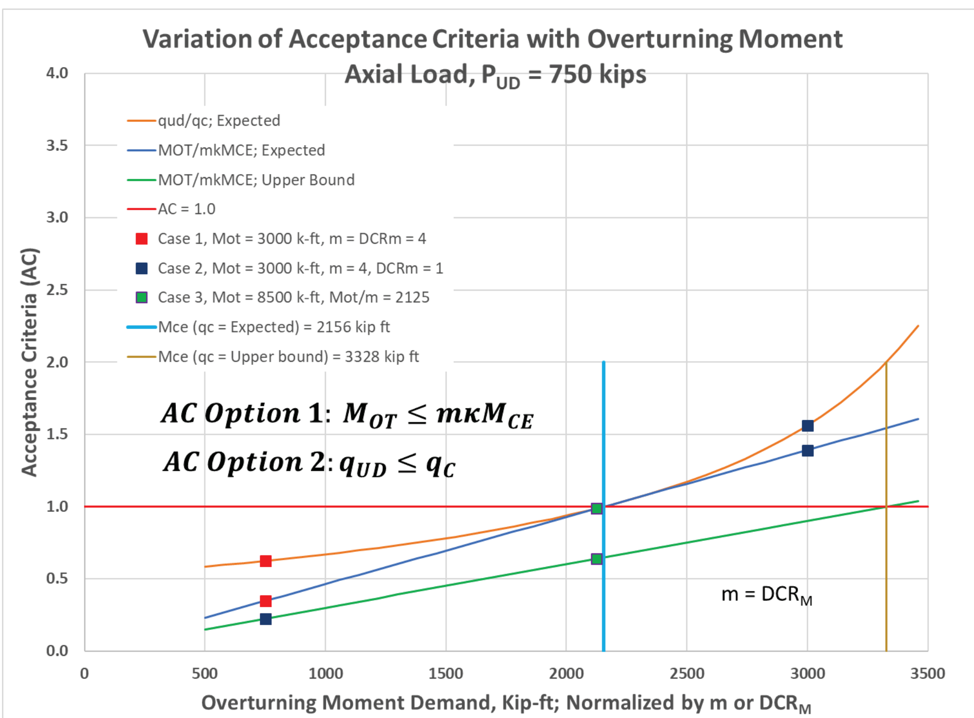


Figure C-78 Variation of acceptance criteria between Option 1 and Option 2.

From the three example cases, it is clear that both options result in the same final outcome. Either the acceptance criteria is satisfied or is not satisfied when DCR_A the reduction in seismic demand of axial load from superstructure yielding is the same and the same bearing capacity of soil q_c is used. If DCR_A used in Option 2 uses an m > DCR_A, the acceptance criteria using Option 2 is more conservative than Option 1, as the higher axial loads adds stability to the footing till the axial load on the footing starts to approach around 80% of the ultimate bearing capacity of the footing.

The variation in acceptance ratio with axial load for the two options is shown in Figure C-79 and Figure C-80.

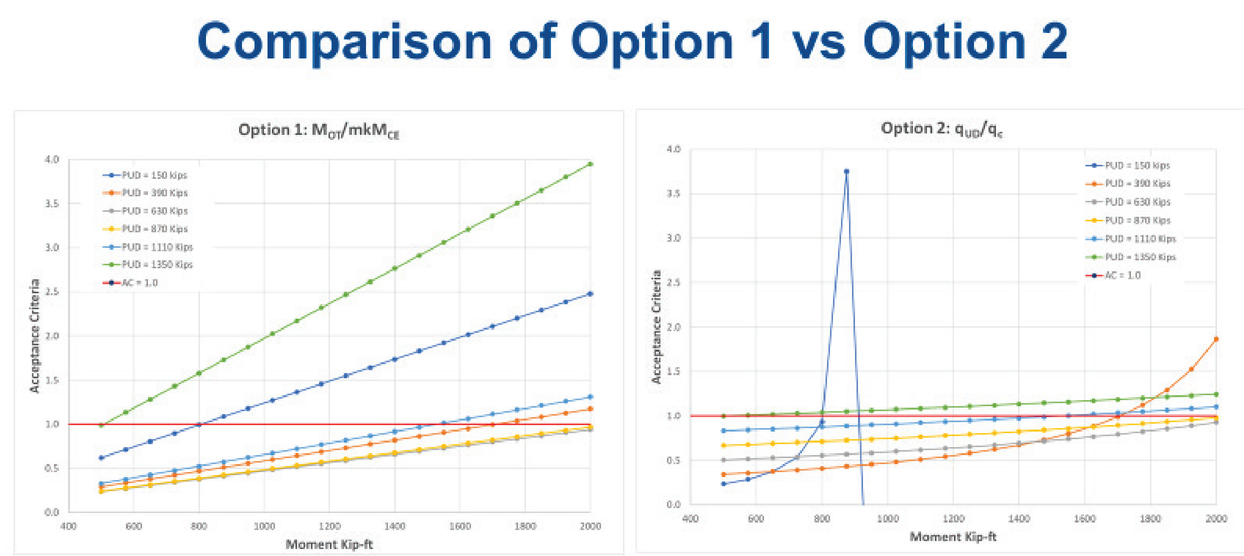


Figure C-79 Acceptance Ratios for the two options with varying axial load

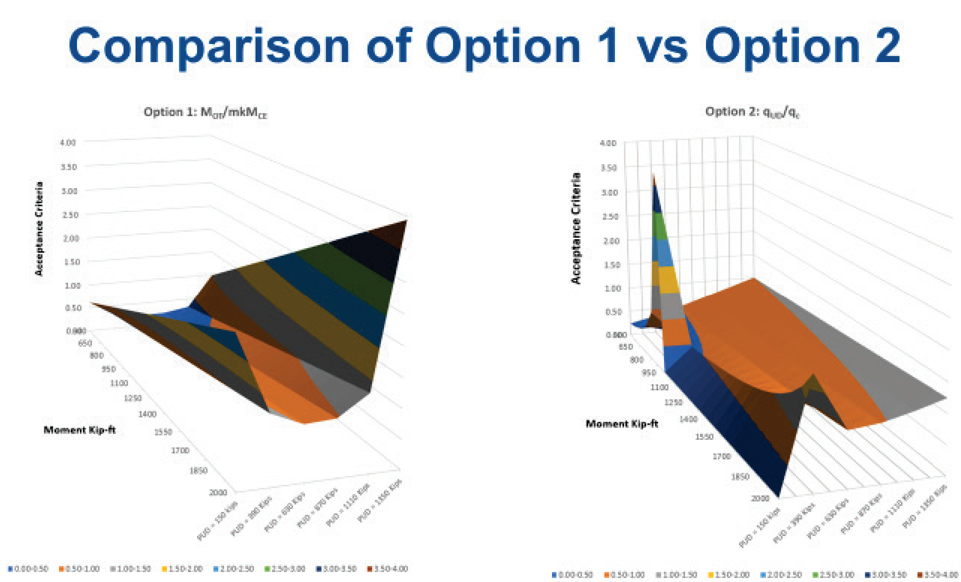
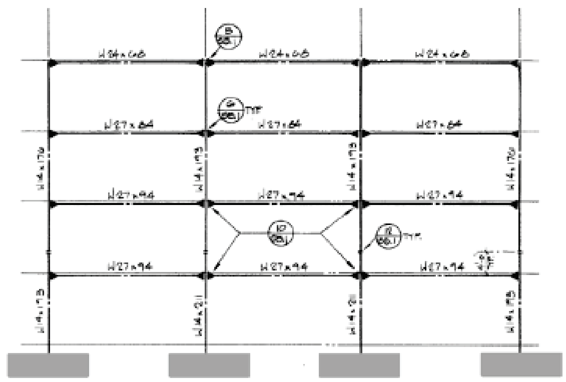


Figure C-80 Three-dimension representation showing the comparison between the two options with varying axial load.

Example 2: Three Bay Moment Frame

MF Example



End Column
DL = 140 k
LL = 70 k
EQ = ±690 k

Interior Column
DL = 150 k
LL = 70 k
EQ = ±78 k

End Column, Compression
1.1(DL + 0.25LL) = 173.25 Kips
EQ = P_seismic = 690 Kips

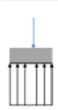
End Column, Tension
0.9(DL + 30k ftg wt) = 153 Kips
EQ = P_seismic = 690 Kips

Figure C-81 Three bay moment frame example

For this example, a new acceptance criterion was proposed, Option 2a, where AC = $(q_g + q_e/m)/q_c$. Here the acceptance criteria based on the soil pressure under the footing. Results of the comparison of the two options when footing is in compression are shown in Figure C-82 through Figure C-84.

MF Example – Option 1 Compression

Foundation Demands – Option 1?



Pud = 1.1*(140k+0.25*70k) + 690k = 850k
Pce = 2*3*(4ksf)*8'*8' = 1,540 k
mPce = 4*1,540 = 6,160 k
AC = 850/6,160 = 0.14

End Column
Compression
1.1(DL + 0.25LL) = 173.25 Kips
EQ = P_seismic = 690 Kips

Pud = 173.25 + 690 = 863.5

If q_{allow} = 12 ksf
P_{ce} = 3*(4ksf)*8'*8' = 768 kips
mPce = 4*768 = 3,072 kips
AC = 863.5/3,072 = 0.28

Figure C-82 Acceptance criteria using Option 1

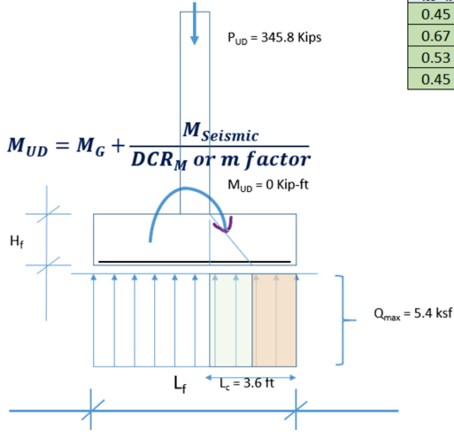
MF Example – Option 2 Compression

Footing 8' x 8' x 2'

Column	b in	14
14x14	h in	14
Footing	B_f ft	8
8'x8'x2'	L_f ft	8
	H_{ftg} ft	2
Axial Load	P_G =	173.25
	P_{seism}	690
DCR_A	DCR_A	4
Moment	M_{seis} kip ft	0
DCR_M	DCR_m	1
	Q_{allowG} ksf	4
	f'$_c$ psi	3000
	f_y ksi	60
	#of bars	12
	Area per bar	1
	As	12
	As_{min}	6.4
	d	20.0

$$P_{UD} = P_G + \frac{P_{Seismic}}{DCR_A \text{ or } m \text{ factor}}$$

$$M_{UD} = M_G + \frac{M_{Seismic}}{DCR_M \text{ or } m \text{ factor}}$$



Soil Pressure distribution

Summary - Foundation Acceptance Criteria

Soil Pressure Ratio q_{UD}/q_c	Footing M_u/M_n	Footing V_u/V_n	Ratio (M_u/M_n) / (V_u/V_n)	m Factor	Perfomance Level
0.45	0.23	0.36	0.63	Eq. Using DCR_A & DCR_M	
0.67	0.34	0.54	0.63	2	IO
0.53	0.26	0.42	0.63	3	LS
0.45	0.23	0.36	0.63	4	CP

End Column in Compression
1.1(DL + 0.25DL) = 173.25 Kips
EQ = P_seismic = 690 Kips
m = 4 or DCR_A = 4.0

AC Option 1 = 0.28 vs q_{UD}/q_c = 0.45

Footing Design Demands @ Critical Section			
	Maximum $Q_{max} = q_c$	Actual Demand	Ratio
Moment k-ft	560	252	0.45
Shear kips	168	76	0.45

Figure C-83 Acceptance criteria comparing Option 1 and Option 2 for compression loads

MF Example – Option 2a Compression

Foundation Demands – Option 2a

qg = 1.1*(140k+0.25*70k)/(8'*8') = 2.7 ksf
qe = 690k/(8'*8') = 10.7ksf

qg + qe/m = 2.7+10.7/4 = 5.4 ksf
qc = 3*4ksf = 12 ksf

AC = 5.4/12 = 0.44
Check footing for 2.7+10.7/(4*0.44) = 8.8 ksf

End Column in Compression
1.1(DL + 0.25LL) = 173.25 Kips
EQ = P_seismic = 690 Kips
m = 4 or DCR_A = 4.0

AC = 0.44 vs q_{UD}/q_c = 0.45

Figure C-84 Acceptance criteria comparing Option 2a and Option 2

For the same soil bearing capacity q_c, the AC for Option 1 is less conservative than either Option 2 or 2a. For this case Option 2 gives the same results as Option 2a. Therefore, the proposed formulation q_{UD}/q_c is equivalent to the AC = $(q_g + q_e/m)/q_c$.

Results of the comparison of the two options when the footing is in tension is given in Figure C-85 and Figure C-86. Here three options are considered. In Option 1, the axial load P_{UD} is the net uplift on the footing which is resisted by m-factor times the restoring gravity load, resulting in an AC of 0.4. If the seismic demand P_E = 690 kips is resisted by m-factor times the gravity restoring force of 153 kips, from the tension load combination Eq. 7-2 of ASCE/SEI 41-17, would result in an AC = 0.56. In Option 2a the soil pressure q_g from the compression load combination is compared with an equivalent upward pressure q_e/m. Here it shows that $q_g - q_e/m > 0$ or no uplift occurs. Therefore, the AC is satisfied. In reality the tension load combination should have been used to gravity pressure on the soil and compared with q_e/m. The outcome would however be the same and show the footing AC for tension is satisfied.

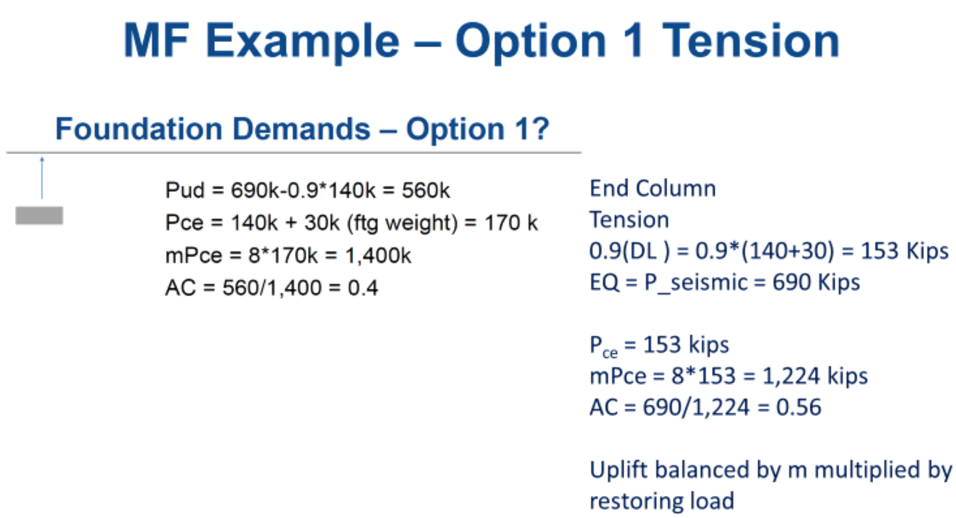


Figure C-85 Acceptance criteria using Option 1

MF Example – Option 2a Tension

Foundation Demands – Option 2a

qg = 1.1*(140k+0.25*70k)/(8'*8') = 2.7 ksf
qe = 690k/(8'*8') = 10.7ksf

qg + qe/m = 2.7-10.7/8 = 1.4 ksf down (no uplift)

End Column
Tension
0.9(DL) = 0.9*(140+30) = 153 Kips
EQ = P_seismic = 690 Kips

P_{ce} = 153 kips
mPce = 8*153 = 1,224 kips
AC = 690/1,224 = 0.56

Uplift balanced by m multiplied by restoring load, all things equal, end result is the same.

Figure C-86 Acceptance criteria using Option 2 and Option 2a

Summary

The two options, Option 2 (q_{UD}/q_c) and 2a AC = $(q_g + q_e/m)/q_c$ results in the same acceptance criteria for compression provided the same load combination and *m*-factors are used. Option 2 when footing is in uplift AC should be as stated in ASCE/SEI 41-17, where the seismic axial demand is equated with *m*-factor times the gravity restoring load.

CONCLUSION

For both examples considered, using either Option 1 or Option 2 results in the same acceptance criteria at the ultimate overturning capacity of the isolated footing.

C.4.8 Overarching Issue not Addressed by Either Option

The flexible base procedures have been developed for an isolated spread footing subjected to a dominant moment demand. Application to isolated spread footings subject to predominantly axial force, combined footings with multi-directional loads, and mat foundations is not clear.

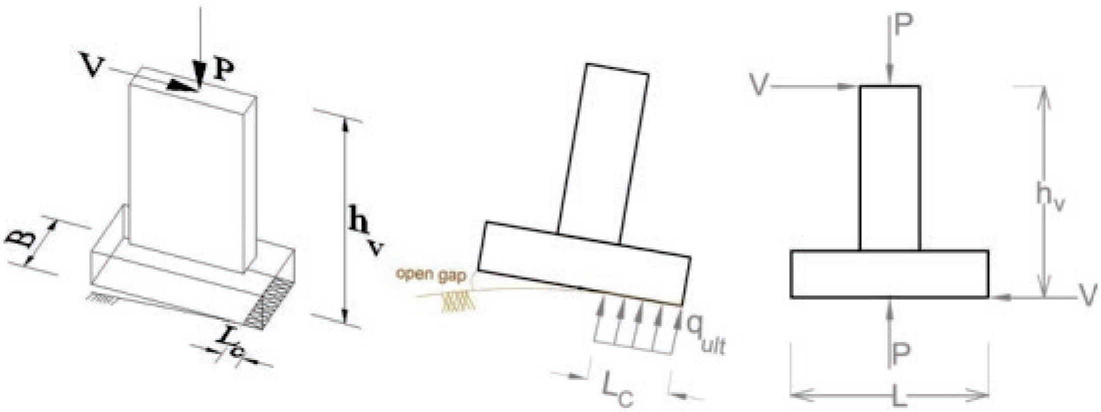


Figure C-87 Rocking on an isolated footing

m-factors in Chapter 8 were calibrated for rocking behavior (Figure C-87) from many tests using different rectangular and I-shaped footings to get allowable rotation demand, $q_{allowable}$, considering gradual accumulation of settlement with the number of cycles as a localized bearing failure converted to *m*-factors through m ~ ($q_{allowable}$ * K50) /$M_{capacity}$. The actual magnitude of the elastic stiffness of the springs is determined iteratively using a monotonic pushover analysis, so that the secant rotational stiffness of the foundation corresponding to 50% mobilization of the foundation moment capacity, M_{cf} , is equal to 300M_{cf} (Deng et al. 2014).

Axial action behavior is different, settlement accumulates with every cycle with very little recentering. Stiffness is very large for recompression and stiffness is much less for virgin compression as shown in Figure C-89.

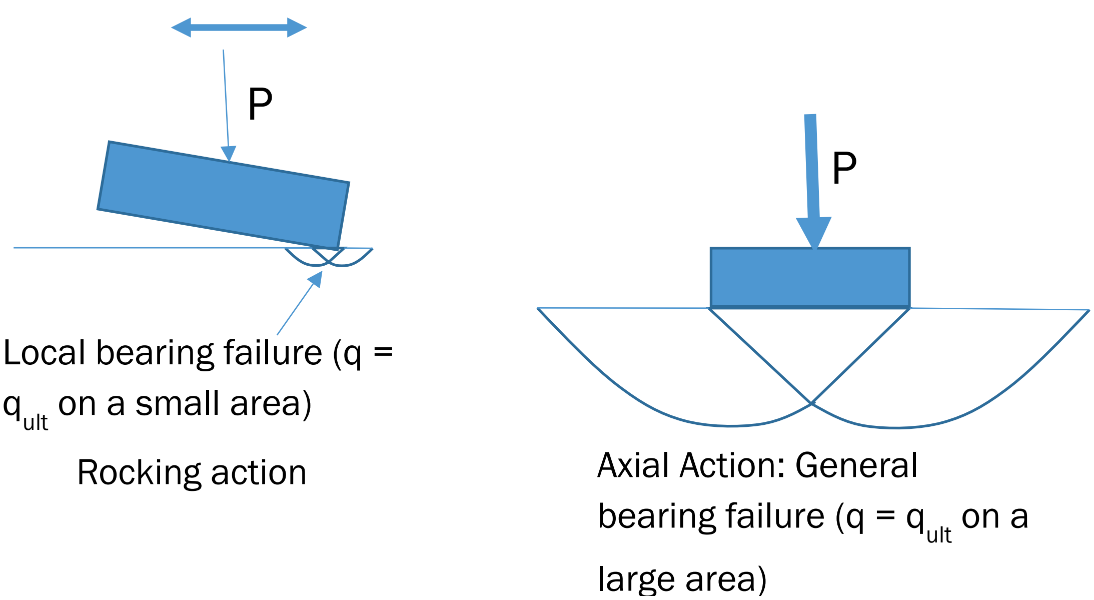


Figure C-88 Comparison of soil bearing failure from rocking and axial actions.

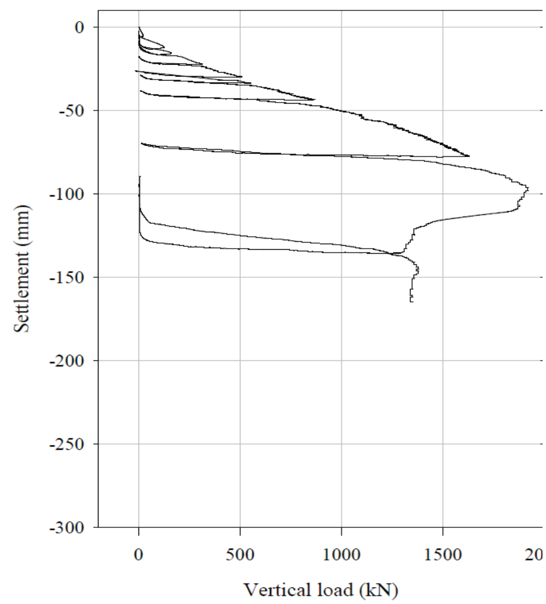


Figure C-89 Soil force deformation for cyclic axial compression action.

C.4.9 Decision on Option Selection

From the results of the comparison between the two options, the following was decided.

- Continue with the methodology in option 1, but make necessary adjustments for footing design check
- Revisit the *m*-factors when seismic overturning demand on the foundation is primarily resisted by axial resistance by the soil.

When demands from a fixed base linear analysis of the superstructure are transferred to another program to check the foundations, if the foundation analysis program is nonlinear, only the seismic demands are permitted to be divided by the *m*-factor prior to the foundation analysis check.

C.4.10 Options Recommendations from Review of Case Study Results

From numerous case studies and discussions on selected topics related to overturning actions on shallow foundations recommendation for a number of code change proposals were formulated and incorporated into the rewrite of Chapter 8 of ASCE/SEI 41. The flowchart of the structure of the rewrite is given below. The recommended changes that resulted from this case study for Archetype Building 2 and from the case study from Archetype Building 1 where further investigated and enhanced and are presented in Chapter 1.

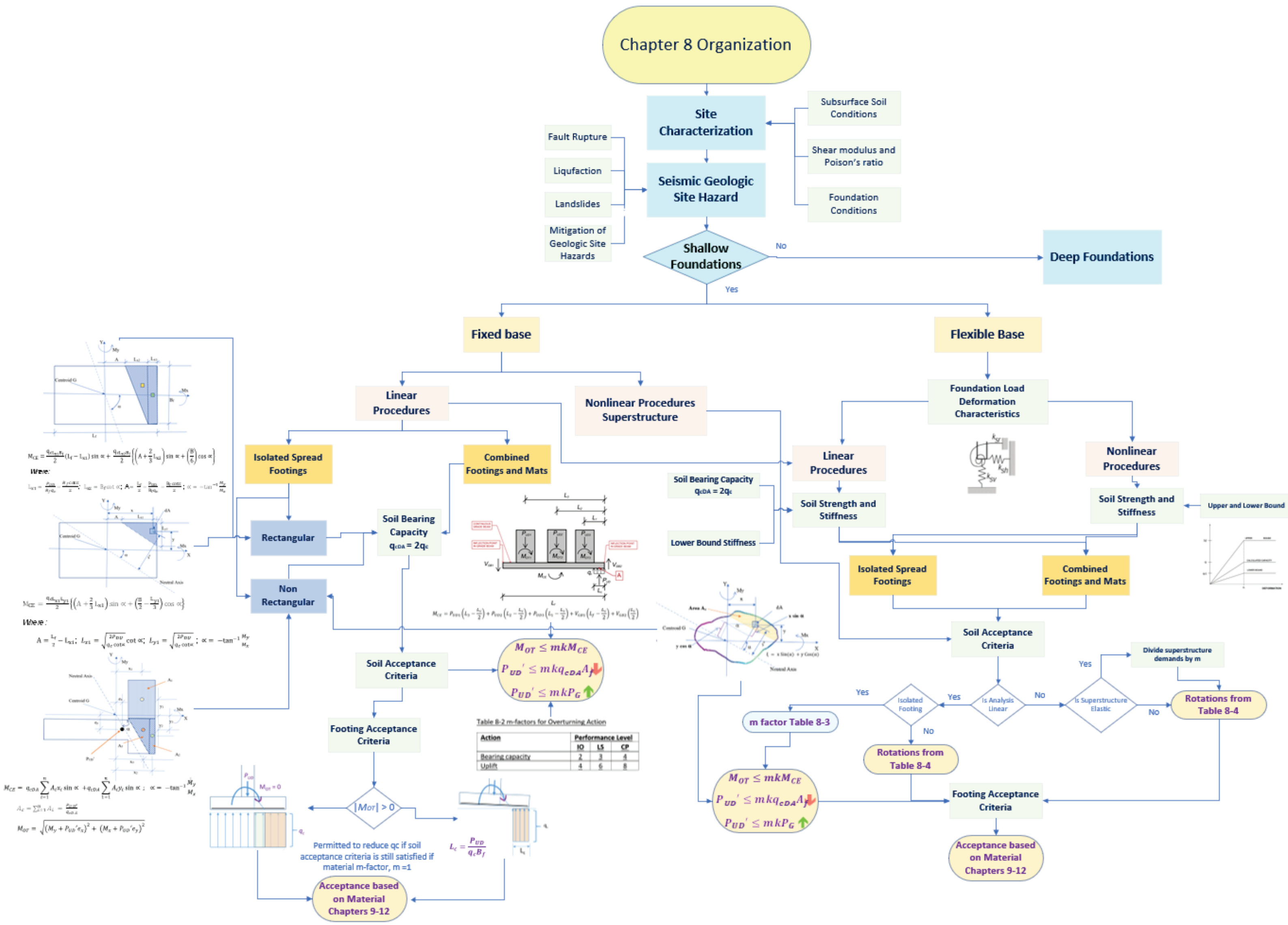


Figure C-90 Flowchart of proposed restructure of Chapter 8, for shallow foundations

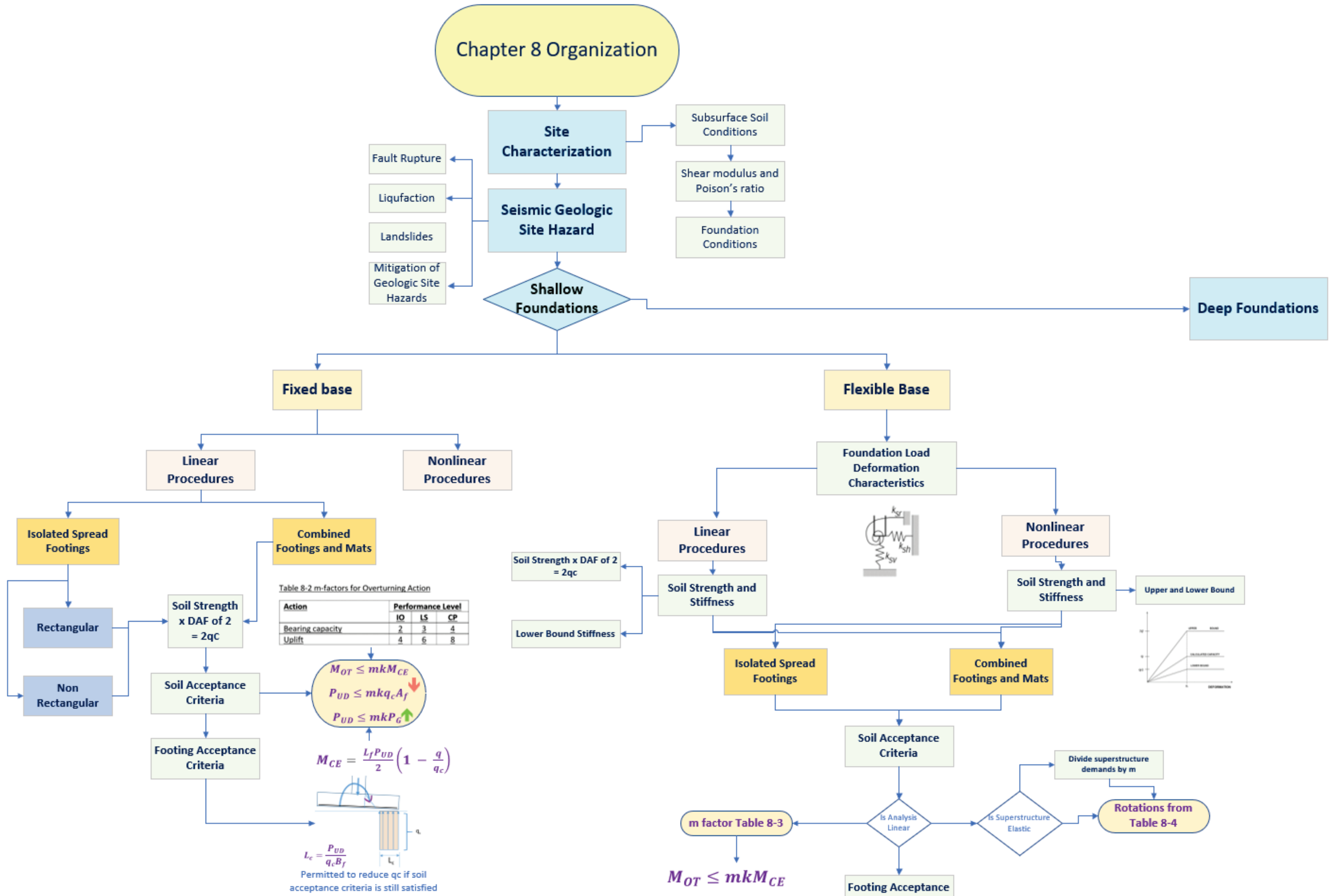


Figure C-90 Flowchart of proposed restructure of Chapter 8, for shallow foundations

C.5 References

Deng, L., Kutter, B., and Kunnath, S., 2014. Seismic design of rocking shallow foundations: displacement-based methodology, J. Bridge Eng., Applied Technology Council (ATC), 2011. Quantification of Building Seismic Performance Factors: Component Equivalency Methodology, FEMA P-795, prepared for the Federal Emergency Management Agency, Redwood City, CA. 10.1061/(ASCE)BE.1943-5592.0000616.

Appendix C1 -Base Shear Calculations:

C1.1 ASCE/SEI 7-10

$$\boldsymbol{V = C_s W}$$

Where

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$$

Location, Van Nuys, CA

BSE-1N

S_{DS}	1.386	g
S_{D1}	0.842	g
C_t	0.016	
h_n	65.7	ft
x	0.9	
C_u	1.40	S_{D1} > 0.4 s
R	8.0	Special Concrete Moment Resisting Frame
I_e	1.0	Risk Category II
$T_{Building}$	1.575	Fixed based period
T_0	0.122	sec
T_s	0.608	sec
T_a	0.69	sec
T	0.97	sec
C_s	**0.11**	

BSE-1N Vertical Distribution of Seismic Forces ASCE 7

Floor	Weight	H_x	$w_x h^k$	C_{vx}	Fx
7	1341	65.7	234786	0.25	273
6	1381	57	202908	0.22	236
5	1381	48.33	165523	0.18	192
4	1381	39.6	129441	0.14	150
3	1381	30.9	95302	0.10	111
2	1381	22.2	63367	0.07	74
1	1751	13.5	43486	0.05	51
	9997		934813		1087

k = 1.23

V = **1087**

C1.2 ASCE 41-17

Pseudo seismic force demands (Model A)

ASCE 41-17

Pseudo Seismic Force - LSP (Fixed Base)

$$V = C_1 C_2 C_m S_a W$$

C_1	1	T > 1 second	
C_2	1	T ≥ 0.7 s	
C_m	1	T > 1 second	
T_e	1.574	Seconds	(Variable EI for columns based on Axial Load) (0.3 EI for beams)

Location, Van Nuys, CA

BSE-1N

S_{XS}	1.386	
S_{X1}	0.842	
S_a	0.535	5% Damped spectrum

BSE-2N

S_{XS}	2.079	
S_{X1}	1.263	
S_a	0.802	5% Damped spectrum

BSE-1N Vertical Distribution of Seismic Forces

Floor	Weight	H_x	$w_x h^k$	C_{vx}	Fx
7	1341	65.7	833733	0.28	1496
6	1381	57	690197	0.23	1238
5	1381	48.33	535593	0.18	961
4	1381	39.6	394322	0.13	707
3	1381	30.9	269315	0.09	483
2	1381	22.2	162009	0.05	291
1	1751	13.5	95633	0.03	172
	9997		2980802		5348

k = 1.537

V = **5348**

BSE-2N Vertical Distribution of Seismic Forces

Floor	Weight	H_x	$w_x h^k$	C_{vx}	Fx
7	1341	65.7	833733	0.28	2244
6	1381	57	690197	0.23	1857
5	1381	48.33	535593	0.18	1441
4	1381	39.6	394322	0.13	1061
3	1381	30.9	269315	0.09	725
2	1381	22.2	162009	0.05	436
1	1751	13.5	95633	0.03	257
	9997		2980802		8022

k = 1.537

V = **8022**

Pseudo seismic force demands (Model B)

ASCE 41-17

Pseudo Seismic Force - LSP (With Area Springs no Grade beams)

$$V = C_1 C_2 C_m S_a W$$

C_1	1	T > 1 second	
C_2	1	T ≥ 0.7 s	
C_m	1	T > 1 second	
T_e	1.626	Seconds	(Variable EI for columns based on Axial Load) (0.3 EI for beams)

Location, Van Nuys, CA

BSE-1N

S_{XS}	1.386	
S_{X1}	0.842	
S_a	0.518	5% Damped spectrum

BSE-2N

S_{XS}	2.079	
S_{X1}	1.263	
S_a	0.777	5% Damped spectrum

BSE-1N Vertical Distribution of Seismic Forces

Floor	Weight	H_x	$w_x h^k$	C_{vx}	Fx
7	1341	65.7	929573	0.28	1460
6	1381	57	766701	0.23	1205
5	1381	48.33	592413	0.18	931
4	1381	39.6	433902	0.13	682
3	1381	30.9	294441	0.09	463
2	1381	22.2	175608	0.05	276
1	1751	13.5	102329	0.03	161
	9997		3294966		5177

k = 1.563

V = **5177**

BSE-2N Vertical Distribution of Seismic Forces

Floor	Weight	H_x	$w_x h^k$	C_{vx}	Fx
7	1341	65.7	929573	0.28	2191
6	1381	57	766701	0.23	1807
5	1381	48.33	592413	0.18	1396
4	1381	39.6	433902	0.13	1023
3	1381	30.9	294441	0.09	694
2	1381	22.2	175608	0.05	414
1	1751	13.5	102329	0.03	241
	9997		3294966		7765

k = 1.563

V = **7765**

Pseudo seismic force demands (Model C)

ASCE 41-17

Pseudo Seismic Force - LSP (With Area Springs no Grade beams, LB Springs)

$$V = C_1 C_2 C_m S_a W$$

C_1	1	T > 1 second	
C_2	1	T ≥ 0.7 s	
C_m	1	T > 1 second	
T_e	1.665	Seconds	(Variable EI for columns based on Axial Load) (0.3 EI for beams)

Location, Van Nuys, CA

BSE-1N

S_{XS}	1.386	
S_{X1}	0.842	
S_a	0.506	5% Damped spectrum

BSE-2N

S_{XS}	2.079	
S_{X1}	1.263	
S_a	0.759	5% Damped spectrum

BSE-1N Vertical Distribution of Seismic Forces

Floor	Weight	H_x	$w_x h^k$	C_{vx}	Fx
7	1341	65.7	1008616	0.28	1435
6	1381	57	829594	0.23	1181
5	1381	48.33	638950	0.18	909
4	1381	39.6	466172	0.13	663
3	1381	30.9	314813	0.09	448
2	1381	22.2	186551	0.05	265
1	1751	13.5	107656	0.03	153
	9997		3552353		5056

k = 1.5825

V = **5056**

BSE-2N Vertical Distribution of Seismic Forces

Floor	Weight	H_x	$w_x h^k$	C_{vx}	Fx
7	1341	65.7	1008616	0.28	2153
6	1381	57	829594	0.23	1771
5	1381	48.33	638950	0.18	1364
4	1381	39.6	466172	0.13	995
3	1381	30.9	314813	0.09	672
2	1381	22.2	186551	0.05	398
1	1751	13.5	107656	0.03	230
	9997		3552353		7583

k = 1.5825

V = **7583**

Pseudo seismic force demands (Model D)

ASCE 41-17

Pseudo Seismic Force - LSP (With Area Springs no Grade beams, UB Springs)

$$V = C_1 C_2 C_m S_a W$$

C_1	1	T > 1 second	
C_2	1	T ≥ 0.7 s	
C_m	1	T > 1 second	
T_e	1.604	Seconds	(Variable EI for columns based on Axial Load) (0.3 EI for beams)

Location, Van Nuys, CA

BSE-1N

S_{XS}	1.386	
S_{X1}	0.842	
S_a	0.525	5% Damped spectrum

BSE-2N

S_{XS}	2.079	
S_{X1}	1.263	
S_a	0.787	5% Damped spectrum

BSE-1N Vertical Distribution of Seismic Forces

Floor	Weight	H_x	$w_x h^k$	C_{vx}	Fx
7	1341	65.7	887749	0.28	1475
6	1381	57	733350	0.23	1219
5	1381	48.33	567673	0.18	943
4	1381	39.6	416693	0.13	692
3	1381	30.9	283537	0.09	471
2	1381	22.2	169720	0.05	282
1	1751	13.5	99441	0.03	165
	9997		3158163		5248

k = 1.552
V = **5248**

BSE-2N Vertical Distribution of Seismic Forces

Floor	Weight	H_x	$w_x h^k$	C_{vx}	Fx
7	1341	65.7	887749	0.28	2213
6	1381	57	733350	0.23	1828
5	1381	48.33	567673	0.18	1415
4	1381	39.6	416693	0.13	1039
3	1381	30.9	283537	0.09	707
2	1381	22.2	169720	0.05	423
1	1751	13.5	99441	0.03	248
	9997		3158163		7872

k = 1.552
V = **7872**

C2 Calculation of Target Displacement for NSP

The target displacement δ_t is calculated in accordance with ASCE 41-17 equation 7-28 as:

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g$$

Where:

T_e is the effective fundamental period of the building in the direction under consideration; C_0, C_1 and C_2 are defined in Section 7.4.3.3.2 of ASCE/SEI 41-17. S_a is the response spectral acceleration at the effective fundamental period in the direction under consideration.

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response. For periods less than 0.2 second, C_1 need not be taken greater than the value at T = 0.2 second. For periods greater than 1.0 second, C_1 = 1.0.

C_2 = Modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation and strength deterioration on maximum displacement response. For periods greater than 0.7 second, C_2 =1.0.

C2.1 Determination of Effective Period

The effective fundamental period, T_e, in the direction under consideration, is determined from the force-displacement relation of the nonlinear static pushover analysis, used to determine the initial lateral stiffness K_i and the idealized curve used to estimate the effective lateral stiffness, K_e, of the building. The effective fundamental period, T_e, is then be calculated as:

$$T_e = T_i \sqrt{\frac{K_i}{K_e}}$$

where:

T_i = Elastic fundamental period in the direction under consideration calculated by elastic dynamic analysis.

K_i = Elastic lateral stiffness of the building in the direction under consideration.

K_e = Effective lateral stiffness of the building in the direction under consideration.

T_e should always be greater than or equal to T_i.

C2.1 Determination of Effective Period

The target displacement calculation for an assumed period of 1.8 seconds is shown in Figure C3-1.

Target Displacement - Calculation

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g$$

C_0	1.44	Table 7-5
C_1	1	T_e > 1 s
C_2	1	$T_e \geq 0.7$ s
S_a	0.7	5% Damped spectrum, BSE-2N
T_e	1.80	Assumed

Target Displacment

Parameter	Modal Load Pattern
	inches
Roof Disp. δ_t=	31.97

Figure C3-1 Target displacement calculation at the BSE-2N earthquake hazard level

C3 Base Shear, Hinge Summary Table for NSP for the Target Limit States

The plastic hinge progression to the target displacements for limit state LS and CP are shown Table C3-1 and Table C3-2. Clearly the superstructure acceptance criteria (LS for BSE-1N and CP for BSE-2N) was not satisfied at both hazard levels.

Table C3-1 Hinge Summary at BSE-1N

TABLE: Base Shear vs Monitored Displacement													
Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	in	kip										
9	11.4	1741.2	281	181	0	0	0	367	95	0	0	462
10	12.5	1771.1	257	205	0	0	0	357	93	12	0	462
11	12.7	1776.6	257	205	0	0	0	348	101	13	0	462
12	12.7	1776.7	257	205	0	0	0	348	101	13	0	462
13	14.3	1814.0	250	212	0	0	0	333	115	14	0	462
14	16.0	1850.7	245	217	0	0	0	302	144	16	0	462
15	17.5	1884.1	240	222	0	0	0	286	145	31	0	462
16	18.9	1914.5	227	235	0	0	0	268	125	69	0	462
17	20.1	1937.2	200	260	2	0	0	267	103	91	1	462
18	20.1	1937.4	200	260	2	0	0	267	103	91	1	462
19	20.4	1941.7	200	260	2	0	0	266	102	92	2	462
20	20.4	1942.0	200	260	2	0	0	266	102	92	2	462
21	20.5	1943.0	200	260	2	0	0	266	102	92	2	462
22	20.5	1943.1	200	260	2	0	0	266	102	92	2	462
23	20.5	1943.8	200	260	2	0	0	266	102	92	2	462
24	20.5	1943.9	200	260	2	0	0	266	102	92	2	462
25	20.5	1944.4	200	260	2	0	0	264	104	92	2	462
26	20.5	1944.5	200	260	2	0	0	264	104	92	2	462
27	20.7	1946.5	199	261	2	0	0	262	103	95	2	462
28	20.7	1946.6	199	261	2	0	0	262	101	97	2	462
29	21.4	1957.6	194	266	2	0	0	255	96	109	2	462

Target Displacement: LS = 21.4”

Table C3-2 Hinge Summary at BSE-2N

TABLE: Base Shear vs Monitored Displacement													
Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	in	kip										
45	22.9	1979.3	179	281	2	0	0	244	102	114	2	462
46	22.9	1979.4	179	281	2	0	0	244	102	114	2	462
47	22.9	1979.9	178	282	2	0	0	242	104	114	2	462
48	22.9	1979.9	178	282	2	0	0	241	105	114	2	462
49	22.9	1980.4	178	282	2	0	0	238	108	114	2	462
50	22.9	1980.5	178	282	2	0	0	238	108	114	2	462
51	23.0	1981.2	177	283	2	0	0	235	107	118	2	462
52	23.0	1981.2	177	283	2	0	0	235	107	118	2	462
53	23.1	1982.1	177	283	2	0	0	233	107	120	2	462
54	23.1	1982.2	177	283	2	0	0	233	107	120	2	462
55	23.1	1982.6	177	283	2	0	0	232	108	120	2	462
56	23.1	1982.9	177	281	4	0	0	232	108	120	2	462
57	23.1	1982.9	177	280	5	0	0	232	108	120	2	462
58	23.1	1983.0	177	280	5	0	0	232	108	120	2	462
59	25.3	2013.0	165	281	16	0	0	217	99	130	16	462
60	27.4	2041.7	159	282	21	0	0	202	98	145	17	462
61	29.5	2059.2	157	272	33	0	0	195	73	161	33	462
62	30.6	2064.5	157	269	36	0	0	195	71	160	36	462
63	31.7	2063.7	157	237	68	0	0	195	69	140	58	462
64	32.0	2060.5	157	236	69	0	0	195	69	129	69	462

Target Displacement: CP = 32”

Part 3, Appendix D: Design Examples

D.1 Motivation

Chapter 8 of ASCE/SEI 41-17 has been completely rewritten and restructured for usability and technical content in ASCE/SEI 41-23, the next version of the standard. Given the extensive changes made to this chapter, three separate design examples are developed to help users of ASCE/SEI 41 on how the new provisions are to be applied and to understand the impact of the changes. The first example checks the foundation acceptance for a single cantilevered shear wall on a strip footing. The second example checks the foundation acceptance for a single bay braced frame on isolated and combined footings. The third example is of a stair tower on a Mat foundation. Each example demonstrates use of a different provision in the Chapter, and how the new provisions compare with the provisions in ASCE/SEI 41-17. These design examples only look at the acceptance for overturning stability and soil bearing, not the acceptance of the foundation structural component. These design examples reflect the final version of the strikeout/underline provisions from ATC-140 WG-2 effort and section numbers and acceptance may differ from the final release version of ASCE/SEI 41-23.

D.2 Design Example – 1: Single Cantilevered Shear Wall

D.2.1 Problem Statement:

For the cantilevered shear wall shown in Figure D.2-1, determine the soil bearing acceptance ratio at the foundation soil interface and the foundation acceptance at the Collapse Prevention performance level for the applied pseudo force demands for the following analysis and modeling options:

1. Soil foundation interface is modeled as a fixed base.
2. Soil foundation interface is modeled as a flexible base.

CODE: ASCE 41-23 AND ASCE 41-17

SPECIFICATIONS:

Allowable Soil pressure (D + L): q_{allow} = 3 ksf

Existing Concrete Strength: f'_c = 3,000 psi.

Existing Steel Strength f_y = 40,000 psi.

GIVEN:

Footing dimensions:

B_f = 5.0 ft

L_f = 40 ft

$A_f = B_f L_f$ = (5)(40) = 200 ft^2

LOADING:

M_{OT} = 30,000 kip-ft

P_D = 300.0 kips (Dead, includes weight of the footing)

P_L = 50.0 kips (0.25 unreduced live load)

P_E = 0.0 kips

MAXIMUM AXIAL DEMAND CAPACITY RATIO:

DCR_{max} = 1.0 (No reduction in seismic axial load demands to the wall due to superstructure yielding)

KNOWLEDGE FACTOR:

κ = 1.0

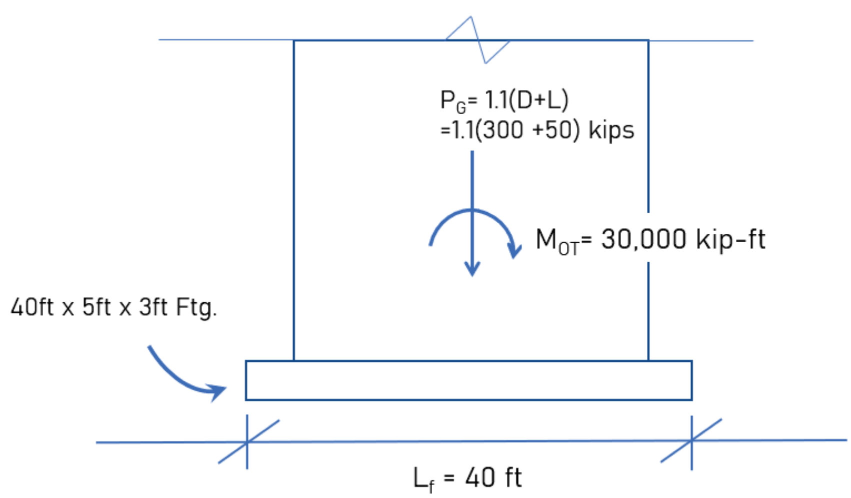


Figure D.2-1 Cantilever shear wall on a 40´x5´x3´ spread footing

D.2.2 CASE 1: Soil Foundation Interface Modeled as a Fixed Base

D.2.2.1 SOLUTION (PROPOSED PROVISIONS (ASCE/SEI 41-23)

D.2.2.1.1 Soil Strength for short term seismic loads:

q_{cDA} = 2(q_c) = 2(3)(q_{allow}) = 6(3) = 18ksf (ASCE/SEI 41-23 *Eq.* 8-9)

D.2.2.1.2 Determination of Foundation Moment Capacity

$$M_{CE} = \frac{P_{UF}L_f}{2}\left(1 - \frac{q}{q_{cDA}}\right) kip - ft \qquad (\text{ASCE/SEI } 41 - 23 \;\; Eq. 8 - 12)$$

Where:

Axial load demand:

$$P_{UF} = P_G + \frac{P_E}{DCR_{max}} = 1.1(300 + 50) + \frac{0}{1.0} = 385\, kips \qquad (\text{ASCE/SEI } 41 - 23\; Eq. 8 - 13)$$

Soil bearing pressure:

$$q = \frac{P_{UF}}{B_f L_f} = \frac{385}{200} = 1.925\, ksf$$

Foundation Moment Capacity

$$M_{CE} = \frac{P_{UF}L_f}{2}\left(1 - \frac{q}{q_{cDA}}\right) = \frac{385 * 40}{2}\left(1 - \frac{1.925}{18}\right) = 6876.5\, kip - ft$$

D.2.2.1.3 Acceptance Criteria Soil Bearing and Overturning, ASCE/SEI 41-23, Sec. 8.4.4.1.1.3.1

The component ductility or m-factors for soil bearing and overturning are given in ASCE/SEI 41-23 Table 8-4.

m_{CP} = 4.0

Acceptance Criteria, soil bearing, from ASCE/SEI 41 – 23 Eq. 8-21 is given as:

$$M_{OT} \le m\kappa M_{CE} \qquad (\text{ASCE/SEI } 41 - 23 \;\; Eq. 8 - 21)$$

Rewriting Eq. 8-21 in terms of an Acceptance Ratio (AR)

$$AR = \frac{M_{OT}}{m\kappa M_{CE}} = \frac{30{,}000}{4 * 1 * 6876.5} = 1.091$$

D.2.2.2 SOLUTION (ASCE 41-17)

D.2.2.2.1 Soil Strength

Use of upper bound strenght is permitted per (ASCE/SEI 41-17 §8.4.2.3.1):

q_c = (1+ C_v)(q_c) = (1+1)(3)(q_{allow}) = 6(3) = 18ksf

D.2.2.2.2 Foundation Moment Capacity

$$M_{CE} = \frac{P_{UD}L_f}{2}\left(1 - \frac{q}{q_c}\right) kip - ft \qquad (\text{ASCE/SEI } 41 - 17\ Eq. 8 - 10)$$

D.2.2.2.3 Foundation Moment Capacity and Acceptance Criteria – When actions from seismic and gravity are additive

m-factor (overturning compression, Sec 8.4.2.3.2.1)

m_{CP} = 4.0

Axial load demand, using the compression load combiation (ASCE/SEI 41-23 Eq. 7-1):

$$P_{UD} = P_G + \frac{P_E}{DCR} = 1.1 * 350 + \frac{0}{1.0} = 385\ kips$$

Soil bearing pressure:

$$q = \frac{P_{UD}}{A_f} = \frac{385}{200} = 1.925\ ksf$$

$$M_{CE} = \frac{P_{UD}L_f}{2}\left(1 - \frac{q}{q_c}\right) = \frac{385 * 40}{2}\left(1 - \frac{1.925}{18}\right) = 6876.5\ kip - ft$$

Acceptance Criteria, soil bearing:

$$m\kappa Q_{CE} > Q_{UD} \qquad ASCE/SEI\ 41 - 17\ (Eq.\ 7 - 36)$$

$$Q_{CE} = M_{CE} \qquad ASCE/SEI\ 41 - 17\ (Sec. 8.4.2.3.2.1)$$

$$Q_{UD} = M_{UD} \qquad ASCE/SEI\ 41 - 17\ (Eq. 7 - 34)$$

Acceptance Ratio (AR)

$$AR = \frac{M_{UD}}{m\kappa M_{CE}} = \frac{30{,}000}{4 * 1 * 6876.5} = 1.091$$

D.2.2.2.4 Foundation Moment Capacity and Acceptance Criteria – When actions from seismic forces and gravity loads are counteracting

Axial load demand, when load combiation dead load is counteractive ASCE/SEI 41-17 (Eq. 7-2):

$$P_{UD} = P_G + \frac{P_E}{DCR} = 0.9 * 300 + \frac{0}{1.0} = 270\ kips$$

Soil bearing pressure:

$$q = \frac{P_{UD}}{A_f} = \frac{270}{200} = 1.35\ ksf$$

Moment Capacity:

$$M_{CE} = \frac{P_{UD} L_f}{2}\left(1 - \frac{q}{q_c}\right) = \frac{270 * 40}{2}\left(1 - \frac{1.35}{18}\right) = 4995\ kip - ft$$

Acceptance Criteria, soil bearing

$$m\kappa Q_{CE} > Q_{UD} \qquad ASCE\ 41 - 17\ (Eq.\ 7 - 36)$$

$$Q_{CE} = M_{CE} \qquad ASCE\ 41 - 17\ (Sec.\ 8.4.2.3.2.1)$$

$$Q_{UD} = M_{UD} \qquad \text{v}\ \ ASCE\ 41 - 17\ (Eq.\ 7 - 34)$$

Acceptance Ratio (AR)

The code (ASCE/SEI 41-17 §8.4.2.3.2.1) is not specific which m-factor is to be used in this case, therefore the AR is shown for two possible options. One using m-factor for compression, and the other, the m-factor for uplift.

> 8.4.2.3.2.1 Foundation Modeled as a Fixed Base. If the base of the structure is assumed to be completely rigid, the foundation overturning action shall be classified as deformation controlled. The overturning demand Q_{UD} shall be determined using Eq. (7-34) and the soil shall be evaluated using Eq. (7-36) with $Q_{CE} = M_{CE}$. The *m*-factors for overturning compression shall be 2.0 for Immediate Occupancy, 3.0 for Life Safety, and 4.0 for Collapse Prevention, and the use of upper-bound component capacities shall be permitted. Where overturning results in an axial uplift force demand on the foundation, this uplift action shall be evaluated using an *m*-factor of 4.0 for Immediate Occupancy, 6.0 for Life Safety, and 8.0 for Collapse Prevention applied to the expected restoring dead load.

Option 1: m_{CP} = 4.0, overturning compression

$$AR = \frac{M_{UD}}{m\kappa M_{CE}} = \frac{30{,}000}{4 * 1 * 4995} = 1.502$$

Option 2: m_{CP} = 8.0 when overturning results in axial uplift force demand

$$AR = \frac{M_{UD}}{m\kappa M_{CE}} = \frac{30{,}000}{8 * 1 * 4995} = 0.751$$

D.2.2.3 RESULTS COMPARISON:

Acceptance Ratio: Soil Foundation Interface Modeled as a Fixed Base

Load Combination	ASCE/SEI 41-23 (Proposed)	ASCE/SEI 41-17
Compression (Eq. 7-1)	1.091	1.091
Uplift (Eq. 7-2)	Not calculated	Option 1 – 1.502
	-	Option 2 – 0.751

D.2.3 CASE 2: Soil Foundation Interface Modeled as a Flexible Base

D.2.3.1 SOLUTION (PROPOSED PROVISIONS, ASCE/SEI 41-23)

D.2.3.1.1 Soil Strength for short term seismic loads:

$q_{cDA} = 2(q_c) = 2(3)(q_{allow}) = 6(3) = 18$ksf ASCE/SEI 41 – 23 (*Eq.* 8-9)

D.2.3.1.2 Determination of Foundation Moment Capacity

$$M_{CE} = \frac{P_{UF}L_f}{2}\left(1 - \frac{q}{q_{cDA}}\right) kip - ft \qquad (\text{ASCE/SEI } 41 - 23\ \ Eq. 8 - 12)$$

Where:

Axial load demand:

Load combiation compression ASCE/SEI 41-23 (Eq. 7-1):

$$P_{UF} = P_G + \frac{P_E}{DCR_{max}} = 1.1(300 + 50) + \frac{0}{1.0} = 385\ kips \qquad (\text{ASCE/SEI } 41 - 23\ Eq. 8 - 13)$$

Soil bearing pressure:

$$q = \frac{P_{UF}}{B_f L_f} = \frac{385}{200} = 1.925\ ksf$$

Foundation Moment Capacity

$$M_{CE} = \frac{P_{UF} L_f}{2}\left(1 - \frac{q}{q_{cDA}}\right) = \frac{385 * 40}{2}\left(1 - \frac{1.925}{18}\right) = 6876.5\ kip - ft$$

D.2.3.1.3 Foundation Acceptance

Determination of m-factor Table 8-7

m-factor (overturning compression) are obtained from ASCE/SEI Table 8-7, Figure D.2-1.

			m-Factors[a]		
			Performance Level		
Footing Shape			IO	LS	CP
i. Rectangle					
$\frac{b}{L_c}$	$\frac{A_{rect} - A_f}{A_{rect}}$	$\frac{A_c}{A_f}$			
≥ 10	0				
		0.20	5	8	10
		0.5	3	5	6
		1	1	1	1
3	0				
		0.20	4	6	8
		0.5	2	3	4
		1	1	1	1
1	0				
		0.20	2.5	5	6
		0.5	1.5	2	3
		1	1	1	1
0.3	0				
		0.20	2	4	5
		0.5	1	1.5	2
		1	1	1	1

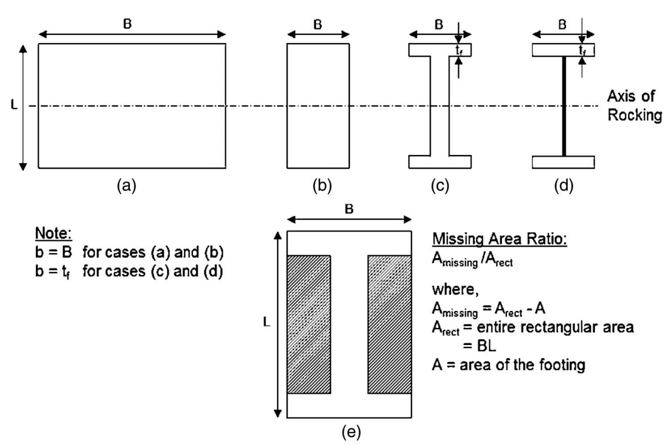


Figure D.2-1 m-Factors from ASCE/SEI 41-23 Table 8-7

$b = B_f = 5$ ft

$$L_c = \frac{P_{UF}}{B_f q_{cDA}} = 4.278\ ft$$

$$A_c = \frac{P_{UF}}{q_{cDA}} = 21.389;\ ft^2$$

$$A_{rect} = A_f = 200;\ ft^2$$

$$A_{miss} = \frac{A_{rect} - A_f}{A_{rect}} = 0;\ ft^2$$

$$b_{ratio} = \frac{b}{L_c} = 1.169;$$

$$A_{c\_ratio} = \frac{A_c}{A_f} = \frac{21.389}{200} = 0.107;$$

Interpolating the m-factor from Table 8-7

$$m_{CP} = 6 + (8-6)\frac{(b_{ratio} - 1.0)}{(3.0 - 1.0)} = 6.169;$$

$m_{CP} = 6.169$

Acceptance Criteria, soil bearing Eq. 8-21

$$M_{OT} \leq m\kappa M_{CE} \qquad \text{ASCE/SEI 41} - 23;\ Eq. 8 - 21$$

Acceptance Ratio (AR)

$$AR = \frac{M_{OT}}{m\kappa M_{CE}} = \frac{30{,}000}{6.169 * 1 * 6876.5} = 0.707$$

D.2.3.2 SOLUTION (ASCE/SEI 41-17)

D.2.3.2.1 Soil Strength

upper bound (ASCE/SEI 41-17 §8.4.2.3.2.2)

$q_c = (1 + C_v)(q_c) = (1+1)(3)(q_{allow}) = 6(3) = 18$ksf

D.2.3.2.2 Foundation Moment Capacity

$$M_{CE} = \frac{P_{UD}L_f}{2}\left(1 - \frac{q}{q_c}\right) kip - ft \qquad \text{ASCE/SEI } (Eq. 8 - 10)$$

D.2.3.2.3 Foundation Moment Capacity and Acceptance Criteria – When actions from seismic and gravity are additive

Axial load demand

Using the compression load combiation ASCE/SEI (Eq. 7-1)

$$P_{UD} = P_G + \frac{P_E}{DCR} = 1.1(300 + 50) + \frac{0}{1.0} = 385\ kips$$

Soil Bearing Pressure

$$q = \frac{P_{UD}}{A_f} = \frac{385}{200} = 1.925\ ksf$$

m-factor (overturning compression, §Sec 8.4.2.3.2.2)

m_{CP} = 6.169 from above since $P_{UF} = P_{UD}$.

$$M_{CE} = \frac{P_{UD}L_f}{2}\left(1 - \frac{q}{q_c}\right) = \frac{385 * 40}{2}\left(1 - \frac{1.925}{18}\right) = 6876.5\ kip - ft$$

Acceptance Criteria, soil bearing:

$$m\kappa Q_{CE} > Q_{UD} \qquad ASCE/SEI\ 41 - 17\ (Eq.\ 7 - 36)$$

Acceptance Ratio (AR)

$$AR = \frac{M_{UD}}{m\kappa M_{CE}} = \frac{30{,}000}{6.169 * 1 * 6876.5} = 0.707$$

D.2.3.2.4 Foundation Moment Capacity and Acceptance Criteria – When actions from seismic forces and gravity loads are counteracting

Axial load demand

When load combiation dead load is counteractive ASCE/sEI (Eq. 7-2) applies

$$P_{UD} = P_G + \frac{P_E}{DCR} = 0.9 * 300 + \frac{0}{1.0} = 270\ kips$$

Soil bearing pressure:

$$q = \frac{P_{UD}}{A_f} = \frac{270}{200} = 1.35\ ksf$$

Moment Capacity:

$$M_{CE} = \frac{P_{UD}L_f}{2}\left(1 - \frac{q}{q_c}\right) = \frac{270 * 40}{2}\left(1 - \frac{1.35}{18}\right) = 4995\ kip - ft$$

Acceptance Criteria, soil bearing

Acceptance Ratio (AR)

The code is not specific which m-factor is to be used in this case, therefore the following are options are considered:

Option 1: m_{CP} = 6.169, overturning compression

$$AR = \frac{M_{OT}}{m\kappa M_{CE}} = \frac{30{,}000}{6.169 * 1 * 4995} = 0.97$$

Option 2: m_{CP} = 10.0 when overturning results in axial uplift force demand

$$AR = \frac{M_{OT}}{m\kappa M_{CE}} = \frac{30{,}000}{10 * 1 * 4995} = 0.60$$

D.2.3.3 RESULTS COMPARISON:

Acceptance Ratio: Soil Foundation Interface Modeled as a Flexible Base

Load Combination	ASCE 41-23 (Proposed)	ASCE 41-17
Compression (Eq. 7-1)	0.707	0.707
Uplift (Eq. 7-2)	Not calculated	Option 1 – 0.97
	-	Option 2 – 0.60

D.2.4 Summary:

Results using ASCE/SEI 41-17 and more conservative than the results using ASCE/SEI 41-23 if the compression m-values are use when seismic loads and gravity are counteraction. ASCE/SEI 41-17 and 41-23 give identical results if only the load combination where seismic and gravity are additive is used.

Case 1: Acceptance Ratio: Soil Foundation Interface Modeled as a Fixed Base

Load Combination	ASCE 41-23 (Proposed)	ASCE 41-17
Compression (Eq. 7-1)	1.091	1.091
Uplift (Eq. 7-2)	Not calculated	Option 1 – 1.502
	-	Option 2 – 0.751

Case 2: Acceptance Ratio: Soil Foundation Interface Modeled as a Flexible Base

Load Combination	ASCE 41-23 (Proposed)	ASCE 41-17
Compression (Eq. 7-1)	0.707	0.707
Uplift (Eq. 7-2)	Not calculated	Option 1 – 0.97
	-	Option 2 – 0.60

D.3 Design Example – 2: Braced Frame on Isolated and Combined Footings

D.3.1 Problem Statement:

A steel braced frame (Figure D.3-1) forms the seismic lateral force resisting system of three-story office building. Foundation demands are the pseudo seismic forces from a Linear Static Procedure. Determine the soil bearing acceptance ratio at the foundation soil interface and the foundation acceptance at the Collapse Prevention performance level assuming the following modeling options for the analysis of the superstructure:

1. Isolated footings, foundation soil interface is modeled as a fixed base.
2. Isolated footings, Foundation soil interface is modeled as a flexible base.
3. Footings interconnected by a 3' wide x 3' deep grade beam with soil interface modeled as a fixed base.
4. Footings interconnected by a 3' wide x 3' deep grade beam with soil interface modeled as a flexible base.

CODE: ASCE 41-23 and ASCE 41-17

SPECIFICATIONS:

Allowable Soil pressure (D + L): q_{allow} = 3 ksf

Existing Concrete Strength: f'_c = 3,000 psi.

Existing Steel Strength f_y = 40,000 psi.

Unit weight of concrete 150 pcf

GIVEN:

Footing dimensions:

B_f = 10.0 ft

L_f = 10 ft

Loading:

M_{OT} = 30,000 kip-ft

$P_{D\_Superstructure}$ = 300.0 kips (Dead)

P_L = 50.0 kips (0.25 Unreduced Live)

Maximum Axial Demand Capacity Ratio:

DCR_{max} = 2.0 given

Knowledge Factor:

κ = 1.0

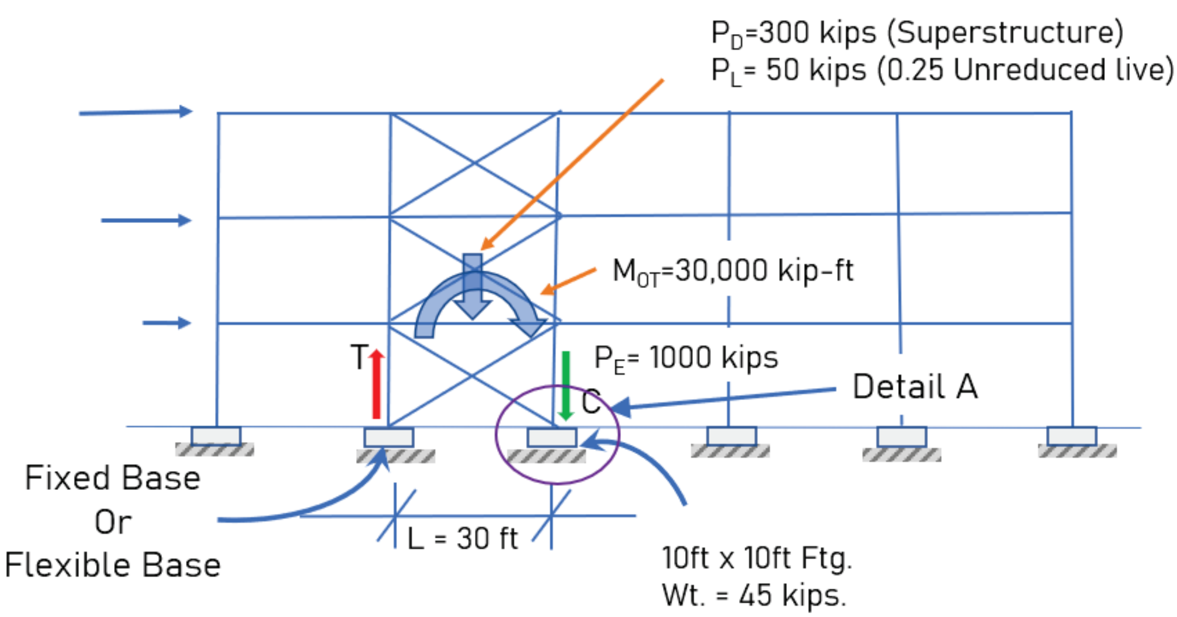


Figure D.3-2: Braced frame example.

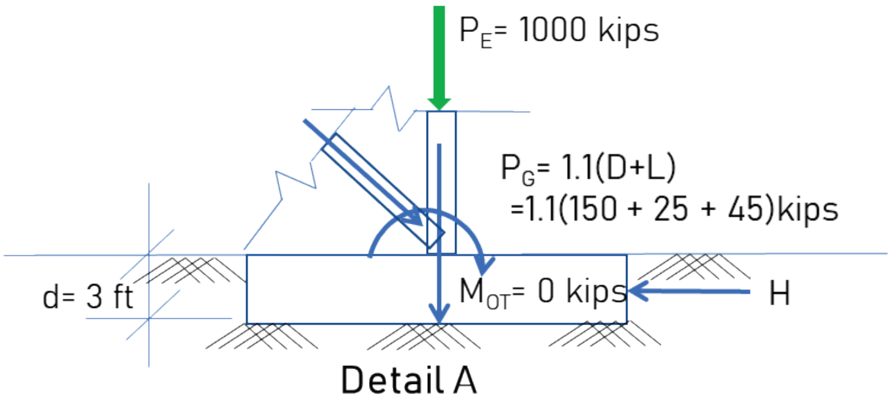


Figure D.3-2 Pseudo seismic demands on 10'x10 isolated footing under the braced frame.

D.3.2 CASE 1: Isolated Footings, Soil Foundation Interface Modeled as a Fixed Base

D.3.2.1 SOLUTION (PROPOSED PROVISIONS ASCE 41-23)

D.3.2.1.1 Soil Strength for short term seismic loads:

$q_{cDA} = 2(q_c) = 2(3)(q_{allow}) = 6(3) = 18\text{ksf}$ ASCE/SEI 41-23 (*Eq.* 8-9)

D.3.2.1.2 Determination of Axial load demand on the footing

Gravity load on each footing

$$P_{D\_superstructure} = 300\ kips$$

$$P_L = 50\ kips$$

$$P_{D\_Footing} = (10 \times 10\ \times\ 3)(0.150) =\ 45\ kips$$

$$P_G = \frac{300}{2} + \frac{50}{2} + 45 = 220\ kips$$

Seismic axial load (compression or uplift)

$$P_E = \frac{M_{OT}}{L'_f} = \frac{30{,}000}{30} = 1{,}000\ kips$$

Axial action compression load demand on the footing

$$P_{UF} = P_G + \frac{P_E}{DCR_{max}} = 1.1(220) + \frac{1000}{2} = 742\ kips \qquad \text{ASCE/SEI } 41-23\ (Eq. 8-13)$$

D.3.2.1.3 Acceptance criteria, axial compression:

Ductility m-factor multiplying soil bearing axial capacity

m_{CP} = 2.5 $\quad$ ASCE/SEI 41 – 23 Table 8-3

$$Acceptance\ Ratio\ (AR) = \frac{P_U}{m_{CP} q_{cDA} A_f} = \frac{742}{(2.5)(18)(100)} = 0.165$$

D.3.2.1.4 Acceptance criteria, axial action - uplift

m_{CP} = 8.0 $\quad$ ASCE/SEI 41 – 23 Table 8-3

$$P_D = \frac{300}{2} + 45 = 195\ kips$$

$$Acceptance\ Ratio\ (AR) = \frac{P_E}{0.9 m_{CP} P_D} = \frac{1000}{(0.9)(8)(195)} = 0.712$$

D.3.2.1.5 Governing Acceptance Ratio

Governing AR = 0.712

D.3.2.2 SOLUTION (ASCE/SEI 41-17)

hThere is non comparable solution in ASCE/SEI 41-17 when seismic axial loads and gravity are additiove for pure axial compression. There is only acceptance criteria for axial uplift and the acceptance ratios from ASCE/SEI 41-17 and from ASCE/SEI 41-23 is the same and is given as:

$$Acceptance\ Ratio\ (AR) = \frac{P_E}{0.9 m_{CP} P_D} = \frac{1000}{(0.9)(8)(195)} = 0.712 \quad \text{ASCE/SEI 41 – 17 §8.4.2.3.2.1}$$

D.3.3 CASE 2: Isolated Footings, Foundation Interface Modeled as a Flexible Base

D.3.3.1 SOLUTION (PROPOSED PROVISIONS ASCE 41-23)

D.3.3.1.1 Acceptance criteria, Axial axial - compression:

Ductility m-factor multiplying soil bearing axial capacity

m_{CP} = 3.0 $\quad$ ASCE/SEI 41-23 Table 8-6

$$Acceptance\ Ratio\ (AR) = \frac{P_{UF}}{m_{CP} q_{cDA} A_f} = \frac{742}{(3.0)(18)(100)} = 0.137$$

D.3.3.1.2 Acceptance Criteria, Axial action - Uplift

m_{CP} = 10.0 ASCE/SEI 41-23 Table 8-6

$$P_D = \frac{300}{2} + 45 = 195\ kips$$

$$Acceptance\ Ratio\ (AR) = \frac{P_E}{0.9m_{CP}P_D} = \frac{1000}{(0.9)(10)(195)} = 0.57$$

D.3.3.1.3 Governing Acceptance Ratio

Governing AR = 0.57

D.3.3.2 SOLUTION (ASCE/SEI 41-17)

There is non comparable solution in ASCE/SEI 41-17 when seismic axial loads and gravity are additiove for pure axial compression. There is only acceptance criteria for axial uplift and the acceptance ratios from ASCE/SEI 41-17 and from ASCE/SEI 41-23 is the same and is given as:

$$Acceptance\ Ratio\ (AR) = \frac{P_E}{0.9m_{CP}P_D} = \frac{1000}{(0.9)(10)(195)} = 0.57 \quad \text{ASCE/SEI}\ \ 41-17\ \ \S 8.4.2.3.2.2$$

D.3.4 CASE 3: Footings interconnected by a 3’ x 3’ grade beam, Fixed Base

D.3.4.1 SOLUTION (PROPOSED PROVISIONS ASCE 41-23)

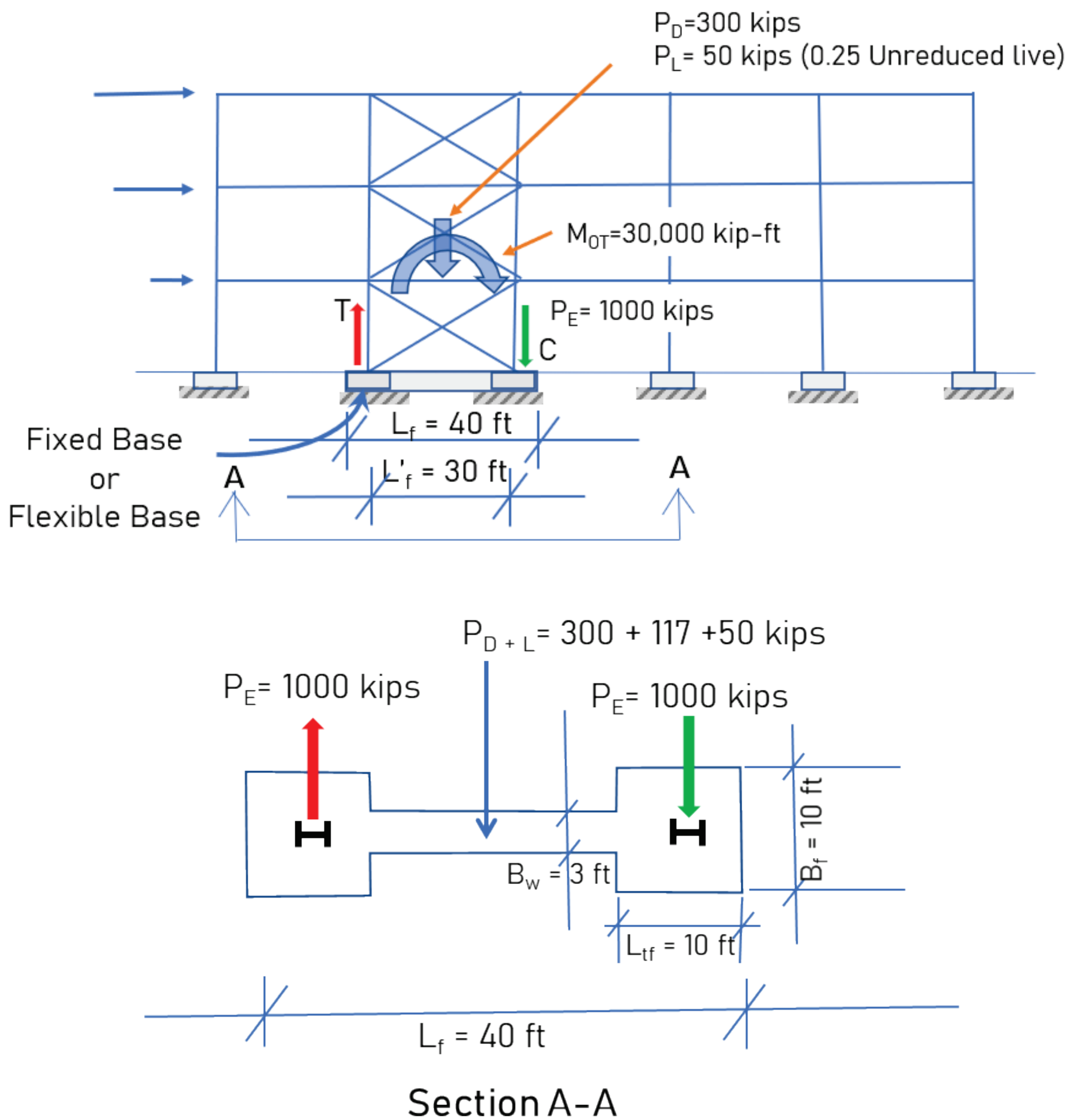


Figure D.3-3 Pseudo seismic demands on 10’x10 isolated footings under the braced frame interconnected by a grade beam.

D.3.4.1.1 Moment Capacity for Combined Axial and Moment action

Area of the combined footing

$$A_f = 2(B_f L_f) + (L_f - 2L_{tf})B_w = 260 ft^2$$

$P_E = 0\ kips$

$B_w = 3\ ft$

$P_G = P_D + P_L + P_{Foundation}$

$P_{Foundation} = 2(45) + (3)(40 - 20)(3)(0.150) = 117 \text{ kips}$

$P_{D+L} = 300 + 50 + 117 = 467 \text{ kips}$

Axial load demand:

$$P_{UF} = P_G + \frac{P_E}{DCR_{max}} = 1.1(467) + \frac{0}{2.0} = 513.7 \text{ kips} \qquad \text{ASCE/SEI } 41-23\ \ (Eq. 8-13)$$

Soil bearing pressure:

$$q = \frac{P_{UF}}{A_f} = \frac{513.7}{260} = 1.976 \text{ ksf}$$

Foundation moment capacity

$$M_{CE} = \frac{P_{UF}L'_f}{2}\left(1 - \frac{q}{q_{cDA}}\right) = \frac{513.7 * 40}{2}\left(1 - \frac{1.976}{18}\right) = 9146.3 \text{ kip} - \text{ft}$$

D.3.4.1.2 Acceptance Criteria, soil bearing, from ASCE/SEI 41-23 Eq. 8-21 is given as:

$$M_{OT} \le m\kappa M_{CE} \qquad \text{ASCE/SEI } 41-23\ (Eq. 8-21)$$

$m_{CP} = 4.0$ ASCE/SEI 41-23 (Table 8-4)

Rewriting Eq. 8-21 in terms of an Acceptance Ratio (AR)

$$AR = \frac{M_{OT}}{m\kappa M_{CE}} = \frac{30{,}000}{4 * 1 * 9146.3} = 0.82$$

D.3.4.2 SOLUTION ASCE 41-17: SOIL FOUNDATION INTERFACE MODELED AS A FIXED BASE

D.3.4.2.1 Soil strength upper bound

$q_c = (1 + C_v)(q_c) = (1+1)(3)(q_{allow}) = 6(3) = 18\text{ksf}$

D.3.4.2.2 Foundation Moment Capacity and Acceptance – When actions from seismic and gravity are additive

Axial load demand

Using the compression load combiation ASCE/SEI 41-17 (Eq. 7-1):

$$P_{UD} = P_G + \frac{P_E}{DCR} = 1.1 * 467 + \frac{0}{2.0} = 513.7\ kips$$

Soil bearing pressure

$$q = \frac{P_{UD}}{A_f} = \frac{513.7}{260} = 1.976\ ksf$$

Moment capacity

$$M_{CE} = \frac{P_{UD} L_f}{2}\left(1 - \frac{q}{q_c}\right) = \frac{513.7 * 40}{2}\left(1 - \frac{1.976}{18}\right) = 9146.3\ kip - ft$$

Acceptance Criteria, soil bearing:

$$m\kappa Q_{CE} > Q_{UD} \qquad ASCE\ 41-17\ (Eq.\ 7-36)$$

$$Q_{CE} = M_{CE} \qquad ASCE\ 41-17\ (Sec. 8.4.2.3.2.1)$$

$$Q_{UD} = M_{UD} \qquad ASCE\ 41-17\ (Eq. 7-34)$$

m-factor (overturning compression, Sec 8.4.2.3.2.1)

$m_{CP} = 4.0$

Acceptance Ratio (AR)

$$AR = \frac{M_{UD}}{m\kappa M_{CE}} = \frac{30{,}000}{4 * 1 * 9146.3} = 0.82$$

D.3.4.2.3 Foundation Moment Capacity and Acceptance – When actions from seismic forces and gravity loads are counteracting

Axial load demand

When load combiation dead load is counteractive ASCE/SEI 41-17 (Eq. 7-2)applies.

$$P_{UD} = P_G - \frac{P_E}{DCR} = 0.9 * (300 + 117) - \frac{0}{2.0} = 375.3\ kips$$

Soil bearing pressure:

$$q = \frac{P_{UD}}{A_f} = \frac{375.3}{260} = 1.443\ ksf$$

Moment Capacity:

$$M_{CE} = \frac{P_{UD}L_f}{2}\left(1 - \frac{q}{q_c}\right) = \frac{375.3 * 40}{2}\left(1 - \frac{1.443}{18}\right) = 6904.1\ kip - ft$$

Acceptance Criteria, soil bearing

$$m\kappa Q_{CE} > Q_{UD} \qquad ASCE/SEI\ 41-17\ (Eq.\ 7-36)$$

$$Q_{CE} = M_{CE} \qquad ASCE/SEI\ 41-17\ (Sec. 8.4.2.3.2.1)$$

$$Q_{UD} = M_{UD} \qquad ASCE/SEI\ 41-17\ (Eq. 7-34)$$

Acceptance Ratio (AR) The code is not specific which m-factor is to be used in this case, therefore the following are options are considered:

Option 1: m_{CP} = 4.0, overturning compression

$$AR = \frac{M_{OT}}{m\kappa M_{CE}} = \frac{30,000}{4 * 1 * 6904.1} = 1.086$$

Option 2: m_{CP} = 8.0 when overturning results in axial uplift force demand

$$AR = \frac{M_{OT}}{m\kappa M_{CE}} = \frac{30,000}{8 * 1 * 6904.1} = 0.543$$

D3.5 CASE 4: Footings interconnected by a 3' x 3' grade beam, Flexible Base

D.3.5.1 SOLUTION (PROPOSED PROVISIONS ASCE/SEI 41-23)

D.3.5.1.1 Acceptance Criteria Soil Bearing and Overturning, ASCE/SEI 41-23, Section 8.4.5.2.3.1.3

m-Factor for I-shaped footing

For the I-shaped footing, for the parameters and *m*-factors defined in Section 8.4.5.2.3.1

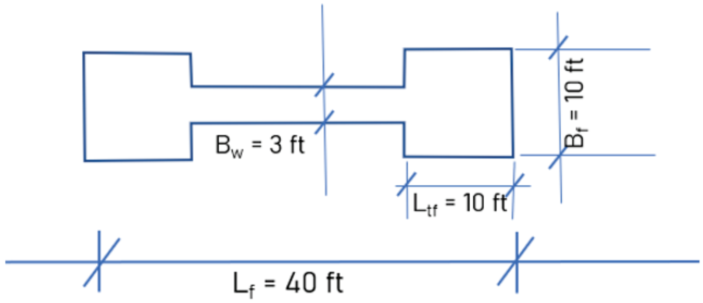
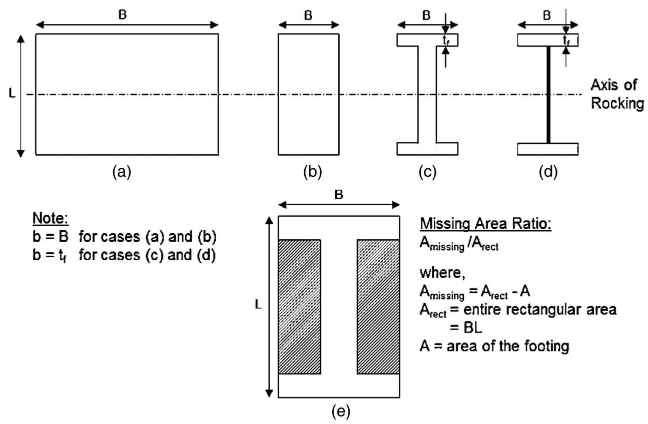


Figure D.3-4 Isolated footing interconnected by a grade beam



ii. I-Shape

$\frac{b}{L_c}$	$\frac{A_{rect} - A_f}{A_{rect}}$	$\frac{A_c}{A_f}$			
$1 \le \frac{b}{L_c} \le 10$	0.3				
		0.20	3	5	7
		0.5	1.5	2.5	3.5
		1	1	1	1
$1 \le \frac{b}{L_c} \le 10$	0.6				
		0.20	2.5	4.5	5.5
		0.5	1	2	2
		1	1	1	1
$1 \le \frac{b}{L_c} \le 10$	1				
		0.20	2	3.5	4.5
		0.5	1	1.5	1.5
		1	1	1	1

Figure D.3-5 m-Factors from ASCE/SEI 41-23 Table 8-7

$$L_c = \frac{P_U}{B_f q_{cDA}} = \frac{513.7}{10 * 18} = 2.854\ ft$$

$$A_c = \frac{P_U}{q_{cDA}} = 28.539;\ ft^2$$

$$A_{rect} = 400;\ ft^2$$

$$A_f = 260;\ ft^2$$

$$A_{miss\_ratio} = \frac{A_{rect} - A_f}{A_{rect}} = \frac{140}{400} = 0.35;$$

$$b_{ratio} = \frac{b}{L_c} = \frac{10}{2.854} = 3.504;\ \ (1 \leq \frac{b}{L_c} \leq 10)$$

$$A_{c\_ratio} = \frac{A_c}{A_f} = \frac{28.539}{260} = 0.11 < 0.2;$$

Interpolating the m-factor from Table 8-7

$$m_{CP} = 7 - (7 - 5.5)\frac{(A_{miss\_ratio} - 0.3)}{(0.6 - 0.3)} = 6.75;$$

$m_{CP} = 6.75$

Acceptance Criteria, soil bearing, from ASCE/SEI 41-23 Eq. 8-21 is given as:

$$M_{OT} \leq m\kappa M_{CE} \qquad \text{ASCE/SEI 41} - 23\ (\ Eq. 8 - 21)$$

Rewriting Eq. 8-21 in terms of an Acceptance Ratio (AR)

$$AR = \frac{M_{OT}}{m\kappa M_{CE}} = \frac{30{,}000}{6.75 * 1 * 9146.3} = 0.486$$

D.3.5.2 SOLUTION ASCE 41-17: FOUNDATION INTERFACE MODELED AS A FLEXIBLE BASE

D.3.5.2.1 Foundation Moment Capacity – When actions from seismic and gravity are additive

$$M_{CE} = \frac{P_{UD}L_f}{2}\left(1 - \frac{q}{q_c}\right) = \frac{513.7 * 40}{2}\left(1 - \frac{1.976}{18}\right) = 9146.3\ kip - ft$$

Acceptance Criteria, soil bearing:

$$m\kappa Q_{CE} > Q_{UD} \qquad ASCE/SEI\ 41 - 17\ (Eq.\ 7 - 36)$$

$$Q_{CE} = M_{CE} \qquad ASCE/SEI\ 41 - 17\ (Sec. 8.4.2.3.2.1)$$

$$Q_{UD} = M_{UD} \qquad ASCE/SEI\ 41 - 17\ (Eq. 7 - 34)$$

Acceptance Ratio (AR)

$$AR = \frac{M_{OT}}{m\kappa M_{CE}} = \frac{30{,}000}{6.75 * 1 * 9146.3} = 0.486$$

D.3.5.2.2 Foundation Moment Capacity and Acceptance Criteria – When actions from seismic forces and gravity loads are counteracting

Axial load demand, when load combiation dead load is counteractive (Eq. 7-2):

$$P_{UD} = P_G - \frac{P_E}{DCR} = 0.9 * (300 + 117) - \frac{0}{2.0} = 375.3\ kips$$

Soil bearing pressure:

$$q = \frac{P_{UD}}{A_f} = \frac{375.3}{260} = 1.443\ ksf$$

Moment Capacity:

$$M_{CE} = \frac{P_{UD} L_f}{2}\left(1 - \frac{q}{q_c}\right) = \frac{375.3 * 40}{2}\left(1 - \frac{1.443}{18}\right) = 6904\ kip - ft$$

Acceptance Criteria, soil bearing

$$m\kappa Q_{CE} > Q_{UD} \qquad ASCE\ 41 - 17\ (Eq.\ 7 - 36)$$

$$Q_{CE} = M_{CE} \qquad ASCE\ 41 - 17\ (Sec. 8.4.2.3.2.1)$$

$$Q_{UD} = M_{UD} \qquad ASCE\ 41 - 17\ (Eq. 7 - 34)$$

Determination *of m-factor (ASCE 41-17, Sec. 8.4.2.3.2.2)*

ii. I-Shape

$\frac{b}{L_c}$	$\frac{A_{rect} - A_f}{A_{rect}}$	$\frac{A_c}{A_f}$			
$1 \le \frac{b}{L_c} \le 10$	0.3	0.20	3	5	7
		0.5	1.5	2.5	3.5
		1	1	1	1
$1 \le \frac{b}{L_c} \le 10$	0.6	0.20	2.5	4.5	5.5
		0.5	1	2	2
		1	1	1	1
$1 \le \frac{b}{L_c} \le 10$	1	0.20	2	3.5	4.5
		0.5	1	1.5	1.5
		1	1	1	1

$$L_c = \frac{P_{UD}}{B_f q_{cDA}} = \frac{375.3}{10 * 18} = 2.085\ ft$$

$$A_c = \frac{P_{UD}}{q_{cDA}} = 20.85;\ ft^2$$

$$A_{rect} = 400;\ ft^2$$

$$A_f = 260;\ ft^2$$

$$A_{miss} = \frac{A_{rect} - A_f}{A_{rect}} = 0.35$$

$$b_{ratio} = \frac{b}{L_c} = 4.796;\ \ (1 \le \frac{b}{L_c} \le 10)$$

$$A_{c\_ratio} = \frac{A_c}{A_f} = \frac{20.85}{260} = 0.08 < 0.2;$$

Interpolating the m-factor from Table 8-3

$$m_{CP} = 7 - (7 - 5.5)\frac{(A_{miss} - 0.3)}{(0.6 - 0.3)} = 6.75;$$

m_{CP} = 6.75

Acceptance Ratio (AR)

The code is not specific which m-factor is to be used in this case, therefore the following are options are considered:

Option 1: m_{CP} = 6.75, overturning compression

$$AR = \frac{M_{UD}}{m\kappa M_{CE}} = \frac{30{,}000}{6.75 * 1 * 6904} = 0.644$$

Option 2: m_{CP} = 10.0 when overturning results in axial uplift force demand

$$AR = \frac{M_{UD}}{m\kappa M_{CE}} = \frac{30{,}000}{10 * 1 * 6904} = 0.435$$

D.3.6 Summary

For both ASCE/SEI 41-17 and ASCE/SEI 41-23, the governing Acceptance Ratio (AR) is higher when the foundations are interconnected than is they were treated as isolated footings for both the fixed base and flexible base solutions. Similar to the results from Design Example 1, the results using ASCE/SEI 41-17 and more conservative than the results using ASCE/SEI 41-23 if the compression m-values are use when seismic loads and gravity are counteraction. ASCE/SEI 41-17 and 41-23 give identical results if only the load combination where seismic and gravity are additive is used.

Case 1: Acceptance Ratio: Isolated Footings, Foundation Interface Modeled as a Fixed Base

Load Combination	ASCE 41-23 (Proposed)	ASCE/SEI 41-17
Compression	0.165	N/A
Uplift	0.712	0.712
Governing LC	0.712	0.712

Case 2: Acceptance Ratio: Isolated Footings, Foundation Interface Modeled as a Flexible Base

Load Combination	ASCE 41-23 (Proposed)	ASCE/SEI 41-17
Compression	0.137	N/A
Uplift	0.57	0.57
Governing LC	0.57	0.57

Case 3: Acceptance Ratio: Combined Footings, Foundation Interface Modeled as a Fixed Base

Load Combination	ASCE 41-23 (Proposed)	ASCE/SEI 41-17
Compression	0.82	0.82
Uplift	N/A	Option 1 = 1.086
		Option 2 = 0.543
Governing LC	**0.82**	**0.82 or 1.086**

Case 4: Acceptance Ratio: Combined Footings, Foundation Interface Modeled as a Flexible Base

Load Combination	ASCE/SEI 41-23 (Proposed)	ASCE/SEI 41-17
Compression	0.486	0.486
Uplift	N/A	Option 1 = 0.644
		Option 2 = 0.435
Governing LC	0.486	0.486 or 0.644

D.4 Design Example – 3: Stair Tower on a Mat Foundation

D.4.1 Problem Statement:

A reinforced concrete shear wall stair tower of a five-story building with 12-foot floor to floor heights is supported on a 3´ thick mat foundation as shown in Figure D.4-1, with top of footing embedded 1' below the ground surface. The shear walls also act as bearing walls resisting gravity loads from the floor slabs in addition to the self-weight of the wall. The gravity dead and live loads are transferred from the walls to the foundation at the top of the Mat Foundation. The applied dead and live loads assumed as (D + 0.25 L) at the top of the 12" thick walls are 1.5 klf and 0.5 klf for the 8" thick walls.

The walls also resist overturning seismic demand from a fixed base analysis at the collapse prevention performance level of 52,800 kip-ft for overturning about the X- axis and 42,240 kip-ft for overturning about the Y- axis acting concurrently in the two orthogonal directions X- and Y-, for orthogonal load combinations of 100% and 30%. Pseudo seismic axial fluctuation on the wall can be ignored.

Determine the soil overturning acceptance using each of the following per Section 8.4.4.1.2 of ASCE/SEI 41-23:

1. Footing considered as an isolated footing.
2. Procedure 2 of Section 8.4.4.1.2.3 using spring stiffness values from Section 8.4.4.1.2.1 item 2
3. Procedure 1 of Section 8.4.4.1.2.3 using spring stiffness values from Section 8.4.4.1.2.1 item 3

CODE: ASCE 41-23

SPECIFICATIONS:

Allowable Soil pressure (D + L): q_{allow} = 3 ksf

Existing Concrete Strength: f'_c = 3,000 psi.

Existing Foundation Steel Strength f_y = 60,000 psi.

Unit weight of concrete 150 pcf

Standard penetration blow count N_{60} = 15

Atmospheric pressure p = 2.12 ksf

Site Class D

S_{xs} = 1.25 g

ν = 0.25

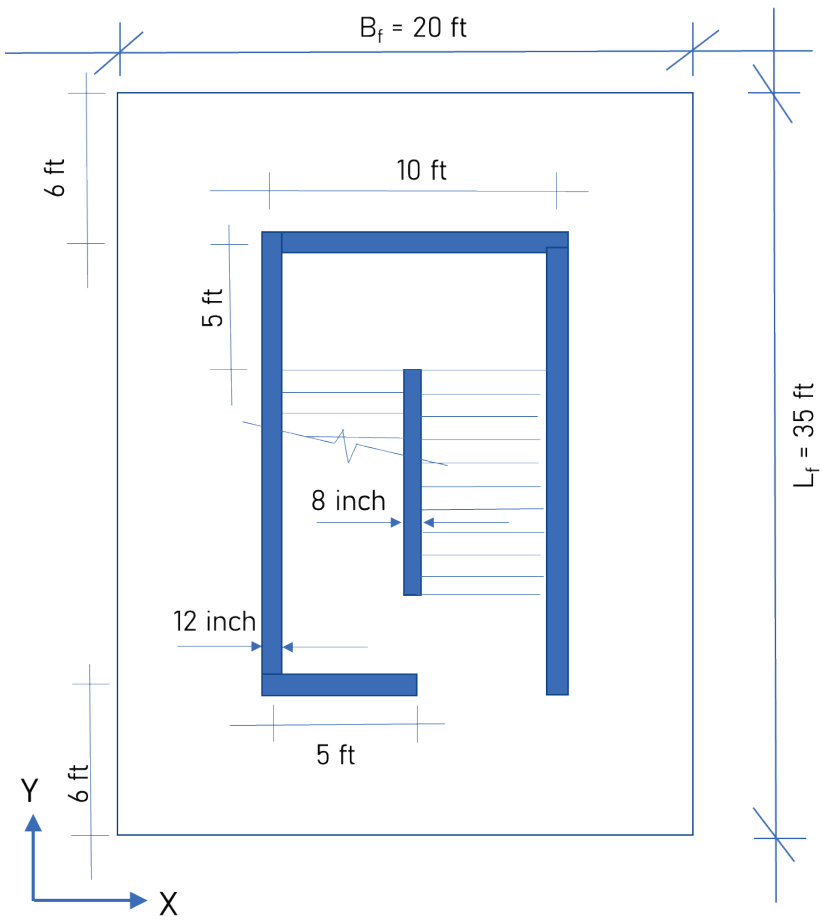


Figure D.4-3: Plan view of stair tower

D.4.2 Solution (Proposed Provisions ASCE 41-23):

GIVEN:

Footing dimensions:

B_f = 20.0 ft

L_f = 35 ft

Loading: Based on a pseudo seismic force demands including orthogonal load combinations of 100% and 30%.

$M_{OT,x}$ = 52,800 kip-ft

$M_{OT,y}$ = 42,240 kip-ft

Maximum Axial Demand Capacity Ratio:

DCR_{max} = 1.0 given

Knowledge Factor:

$\kappa = 1.0$

Ductility factor at collapse prevention performance level

$m_{C.P.}$ = 4.0

Soil Strength for short term seismic loads:

$q_{cDA} = 2(q_c) = 2(3)(q_{allow}) = 6(3) = 18\text{ksf}$ ASCE/SEI 41 – 23 (*Eq.* 8-9)

D.4.2.1 FOUNDATION CONSIDERED AS AN ISOLATED FOOTING

For the footing to be checked as an isolated footing, the footing is assumed as rigid, the axial load and moment demands on the footing are required to be applied at the centroid of the section. This requires the following steps:

- Determination of the axial load on the footing and the center of mass.
- Moment caused by gravity axial load eccentricity must be added to the seismic overturning moment of the footing without modification by the ductility of m-factor.
- When uniaxial overturning moments in both directions exceed $0.2mM_{CE}$ of the corresponding uniaxial moment capacity, bidirectional effects must be considered.

D.4.2.1.1 Determination of Mass Eccentricities at the Top of the Footing

For wall numbers shown in Figure D.4.-2, and the loads per floor tabulated in Table D.4-1, the center of mass and eccentricities are calculated as:

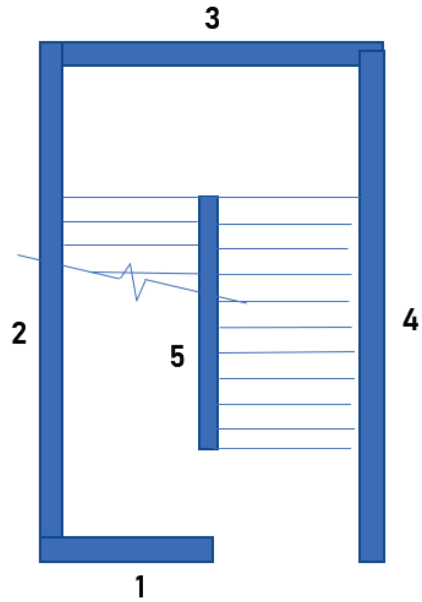


Figure D4-2: Wall numbers

Table D.4-1 Determination of the center of mass of the axial loads on the foundation

Wall Number	Unit weight per floor (klf)	Self weight (kips)	Length (ft)	Weight at top of Footing (klf)	Weight (kips	$X_{c.g.}$	Yc.g.	W*$X_{c.g.}$	W*Yc.g.
1	1.5	1.8	5	16.5	82.5	7.5	6	618.75	495
2	1.5	1.8	23	16.5	379.5	5	17.5	1897.5	6641.25
3	1.5	1.8	10	16.5	165	10	29	1650	4785
4	1.5	1.8	23	16.5	379.5	15	17.5	5692.5	6641.25
5	0.5	1.2	13	8.5	110.5	10	17.5	1105	1933.75
			74		**1117**			**10963.75**	**20496.25**

$X_{c.m.}$ = 10963.75/1117 = 9.82

$Y_{c.m.}$ = 20496.25/1117 = 18.25

e_x = 9.82 – 20/2 = -0.18 ft

$e_{y.}$ = 18.25 – 35/2 = 0.85 ft

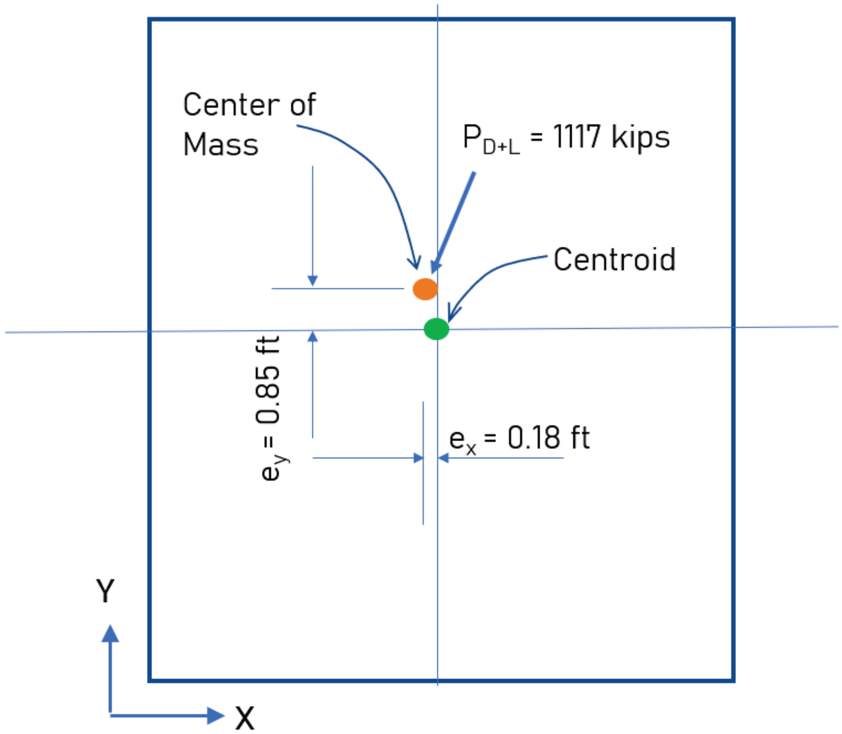


Figure D.4-3: Axial load at top of footing

D.4.2.1.2 Calculation of Inherent moment due to applied axial load

Axial load at the top of footing:

The center of mass of the gravity axial load at the top of the footing from Table D.4-1, is shown in Figure D.4-3. From ASCE/SEI 41-23 (Eq. 8-12), the axial load at the top of the footing $P_{U\_top\_of\_footing}$ is:

$$P_{U\_top\_of\_footing} = P_G + P_E/DCR_{max} \qquad \text{ASCE/SEI 41} - 23 \quad \text{(Eq. 8-9)}$$

$$= 1.1(1117) + 0 = 1228.7 \text{ kips}$$

Inherent Moment on footing due to eccentricities of the applied axial loads:

$$M_{x\_inherent} = -P_U \text{ x } e_{y.} = (1228.7)(0.85) = -1043.63 \text{ kip-ft}$$

$$M_{y\_inherent} = P_U \text{ x } e_{x.} = (1228.7)(-0.18) = -226.8 \text{ kip-ft}$$

Note: e_x and e_y were calculated without the 1.1 factor. It is assumed as negligible and is ignored.

D.4.2.1.3 Load demand at soil structure interface

Axial load at the soil structure interface

Weight of footing = 20 x 35 x 3 x 0.15 = 315 kips

P_{D+L} = 1117 + 315 = 1432 kips

$P_{UF} = 1.1P_G + P_E/DCR_{max}$ = 1.1(1432) + 0 = 1575.2 kips

D.4.2.1.4 Applied pseudo force moment

$M_{OT,x}$ = 52,800 kip-ft

$M_{OT,y}$ = 42,240 kip-ft

D.4.2.1.5 Check if Biaxial effects Need to be considered §8.4.4.1.1.3.1

q = 1575.2/(20)(35) = 2.25 ksf

$M_{CE,x\_uniaxial}$ = (1575.2)(35)/2(1-2.25/18) = 24,120 kip-ft

$M_{CE,y\_uniaxial}$ = (1575.2)(20)/2(1-2.25/18) = 13,783 kip-ft

$M_{OT,x}/m$ = 52,800/4 = 13,200 > 0.2(24,120) = 4,824 kip-ft

$M_{OT,y}/m$ = 42,240/4 = 10,560 > 0.2(13,783) = 2,756 kip-ft

Applied moments > 0.2 the m factor amplified moments in each direction; therefore, bi-directional effects need to be considered.

For rectangular footings, acceptance is based on either Eq. (8-20) or Eq. (8-21).

D.4.2.1.6 Acceptance based on ASCE/SEI 41-23 Eq. 8-20.

$$\left(\frac{M_{OT,x}}{m\kappa M_{CE,x}}\right)^2 + \left(\frac{M_{OT,y}}{m\kappa M_{CE,y}}\right)^2 \leq 1.0 \qquad \text{ASCE/SEI 41} - 23\ (\text{Eq. } 8 - 20)$$

The applied overturning moment needs to be adjusted by the inherent moment of the Mat foundation, therefore

$$M_{OT,x} = M_{OT,x\_applied} + M_{x\_inherent}$$

And

$$M_{OT,y} = M_{OT,y\_applied} + M_{y\_inherent}$$

$$\left(\frac{52{,}800-(4)(1044)}{(4)(1)(24120)}\right)^2+\left(\frac{42{,}240-(4)(226.9)}{(4)(1)(13{,}783)}\right)^2=0.816\le 1.0$$

D.4.2.1.7 Acceptance based on ASCE/SEI 41-23 (Eq. 8-21).

$$M_{OT}\ \le m\kappa M_{CE} \qquad \text{ASCE/SEI 41}-23\ (Eq. 8-21)$$

Where:

$$M_{OT}=\sqrt{(M_{OT.x})^2+\left(M_{OT,y}\right)^2} \qquad \text{ASCE/SEI 41}-23\ (Eq. 8-19)$$

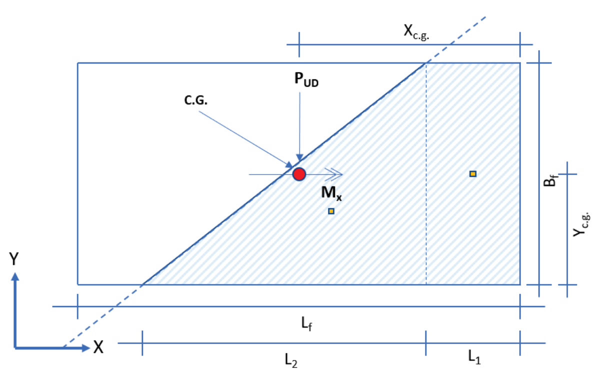
and

$$M_{CE}=\sqrt{\left(\frac{M_{OT.x}}{m}\right)^2+\left(M_{CE,y}\right)^2} \qquad \text{ASCE/SEI 41}-23\ (C8-5)$$

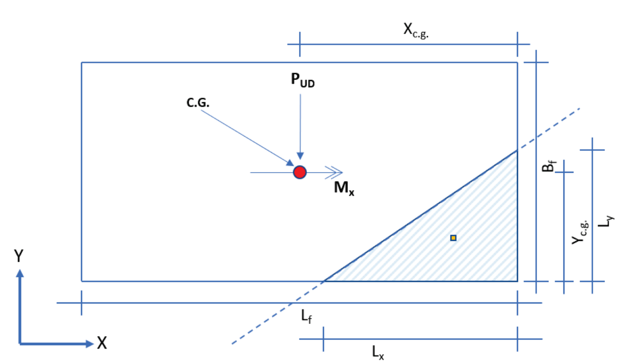
This requires the moment capacity to be determined considering bi-directional overturning moment action.

D.4.2.1.8 Foundation Moment Capacity for given axial load and weak axis moment

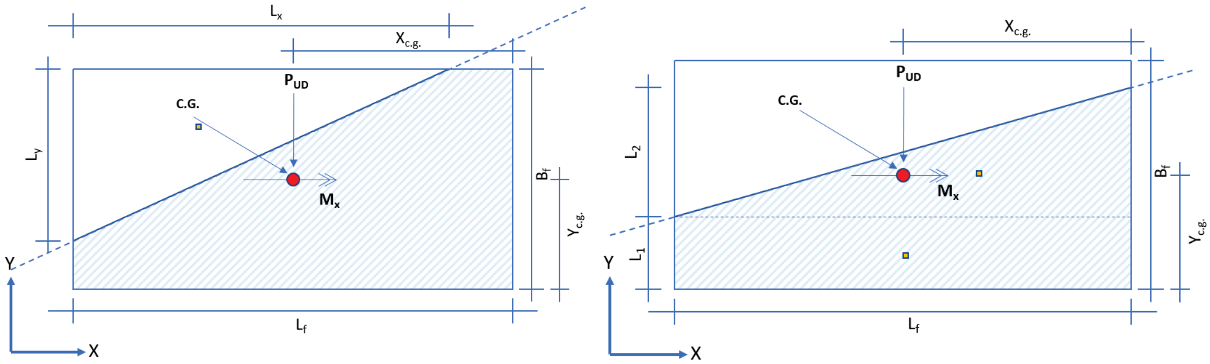
For an applied axial load P_U and an M_x or M_y moment on an isolated footing, the ultimate moment capacity in the orthogonal direction is determined by solving the equations of equilibrium of the applied load and the resisting soil pressure block under the footing. There are four distinct cases (Figure D.4-4) where a feasible solution is obtained for major axis moment for a given axial load and minor axis moment of the footing depending on where the resultant zero-pressure line intersects the footing edges.



Case 1



Case 2



Case 3 **Case 4**

FigureD.4-4 Soil pressure distribution cases where the Zero-Pressure line intersects footing edges

Case 1 – Zero pressure line intersects two opposite edges

If the zero-pressure line of the soil pressure block intersects two opposite edges of the footing as sown in Figure D.4-5, the ultimate moment in the orthogonal direction is given by the following expressions:

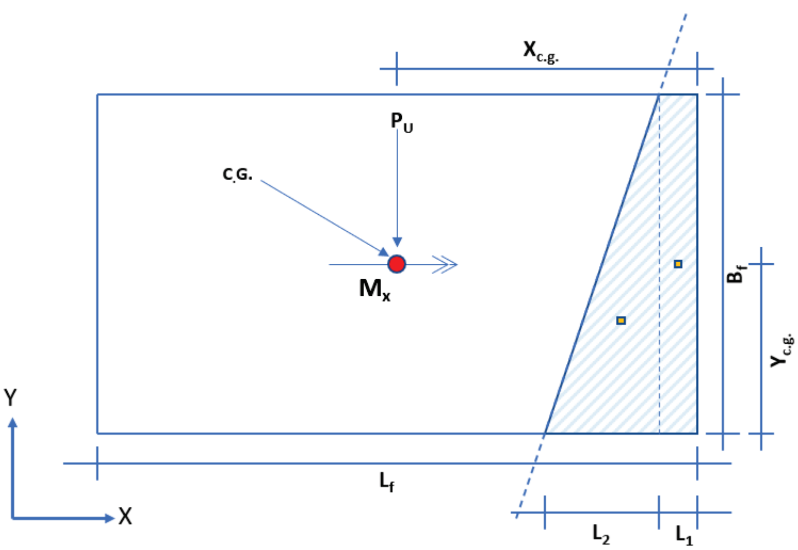


Figure D.4-5 Zero-Pressure Line intersects two opposite edges of the footing

$$M_{y,CE} = \frac{1}{2}q_c B_f \left\{L_1\left(X_{c.g.} - \frac{L_1}{2}\right) + \frac{1}{2}L_2\left(X_{c.g.} - L_1 - \frac{L_2}{3}\right)\right\}$$

$$L_2 = \frac{6}{q_c B_f}\left\{P_U - \frac{2\left(P_U Y_{c.g.} - M_x\right)}{B_f}\right\}$$

$$L_1 = \frac{P_U}{q_c B_f} - \frac{L_2}{2}$$

Interchanging the x and y coordinates to match the example, these equations can be written as:

$$M_{x,CE} = \frac{1}{2} q_c B_f \left\{ L_1 \left(Y_{c.g.} - \frac{L_1}{2} \right) + \frac{1}{2} L_2 \left(Y_{c.g.} - L_1 - \frac{L_2}{3} \right) \right\}$$

$$L_2 = \frac{6}{q_c B_f} \left\{ P_U - \frac{2\left(P_U X_{c.g.} - M_y\right)}{B_f} \right\}$$

$$L_1 = \frac{P_U}{q_c B_f} - \frac{L_2}{2}$$

The above equations are derived for the presumed actual demands on the footing accounting for overturning stability, therefore the pseudo force demands are divided by the *m*-factor for the desired performance level can be written as:

$M_y = M_{OT,y}/m_{C.P.} + M_{y\_inherent}$

M_y = 42,240/4 -226.8 = 10,333.2 kip-ft

Substituting in the above equations,

$$L_2 = \frac{6}{18 \times 20} \left\{ 1575.2 - \frac{2(1575.2 \times 10 - (10333.2)}{20} \right\}$$

L_2 = 17.22 ft

$$L_1 = \frac{1575.2}{18 \times 20} - \frac{17.22}{2}$$

L_1 = -4.23 ft

Solution is infeasible.

Case 2 – Zero pressure line intersects two adjacent edges

If the zero-pressure line of the soil pressure block intersects two adjacent edges of the footing as sown in Figure D.4-6, the ultimate moment is the orthogonal direction is given by the following expressions:

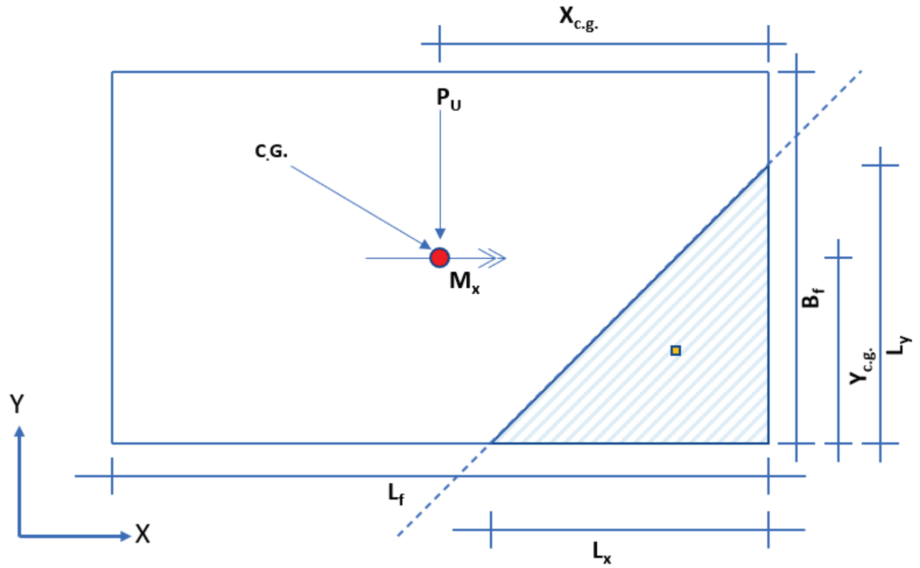


Figure D.4-6: Zero Pressure Line intersects two adjacent edges of the footing

$$M_{y,CE} = \frac{1}{2} q_{cDA} L_x L_y \left(X_{c.g.} - \frac{L_x}{3} \right)$$

$$L_y = 3 \left(Y_{c.g.} - \frac{M_x}{P_U} \right)$$

$$L_x = 2 \left(\frac{P_U}{q_c L_y} \right)$$

Since the major axis of overturning is about the X-axis, these equations can be written as:

$$M_{x,CE} = \frac{1}{2} q_c L_x L_y \left(Y_{c.g.} - \frac{L_y}{3} \right)$$

$$L_x = 3 \left(X_{c.g.} - \frac{M_y}{P_U} \right)$$

$$L_y = 2 \left(\frac{P_U}{q_c L_x} \right)$$

Where:

$M_y = M_{OT,y}/m_{C.P.} + M_{y\_inherent}$

M_y = 42,240/4 -226.8 = 10,333.2 kip-ft

$$L_x = 3 \left(10.0 - \frac{10{,}333.2}{1575.2} \right) ft$$

L_x = 10.3 ft < 20 ft ok

$$L_y = 2\left(\frac{1575.2}{18 \times 10.3}\right)$$

= 17 ft < 35 feet Ok

$$M_{x,CE} = \frac{1}{2} q_c L_x L_y \left(Y_{c.g.} - \frac{L_y}{3}\right)$$

$$M_{x,CE} = \frac{1}{2} 18 \times 10.3 \times 17 \left(17.5 - \frac{17}{3}\right)$$

$M_{x,CE}$ = 18,661.5 kip-ft

$$M_{CE} = \sqrt{\left(\frac{M_{OT.x}}{m}\right)^2 + \left(M_{CE,y}\right)^2} \qquad \text{ASCE/SEI } 41-23\ (C8-5)$$

For our case the x and y are interchanged, therefore:

$$M_{CE} = \sqrt{\left(\frac{M_{OT.y}}{m}\right)^2 + \left(M_{CE,x}\right)^2}$$

$$M_{OT} = \sqrt{(52{,}800 - (4)(1044))^2 + (42{,}240 - (4)(226.9))^2} = 63{,}817\ ft - kip$$

and

$$M_{CE} = \sqrt{(10{,}333)^2 + (18{,}661)^2}$$

$$M_{CE} = 21{,}331\ ft - kips$$

D.4.2.1.9 Foundation acceptance criteria

Acceptance Ratio **(AR) =** 63,817/(4 x 21,331) **= 0.75**

Foundation is acceptable for soil bearing but must consider all load combinations for direction of applied overturning moment.

D.4.3.2 SOLUTION USING PROCEDURE 2 OF SECTION 8.4.4.1.2.3

D.4.3.2.1 Effective width of the footing for soil stiffness calculation, Section 8.4.4.1.2.1, item 2:

From Table D.4-1, the axial compression load demand on the footing can be represented as wall line loads at the top of the footing as shown in Figure D.4-7.

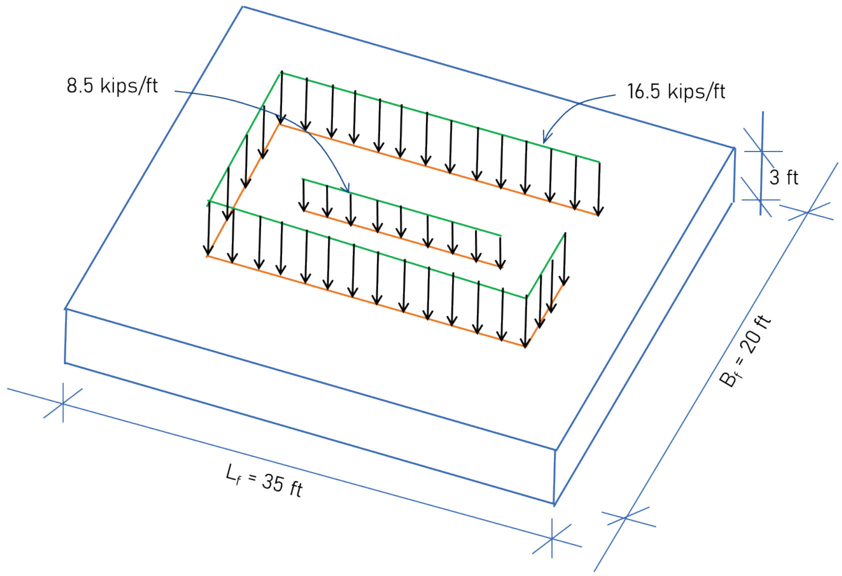


Figure D.4-7: Gravity load distribution at top of footing

Determination of effective width for the continuous 12' wall

Weight per unit length 16.5 kips/ft

Assume a width on all sides of the centerline of the wall of X'/2 feet as shown in Figure D.4-8.

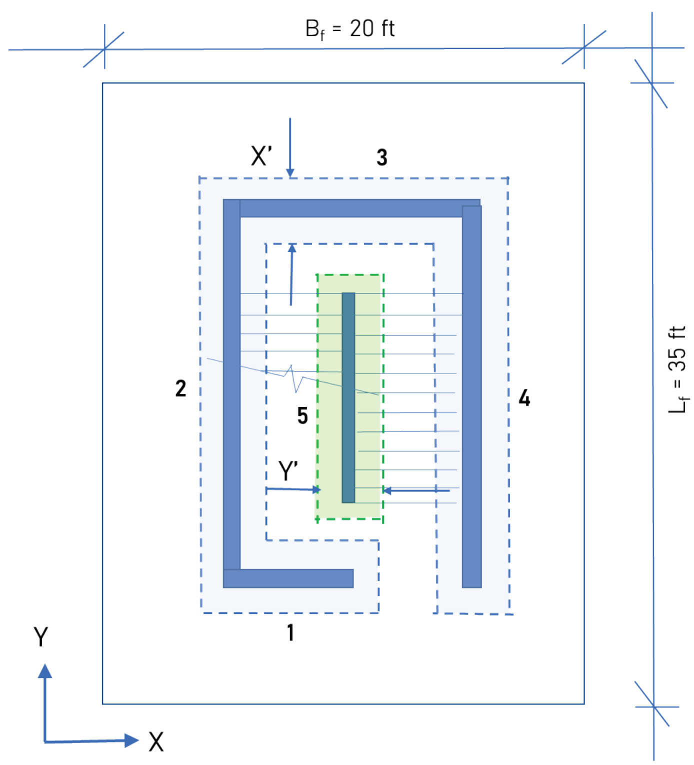


Figure D.4-8: Effective footing width required to support 1.5 times gravity load

Area required to support 1.5 times the gravity load is determined from:

A_{reqd} = 1.5(Axial Load)/q_{allow}

Or,

X'(23+10+5+23+2X'/2) =1.5 x 16.5 x (23 x 2 + 10 + 5)/3.0

Simplifying:

$X'^2 + 61X' + 503.25 = 0$

X' = (-61 + sqrt(61^2 + 4 x 503.25)}/2 = 7.36 ft

Determination of the effective width of for the **middle** *wall*

Assume a width on all sides of the centerline of the wall of Y'/2 feet.

Therefore, area required to support 1.5 the applied load for an allowable bearing value q_{allow} of 3 ksf is determined as:

(13 + Y') x Y' = 1.5 x (13 x 8.5)/3

$Y'^2 + 13 Y' – 55.25 = 0$

Y' = {-13 + sqrt(13^2 + 4 x 55.25)}/(2)

Y' = 2.9 ft

7.36/2 + 2.9/2 = 5.13 ft > 5 ft the distance between the centerline of the 12" and 8" wall, see Figure D.4-8.

Wall areas overlap, use an effective width of 10 + 7.36 = 17.36 ft

D.4.3.2.2 Determination of Soil Spring Stiffness

L_f/B_f = 35/17.36 = 2.0 < 3.0

Use spring stiffness from Figure 8-2 of ASCE 41-23

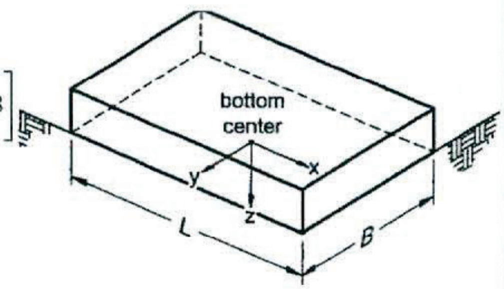
Degree of Freedom	Stiffness of Foundation at Surface	Note
Translation along x-axis	$K_{x,sur} = \frac{GB}{2-\nu}\left[3.4\left(\frac{L}{B}\right)^{0.65} + 1.2\right]$	
Translation along y-axis	$K_{y,sur} = \frac{GB}{2-\nu}\left[3.4\left(\frac{L}{B}\right)^{0.65} + 0.4\frac{L}{B} + 0.8\right]$	
Translation along z-axis	$K_{z,sur} = \frac{GB}{1-\nu}\left[1.55\left(\frac{L}{B}\right)^{0.75} + 0.8\right]$	
Rocking about x-axis	$K_{xx,sur} = \frac{GB^3}{1-\nu}\left[0.4\left(\frac{L}{B}\right) + 0.1\right]$	
Rocking about y-axis	$K_{yy,sur} = \frac{GB^3}{1-\nu}\left[0.47\left(\frac{L}{B}\right)^{2.4} + 0.034\right]$	Orient axes such that $L > B$. If $L = B$, use x-axis equations for both x-axis and y-axis.
Torsion about z-axis	$K_{zz,sur} = GB^3\left[0.53\left(\frac{L}{B}\right)^{2.45} + 0.51\right]$	

Figure D.4.9 Figure 8.2 in ASCE/SEI 41-23, elastic soil stiffness at soil foundation interface.

$$K_{z\_sur} = \frac{GB}{(1-\nu)}\left[1.55\left(\frac{L}{B}\right)^{0.75} + 0.8\right]$$

Where B = 17.36 and L = 35. Adjustment for embedment is permitted.

Calculation of small strain soil Shear Modulus, G

$G_0 = 120p_a(N_{60})^{0.77}$ ksf ASCE/SEI 41-23 (*Eq.* 8-1)

$G_0 = (120)(2.12)\,(15)^{0.77}$ ksf

$G_0 = 2047$ ksf

$S_{xs} = 1.25$

$S_{xs}/2.5 = 0.5$

For Site class D from Table 8-2

$$\frac{G}{G_0} = 0.5 - (0.5 - 0.1)\frac{(0.5 - 0.4)}{(0.8 - 0.4)} = 0.4$$

$$G = 0.4(2047) = 819\ ksf$$

$$K_{z\_sur} = \frac{819 \times 17.36}{(1-0.25)}\left[1.55\left(\frac{35}{17.36}\right)^{0.75} + 0.8\right]$$

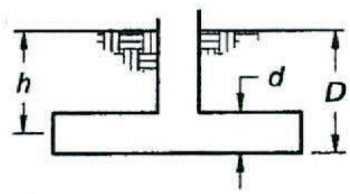
$$K_{z\_sur} = 64{,}864 \text{ k/ft}^3$$

or

$$k_{z\_sur} = 0.062 \text{ k/in}^3$$

Adjustment factor for embedment depth

Degree of Freedom	Correction Factor for Embedment
Translation along x-axis	$\beta_x = \left(1+0.21\sqrt{\frac{D}{B}}\right)\cdot\left[1+1.6\left(\frac{hd(B+L)}{BL^2}\right)^{0.4}\right]$
Translation along y-axis	$\beta_y = \left(1+0.21\sqrt{\frac{D}{L}}\right)\cdot\left[1+1.6\left(\frac{hd(B+L)}{LB^2}\right)^{0.4}\right]$
Translation along z-axis	$\beta_z = \left[1+\frac{1}{21}\frac{D}{B}\left(2+2.6\frac{B}{L}\right)\right]\cdot\left[1+0.32\left(\frac{d(B+L)}{BL}\right)^{2/3}\right]$
Rocking about x-axis	$\beta_{xx} = 1+2.5\frac{d}{B}\left[1+\frac{2d}{B}\left(\frac{d}{D}\right)^{-0.2}\sqrt{\frac{B}{L}}\right]$
Rocking about y-axis	$\beta_{yy} = 1+1.4\left(\frac{d}{L}\right)^{0.6}\left[1.5+3.7\left(\frac{d}{L}\right)^{1.9}\left(\frac{d}{D}\right)^{-0.6}\right]$
Torsion about z-axis	$\beta_{zz} = 1+2.6\left(1+\frac{B}{L}\right)\left(\frac{d}{B}\right)^{0.9}$



d = height of effective sidewall contact (may be less than total foundation height)

Figure D.4-10 Figure 8.2 in ASCE/SEI 41-23, Soil stiffness correction for embedment.

$$\beta_z = \left[1+\frac{1}{21}\frac{D}{B}\left(2+2.6\frac{B}{L}\right)\right]\left[1+0.32\left(\frac{d(B+L)}{BL}\right)^{\left(\frac{2}{3}\right)}\right]$$

$$\beta_z = \left[1+\frac{1}{21}\frac{(4)}{(17.36)}\left(2+2.6\frac{(17.36)}{35}\right)\right]\left[1+0.32\left(\frac{3(17.36+35)}{(17.36)(35)}\right)^{\left(\frac{2}{3}\right)}\right]$$

$$\beta_z = 1.17$$

$$k_z = (1.17)(0.062) = 0.072\ k/in^3$$

Alternatively:

$$k_{sv} = \frac{1.3G}{B_f(1-\nu)} \qquad \text{ASCE/SEI } 41-23\ (Eq. 8-22)$$

$$k_{sv} = \frac{1.3(819)}{17.36(1-0.25)} = 81.7\ k/ft^3$$

or

$$k_{sv} = \ 0.047 = k/in^3$$

Use $k_z = 0.072\ k/in^3$

D.4.3.2.3 Solution using Finite Element Modeling (ETABS):

Applied overturning loads to the footing:

For Procedure 2, soil does not resist tension, it is permitted to reduce the Pseudo seismic forces by the ductility factor m.

$M_{OT,x}$ / m_{CP} = 52,800/4 = 13,200 kip-ft

$M_{OT,y}$ / m_{CP} = 42,240/4 = 10,560 kip-ft

Where:

m_{CP} = 4.0, at the collpase prevention level from ASCE/SEI 41-23, Table 8-5.

The adjusted applied loads to the model resulting in the same overturning moment at the top of the footing are shown in Figure D.4-11 using the applied loads from Table D.4-2.

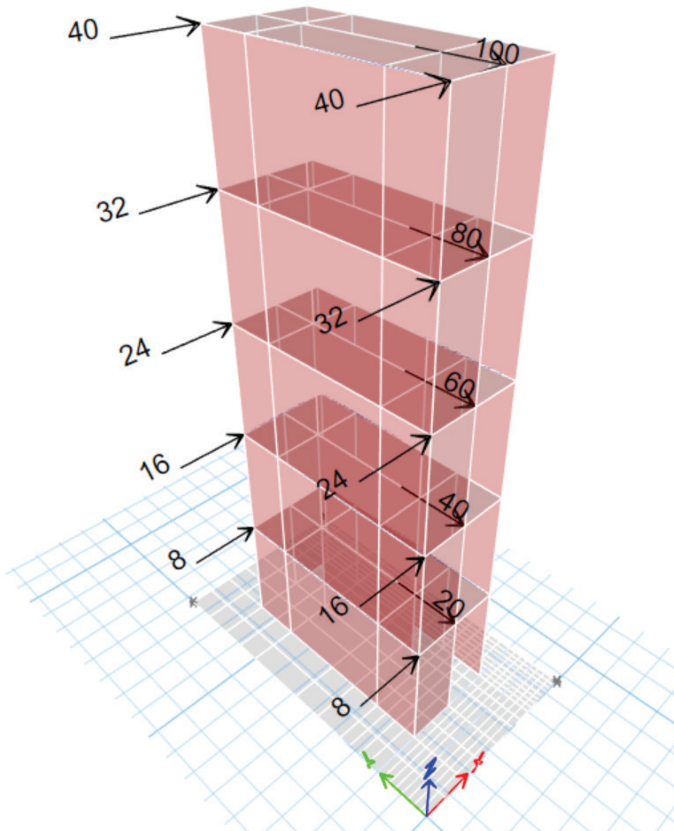


Figure D.4-11 Adjusted overturning demands on the footing

Table D.4-2 Applied loads to structure

	Force X (kips)	Height (ft)	M_y (kip-ft)
Story5	80	60	4800
Story4	64	48	3072
Story3	48	36	1728
Story2	32	24	768
Story1	16	12	192
Sum			**10560**

	Force Y (kips)	Height (ft)	M_x (kip-ft)
Story5	100	60	6000
Story4	80	48	3840
Story3	60	36	2160
Story2	40	24	960
Story1	20	12	240
Sum			**13200**

Maximum Soil Bearing Pressure:

The maximum soil pressure from the computer analysis (Figure D.4-12) assuming soil does not resist tension is 16.8ksf < 18 ksf, footing is **OK** for soil bearing.

Acceptance Ratio = 16.8/18 = **0.933**

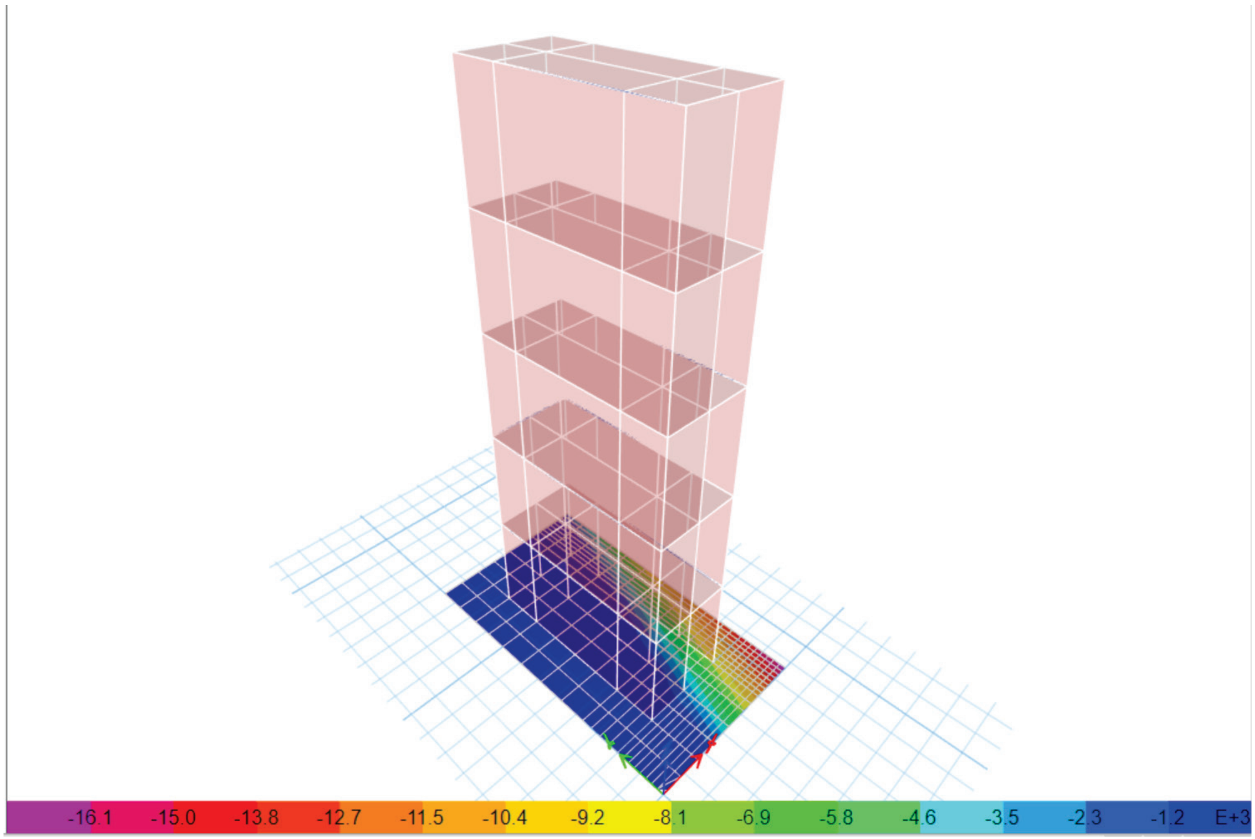


Figure D.4-12 Soil pressure distribution for adjusted loads

Note: For equivalence between the theoretical solution for moment capacity assuming a uniform soil pressure block, and the finite element solution, and adjustment factor is required. When the zero-pressure line of the soil pressure block at the ultimate moment capacity intersects two adjacent edges of the footing, the maximum soil bearing pressure is in the range of 1.5 – 1.69 times the maximum permitted soil bearing pressure for the same applied loads.

D.4.3.2.4 Verify Finite Element results with theoretical ultimate moment capacity:

To verify the finite element results with the theoretical results, the applied load to the foundation should be the moments corresponding to the footing ultimate capacity. The ultimate moment capacity in each direction is determined with appropriate adjustments to account for the center of mass offset of the applied axial load on the footing with footing centroid. The applied overturning moments should satisfy the following equations:

$M_{x,CE} = M_{x,OT} + M_{x\_inherent}$, and

$M_{y,CE} = M_{y,OT} + M_{y\_inherent}$

For an applied moment

$M_y = M_{y,CE} = 10560 - 260 = 10333$ kip-ft

$$L_x := 3\left(X_{c.g.} - \frac{M_y}{P_U}\right) = 10.32$$

$$L_y := 2\left(\frac{P_U}{q_{cDA} \cdot L_x}\right) = 16.959$$

$$M_{xCE} := \frac{1}{2}\, q_{cDA} \cdot L_x \cdot L_y \cdot \left(Y_{c.g.} - \frac{L_y}{3}\right) = 18661.5$$

Moment capacity in orthogonal direction, $M_{x,CE}$

$M_{x,CE}$ = 18661

Therefore $M_{x,OT}$ = $M_{x,CE}$ - $M_{x\_inherent}$

$M_{x,OT}$ = 18661 – (1043.63) = 19,705 kip-ft

Maximum Soil Pressure when applied Moments is at Moment capacity of the footing

Applied overturning moments:

> $M_{x,OT}$ = 10560 kip-ft
>
> $M_{y,OT}$ = 19,705 kip-ft

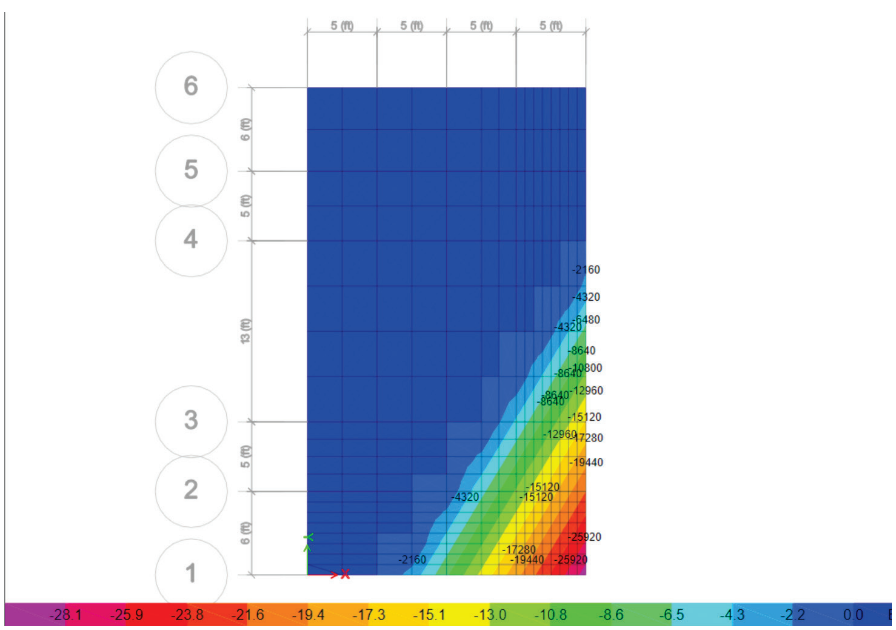
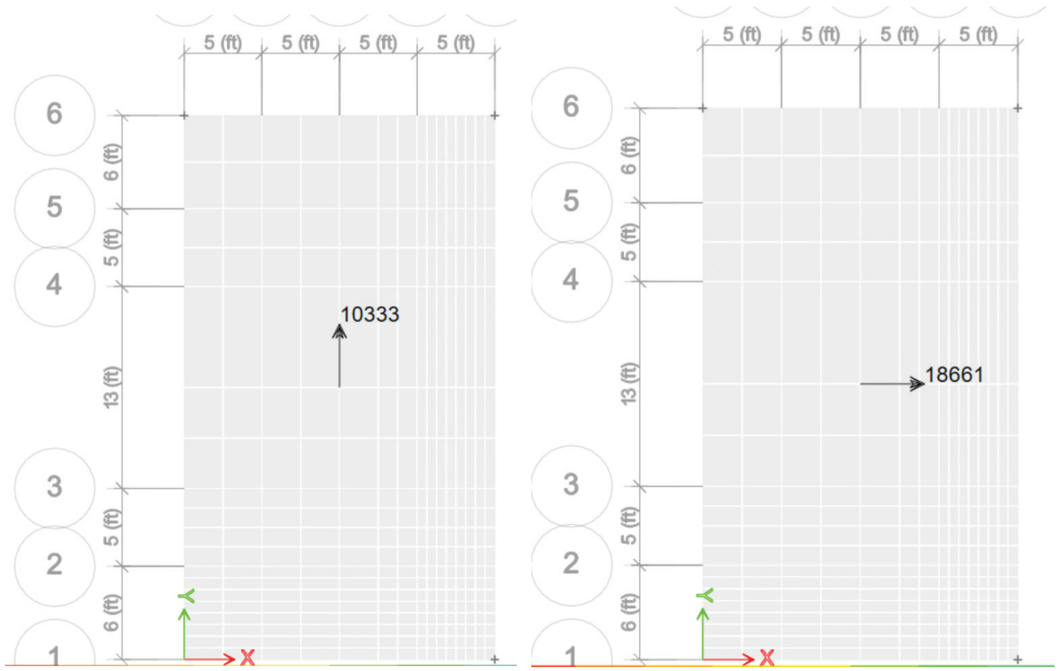


Figure D.4-13 Maximum soil pressure when applied moment is at the moment capacity of the footing

Maximum soil bearing pressure q_{max} = 29.2 ksf.

q_{max}/q_{cDA} = 29.2/18 = 1.62 < 1.69 ok

An additional study was done where the loads were applied directly to the footing, the rigidity of the foundation slab was increased. Loads lat_y and lat_x were applied in the Y- and X- directions as nonlinear static load case. Load case lat_x was applied after load case lat_y was applied which in turn was applied used dead load as the initial conditions.



Load Case: Lat_y **Load Case: Lat_x**

Figure D.4-14 Applied moments on the footing

The maximum soil bearing pressure at the corner was 29.655 ksf.

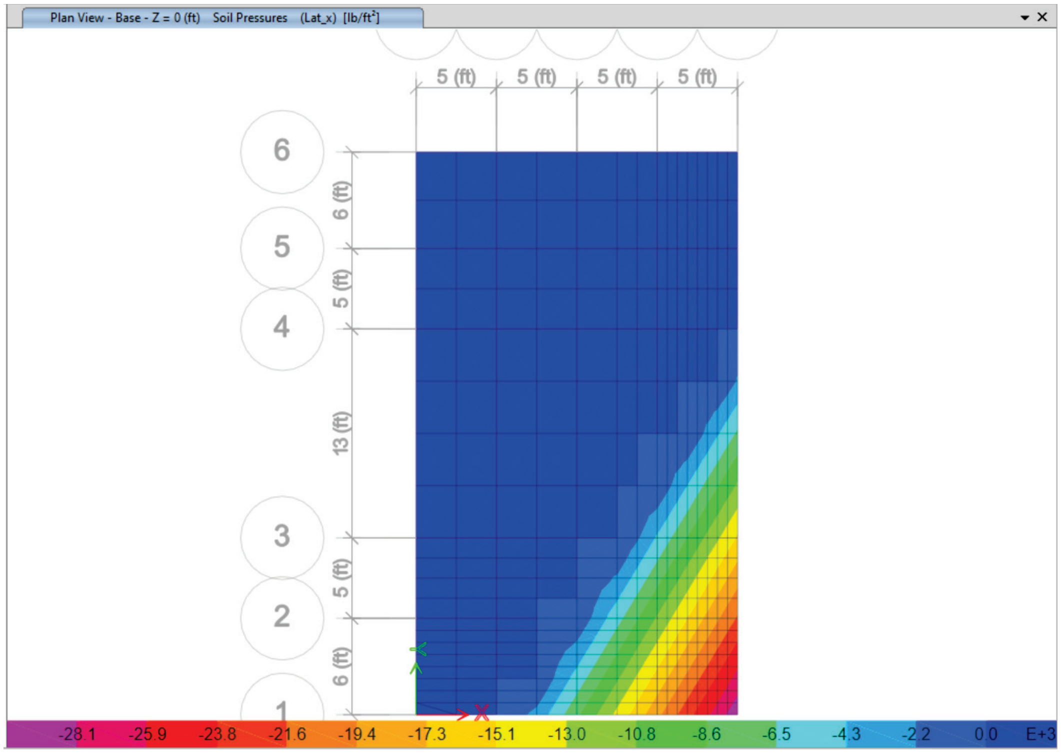


Figure D.4-15 Maximum soil pressure when applied moment is at the moment capacity of the footing for loads applied only on the footing.

Maximum soil bearing pressure q_{max} = 29.655 ksf.

q_{max}/q_{cDA} = 29.655/18 = 1.65 < 1.69 ok

D.4.3.3 SOLUTION USING PROCEDURE 1 OF SECTION 8.4.4.1.2.3

D.4.3.3.1 Soil stiffness calculation, Section 8.4.4.1.2.1 item 3:

Effective width of footing for soil stiffness calculation

The width B_f' used in the stiffness calculations is 4 times the footing thickness.

Therefore B_f' = 4(3) = 12 feet

L_f/B_f' = 35/12 = 2.91 < 3.0

Use spring stiffness from Figure 8-2 of ASCE 41-23

$$K_{z\_sur} = \frac{GB}{(1-\nu)}\left[1.55\left(\frac{L}{B}\right)^{0.75} + 0.8\right] \qquad \text{ASCE/SEI 41} - 23\ (\text{Figure } 8-2)$$

Where B = 12 and L = 35. Adjustment for embedment is permitted.

$$K_{z\_sur} = \frac{819 \times 12}{(1-0.25)}\left[1.55\left(\frac{35}{12}\right)^{0.75} + 0.8\right]$$

$$K_{z\_sur} = 55{,}800\ k/ft^3$$

or

$$k_{z\_sur} = 0.077\ k/in^3$$

Adjustment factor for embedment depth

$$\beta_z = \left[1 + \frac{1}{21}\frac{D}{B}\left(2 + 2.6\frac{B}{L}\right)\right]\left[1 + 0.32\left(\frac{d(B+L)}{BL}\right)^{\left(\frac{2}{3}\right)}\right] \qquad \text{ASCE/SEI } 41-23 \text{ (Figure } 8-2)$$

$$\beta_z = \left[1 + \frac{1}{21}\frac{(4)}{(17.36)}\left(2 + 2.6\frac{(17.36)}{35}\right)\right]\left[1 + 0.32\left(\frac{3(17.36+35)}{(17.36)(35)}\right)^{\left(\frac{2}{3}\right)}\right]$$

$$\beta_z = 1.21$$

$$k_z = (1.21)(0.077) = 0.093\ k/in^3$$

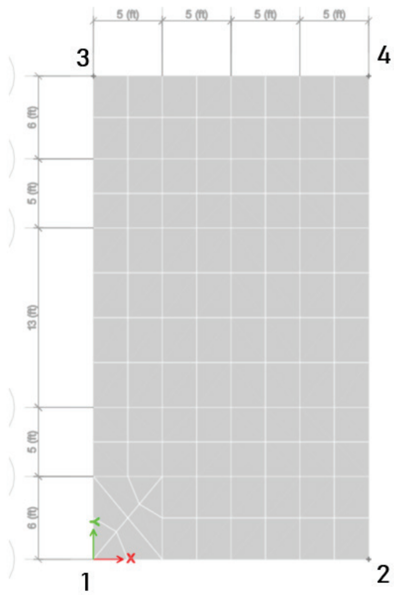
Note, for use in Procedure 1, this stiffness is required to be multiplied by 0.5 for elastic analysis.

Use k = 0.0465 k/in^3

D.4.3.3.2 Foundation Acceptance for soil bearing:

Acceptance is based on the footing rotation being less than 0.75 times the rotation values given in Table 8-8.

Footing deflections and soil pressure distribution under the footing are shown in Figure D.4-16 and Figure D.4-17 respectively.



Point	Defection (in)
1	0.31
2	-4.6
3	4.16
4	-1.18

Figure D.4-16 Deflections at the four corners of the footing

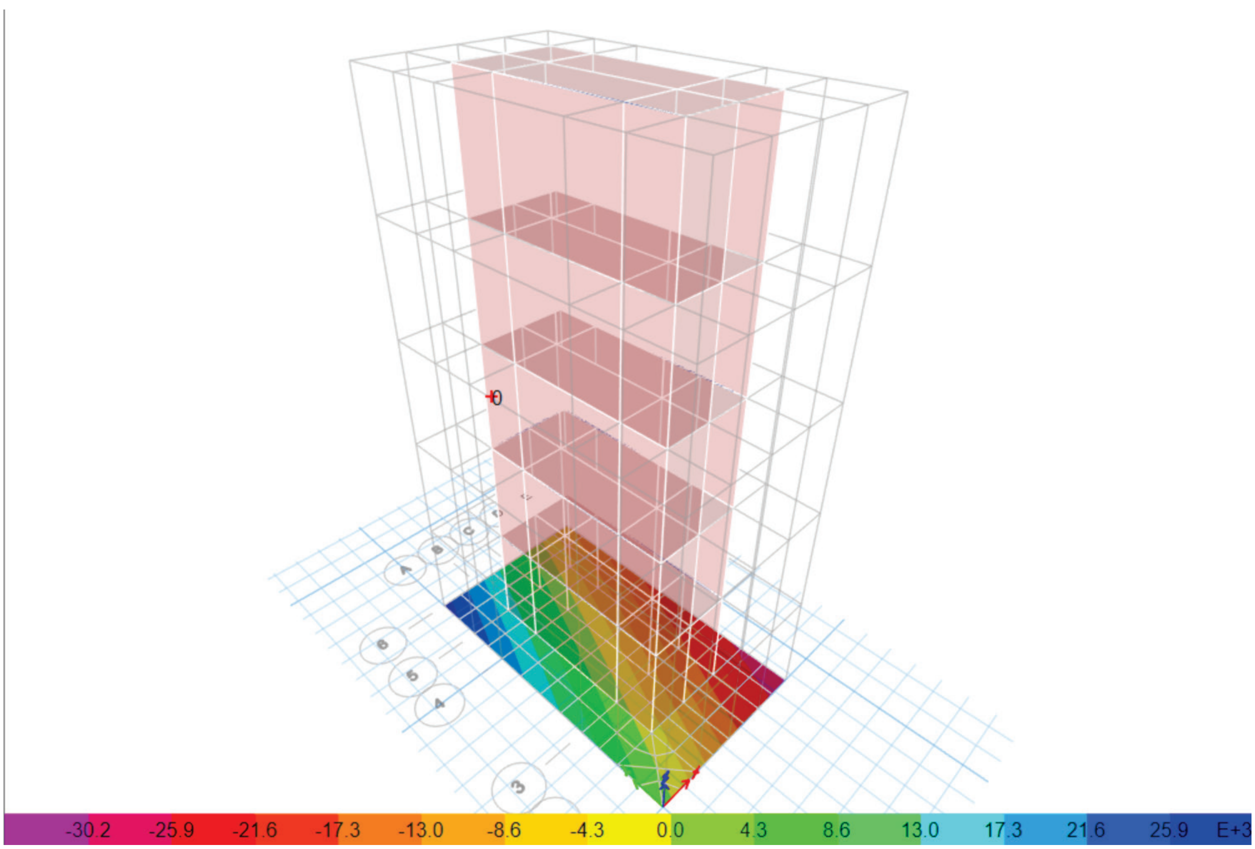
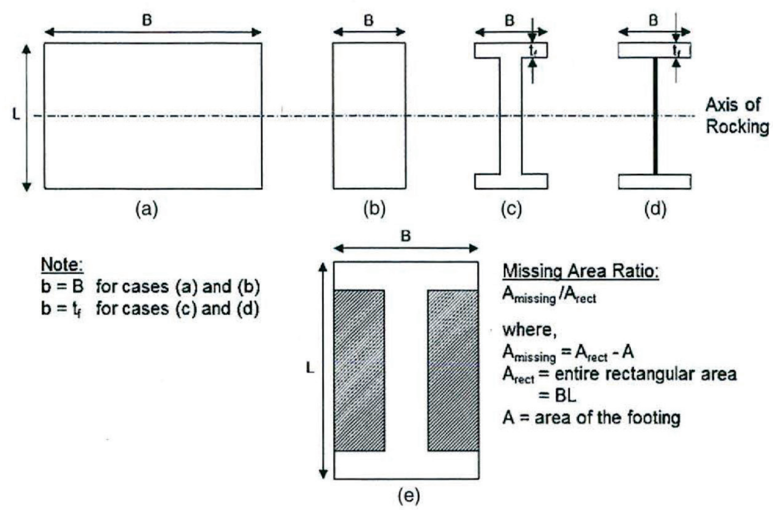


Figure D4-17 Soil pressure distribution under the footing.

q_{max} = 30.7 ksf, q_{min} = 27.9 ksf. Note: loads applied are unreduced pseudo seismic loads

Determination of allowable rotation (Procedure 1)

Footing Shape			Footing Rotation Angle, radians		Elastic Strength Ratio	Performance Level		
			g	d	f	IO	LS	CP
i. Rectangle[a,d]								
$\frac{b}{L_c}$	$\frac{A_{rect} - A_f}{A_{rect}}$	$\frac{A_c}{A_f}$						
≥ 10	0	0.02	0.009	0.1	0.5	0.02	0.08	0.1
		0.13	0.013	0.1	0.5	0.015	0.08	0.1
		0.5	0.015	0.1	0.5	0.002	0.003	0.004
		1	0.015	0.1	0.5	0.0	0.0	0.0
3	0	0.02	0.009	0.1	0.5	0.02	0.068	0.085
		0.13	0.013	0.1	0.5	0.011	0.06	0.075
		0.5	0.015	0.1	0.5	0.002	0.003	0.004
		1	0.015	0.1	0.5	0.0	0.0	0.0
1	0	0.02	0.009	0.1	0.5	0.02	0.056	0.07
		0.13	0.013	0.1	0.5	0.007	0.04	0.05
		0.5	0.015	0.1	0.5	0.002	0.003	0.004
		1	0.015	0.1	0.5	0.0	0.0	0.0
0.3	0	0.02	0.009	0.1	0.5	0.01	0.04	0.05
		0.13	0.013	0.1	0.5	0.007	0.024	0.03
		0.5	0.015	0.1	0.5	0.001	0.003	0.004
		1	0.015	0.1	0.5	0.0	0.0	0.0



$b = B_f = 20$ ft

$$L_c = \frac{P_U}{B_f q_{cDA}} = \frac{1575.2}{20 * 18} = 4.37\ ft$$

$$A_c = \frac{P_U}{q_{cDA}} = 87.5;\ ft^2$$

$$A_{rect} = A_f = 700;\ ft^2$$

$$A_{miss} = \frac{A_{rect} - A_f}{A_{rect}} = 0;\ ft^2$$

$$b_{ratio} = \frac{b}{L_c} = \frac{20}{4.37} = 4.57;$$

$$A_{c\_ratio} = \frac{A_c}{A_f} = \frac{87.5}{700} = 0.125;$$

Interpolating the θ_{CP} from Table 8-8

$$\theta_{CP} = 0.085 + (0.1 - 0.085)\frac{(b_{ratio} - 3.0)}{(10.0 - 3.0)} = 0.088;$$

$$\theta_{CP} = 0.088$$

Permitted rotation for elastic analysis using Procedure 1

$$\theta_{CP} = (0.75)0.088 = 0.066$$

Footing rotation demands are calculated based on the deflections at the four corners of the footing and the distance between them, and are given in Tables D.4-3 and D.4-4 below:

Table D.4-3 Distance between the corner points of the footing

Point	X coord (ft)	Y coord (ft)	Distance from Point (ft)			
			1	2	3	4
1	0	0	-	20	35	40.3
2	20	0		-	40.3	35
3	0	35			-	20
4	20	35				-

Table D.4-4 Footing rotation demands compared with allowable

Point	Defection (in)	Rotation from Point (radians)				Maximum Rotation	Allowable Rotation	Acceptance Ratio
		1	2	3	4			
1	0.31	-	0.020	0.009	0.008	0.020	0.066	0.31
2	-4.6		-	0.018	0.021	0.021	0.066	0.32
3	4.16			-	0.022	0.022	0.066	0.34
4	-1.18				-	-	-	-

Using this procedure, the maximum acceptance ratio, AR = 0.34.

Footing is acceptable for soil bearing using Procedure 1, Max. AR = 0.34.

D.4.3 Summary

A comparison of AR for soil bearing for the case study example between the various methods is shown below:

	Isolated Footing	Procedure 1	Procedure 2
Acceptance Ratio	Eq. (8-20): 0.82	0.34	0.93
	Eq. (8-21): 0.75		

Note: Procedure 1 is dependent on the stiffness used, so this result will change depending on the soil stiffness values used.