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.

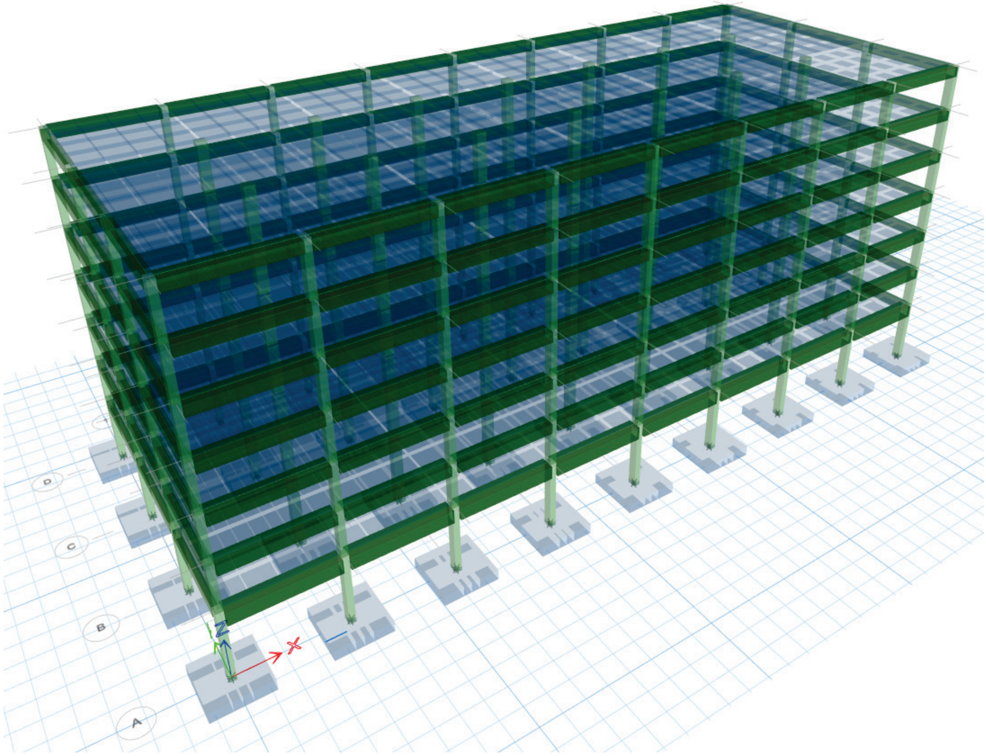


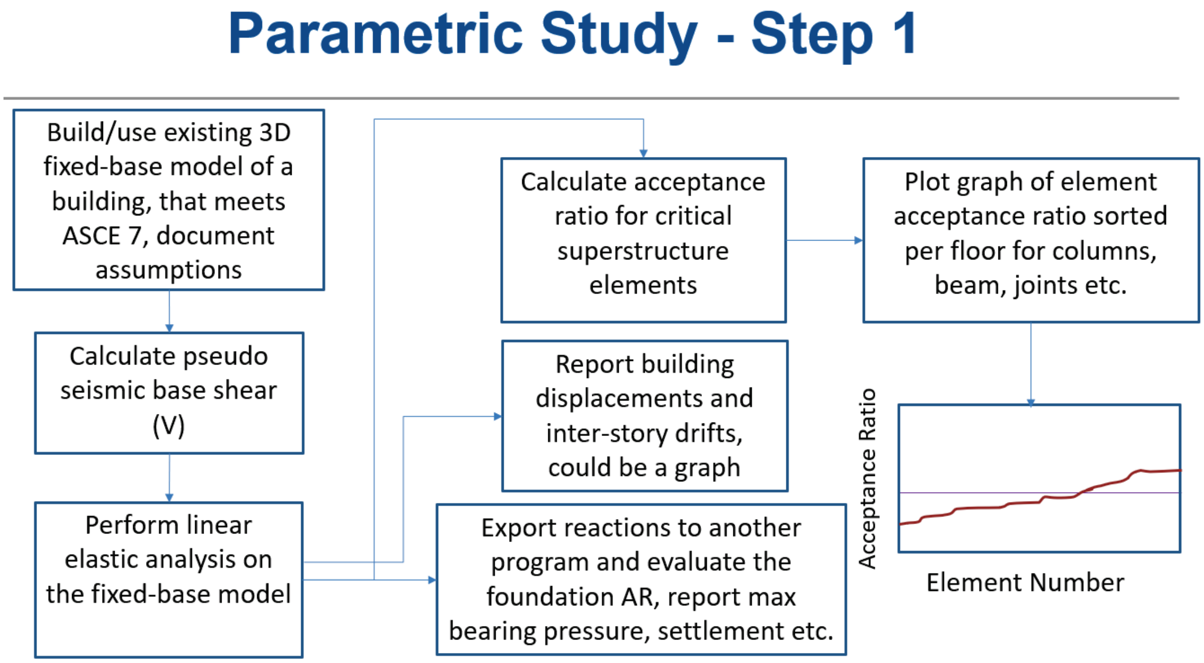
Figure C-1 **7-story Reinforced Concrete Building – Archetype Building 2**

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.

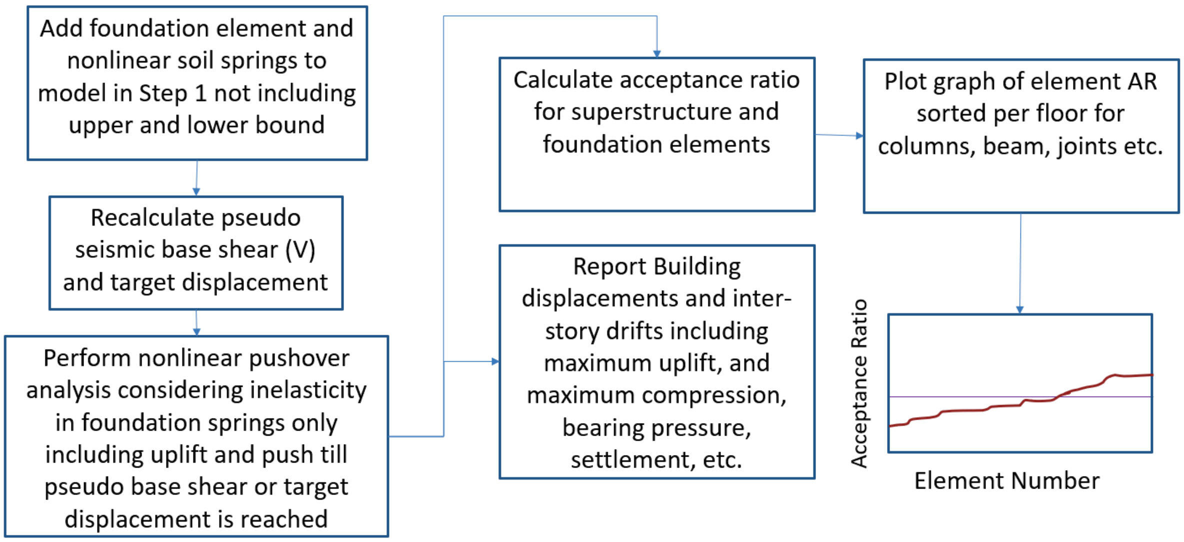


PARAMETRIC STUDY - STEP 2

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



Add foundation element and nonlinear soil springs to model in Step 1 not including upper and lower bound

Recalculate pseudo seismic base shear (V) and target displacement

Perform nonlinear pushover analysis considering inelasticity in foundation springs only including uplift and push till pseudo base shear or target displacement is reached

Calculate acceptance ratio for superstructure and foundation elements

Report Building displacements and inter-story drifts including maximum uplift, and maximum compression, bearing pressure, settlement, etc.

Plot graph of element AR sorted per floor for columns, beam, joints etc.

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

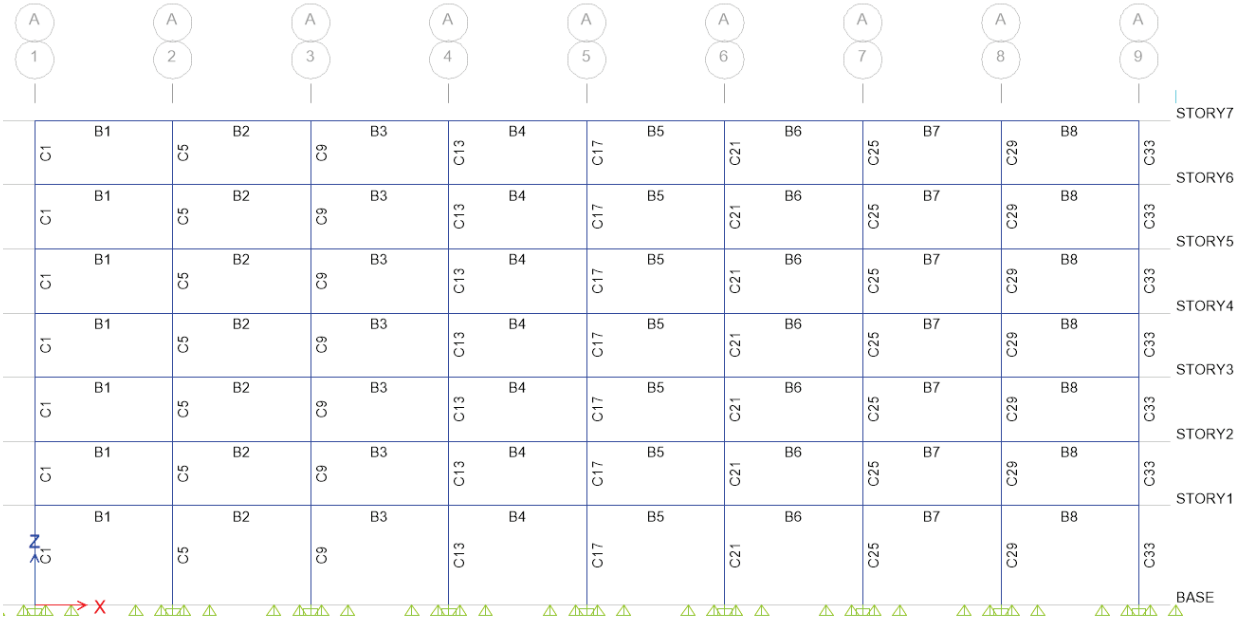


Figure C-2 Key plan of column and beam IDs along grid line A.

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.

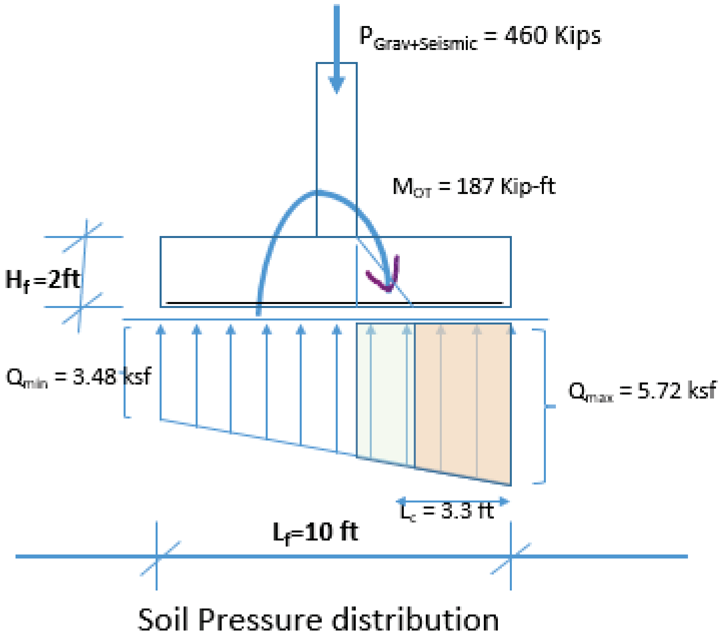


Figure C-3 **Soil pressure distribution under the footing for the governing load combination**

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

$$M_u = (4.73\text{ ksf})(10\text{ ft})\{5\text{ ft} - (14\text{ in})/(2)/(12)\}^2/2 + (5.72\text{ ksf} - 4.73\text{ ksf})(10\text{ ft})\{5\text{ ft} - (14\text{ in})/(2)/(12)\}^2/3$$

$$M_u = 526\text{ kip-ft}$$

Use 10 #9 bars, Moment Capacity $\phi M_n = 0.9(10\text{ in}^2)(60\text{ ksi})(20\text{ in} - (1.47\text{ in})/2)/(12\text{ in}) = 866\text{ kip-ft}$

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 = $\phi V_c = (0.85)(4)\sqrt{(f'_c)}\, b_o\, d$ = 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)

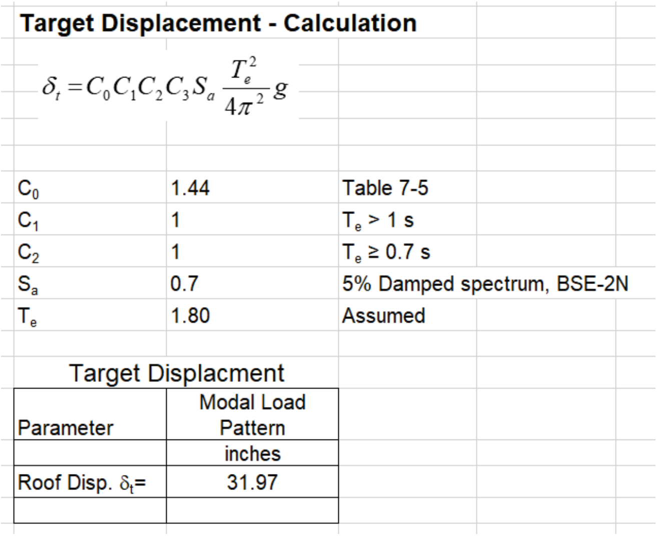
Seismic Hazard Level	Model A Fixed Base	Model B Flexible base k_{sv} = 0.1 kci	Model C Flexible base (LB) k_{sv} = 0.05 kci	Model D Flexible base (UB) k_{sv} = 0.2 kci
BSE-1N	5348	5177	5056	5248
BSE-2N	8022	7765	7583	7872

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

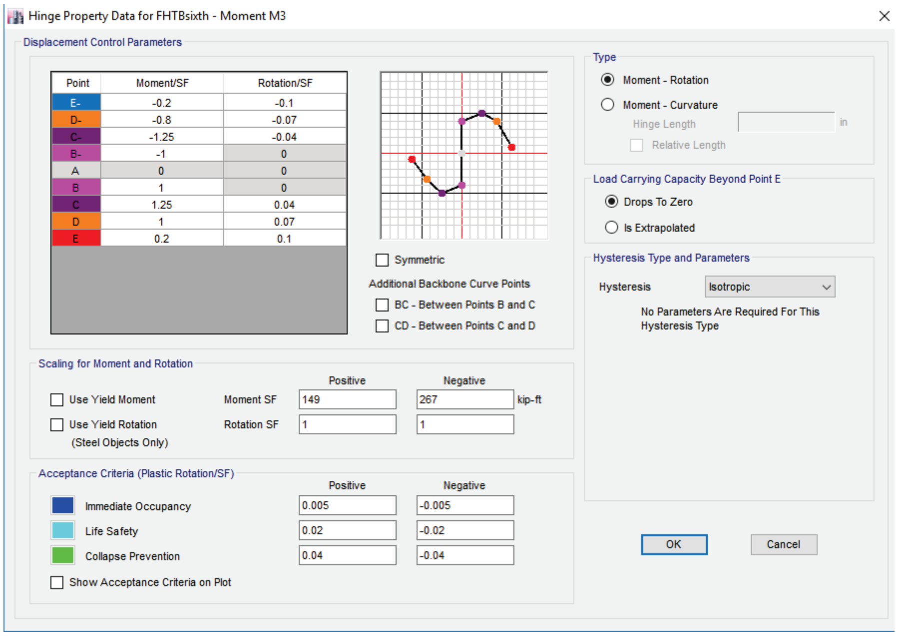
HINGE PROPERTIES AND ASSIGNMENTS

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.



Point	Moment/SF	Rotation/SF
E-	-0.2	-0.1
D-	-0.8	-0.07
C-	-1.25	-0.04
B-	-1	0
A	0	0
B	1	0
C	1.25	0.04
D	1	0.07
E	0.2	0.1

☐ Symmetric

Additional Backbone Curve Points
☐ BC - Between Points B and C
☐ CD - Between Points C and D

		Positive	Negative	
☐ Use Yield Moment	Moment SF	149	267	kip-ft
☐ Use Yield Rotation (Steel Objects Only)	Rotation SF	1	1	

	Positive	Negative
Immediate Occupancy	0.005	-0.005
Life Safety	0.02	-0.02
Collapse Prevention	0.04	-0.04

☐ Show Acceptance Criteria on Plot

● Moment - Rotation
○ Moment - Curvature
Hinge Length in
☐ Relative Length

● Drops To Zero
○ Is Extrapolated

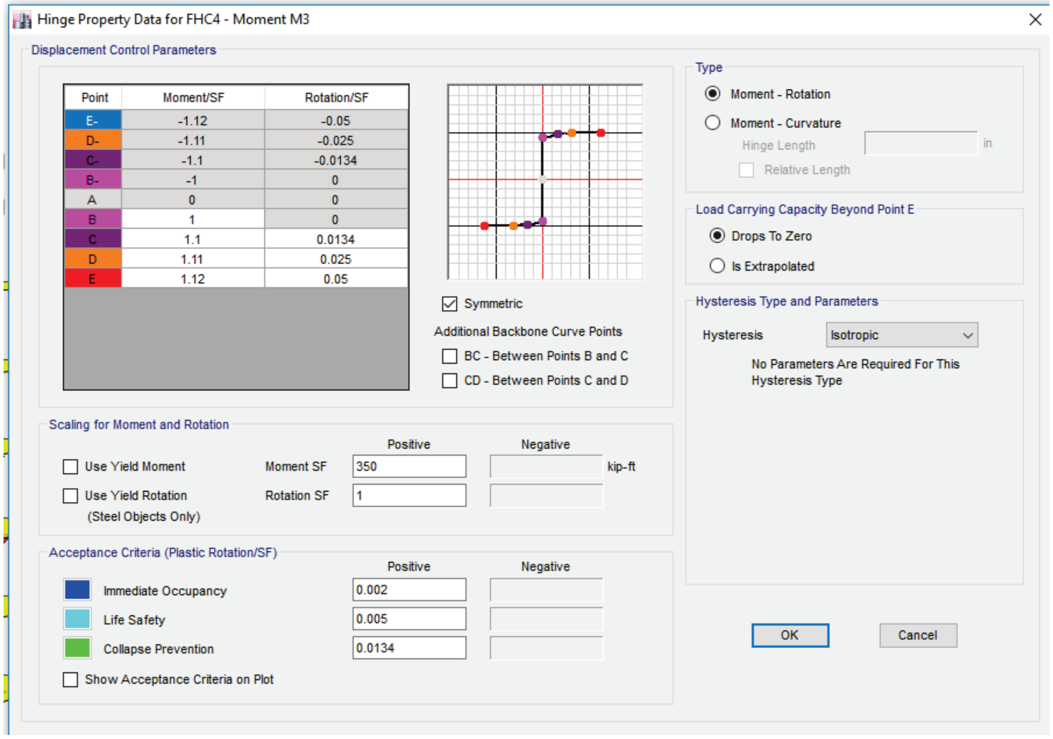
Hysteresis: Isotropic

No Parameters Are Required For This Hysteresis Type

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.



Hinge Property Data for FHC4 - Moment M3

Displacement Control Parameters

Point	Moment/SF	Rotation/SF
E-	-1.12	-0.05
D-	-1.11	-0.025
C-	-1.1	-0.0134
B-	-1	0
A	0	0
B	1	0
C	1.1	0.0134
D	1.11	0.025
E	1.12	0.05

☑ Symmetric

Additional Backbone Curve Points

☐ BC - Between Points B and C

☐ CD - Between Points C and D

Type

● Moment - Rotation

○ Moment - Curvature

Hinge Length: in

☐ Relative Length

Load Carrying Capacity Beyond Point E

● Drops To Zero

○ Is Extrapolated

Hysteresis Type and Parameters

Hysteresis: Isotropic

No Parameters Are Required For This Hysteresis Type

Scaling for Moment and Rotation

		Positive	Negative	
☐ Use Yield Moment	Moment SF	350		kip-ft
☐ Use Yield Rotation (Steel Objects Only)	Rotation SF	1		

Acceptance Criteria (Plastic Rotation/SF)

	Positive	Negative
Immediate Occupancy	0.002	
Life Safety	0.005	
Collapse Prevention	0.0134	

☐ Show Acceptance Criteria on Plot

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.

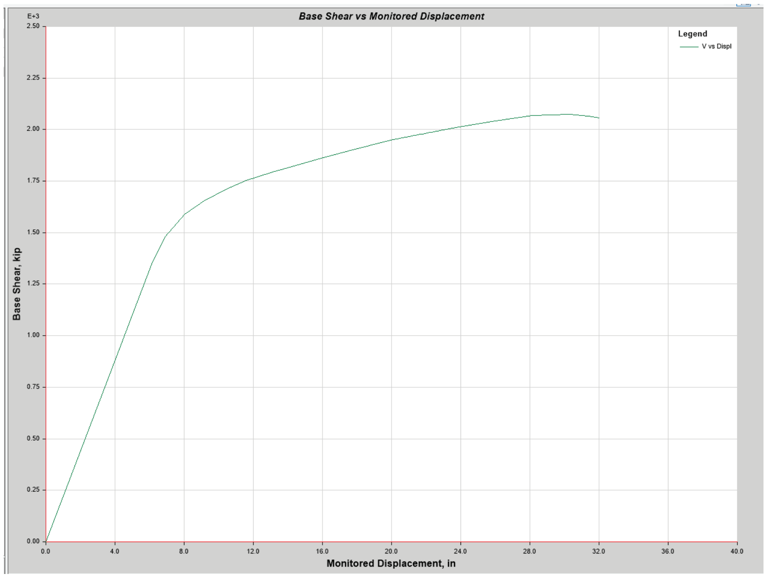
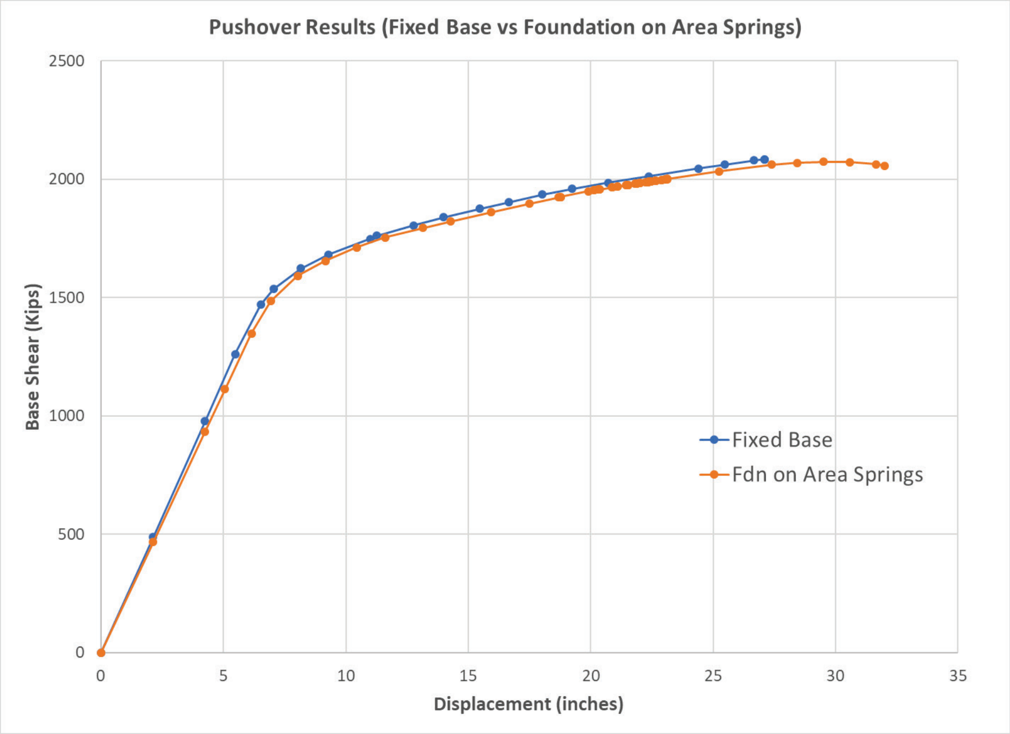


Figure C-7 Static pushover curve to the target displacement– BSE-2N of 32 inches



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.

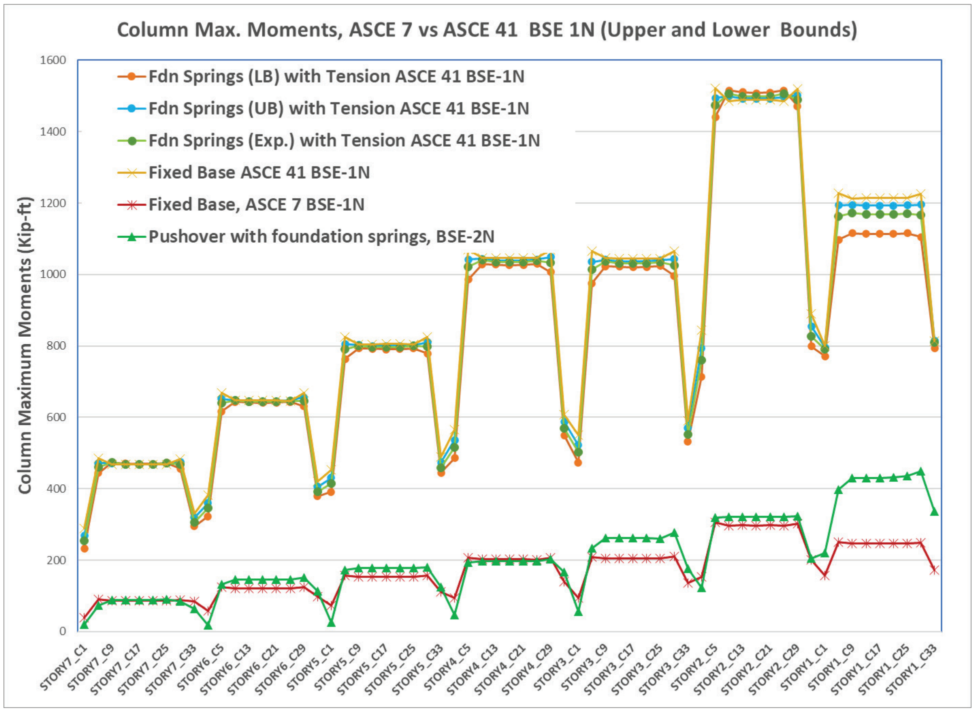


Figure C-9 Column moments per story from left to right, starting with top story on the left to bottom story on the right.

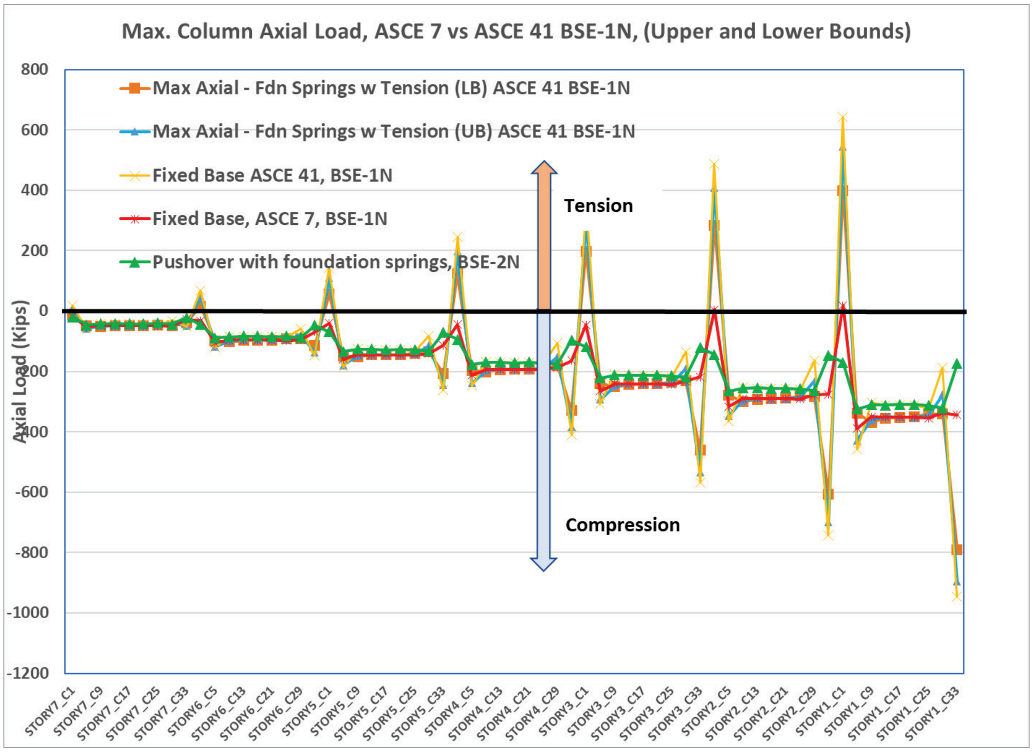


Figure C-10 Column axial load per story from left to right, starting with top story on the left to bottom story on the right.

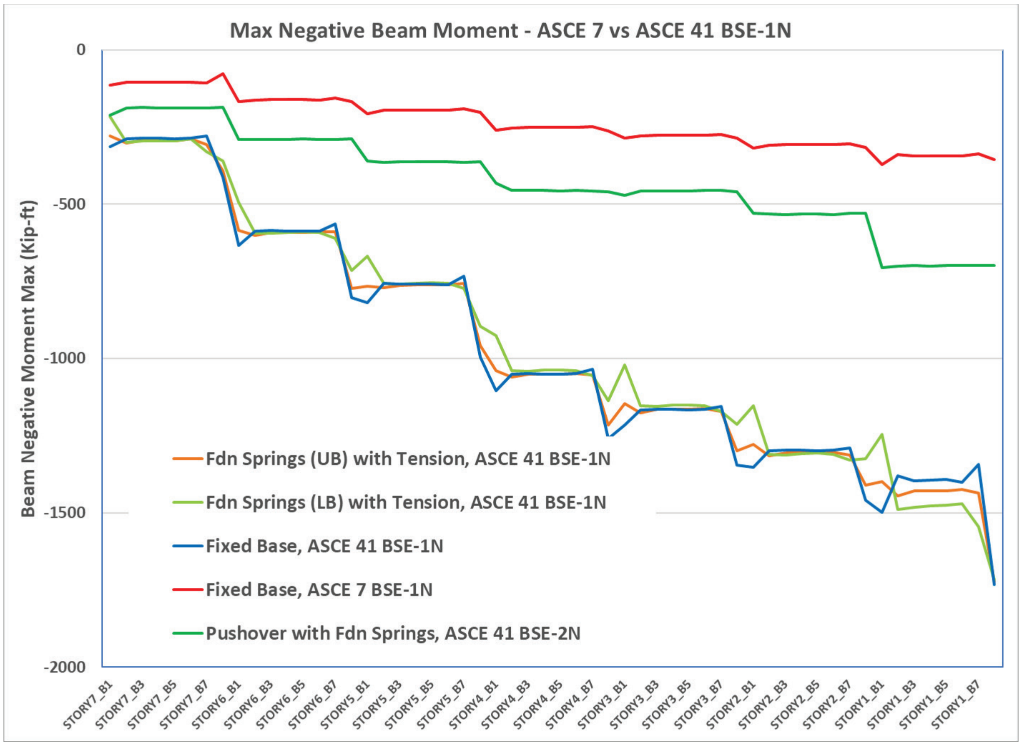


Figure C-11 Soil pressure distribution under the footing is a rectangle and a triangle.

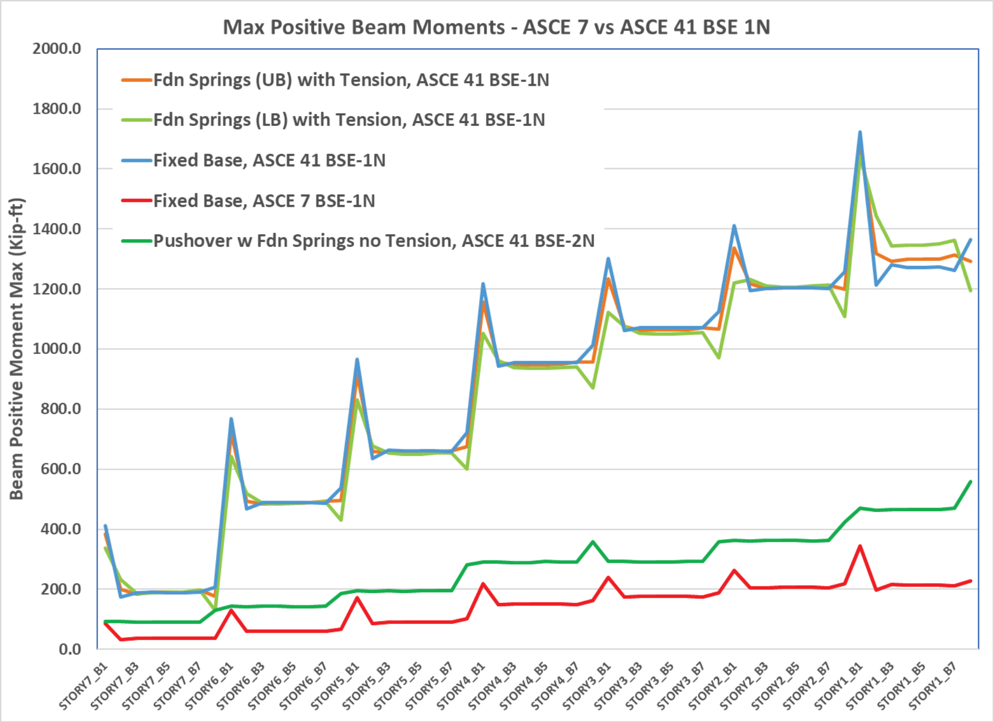


Figure C-12 Soil pressure distribution under the footing is a rectangle and a triangle.

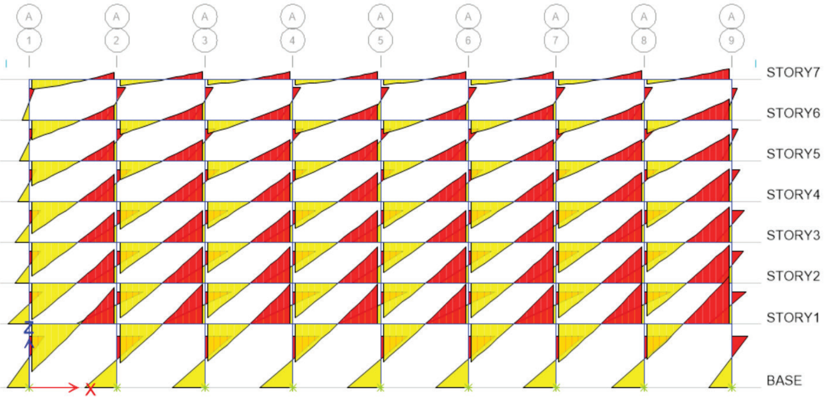
Observations and Conclusions:

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.

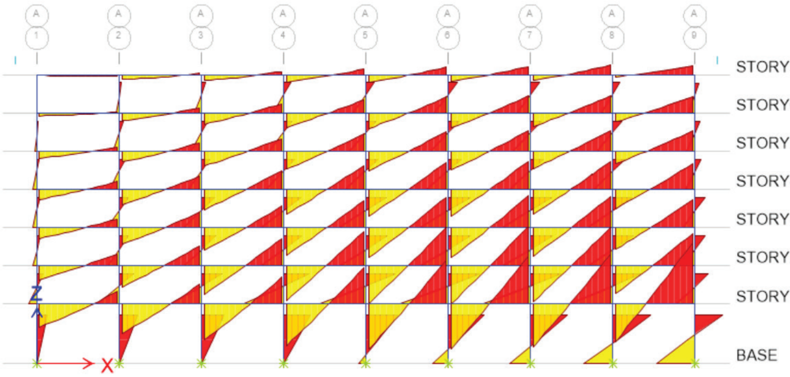
SUPERSTRUCTURE DEMAND COMPARISONS BETWEEN THE LSP AND NSP

Beams Moments

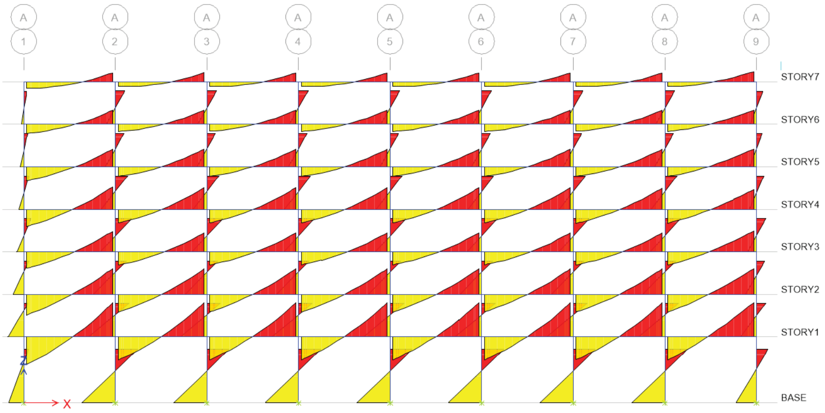
The superstructure moment pattern for X- direction loading for the tension load commination $0.9D + Q_E$, where soil takes tension and where soil does not resist tension, using LSP are compared with the moment pattern from the nonlinear static push case at the BSE-2N earthquake hazard level as shown 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.

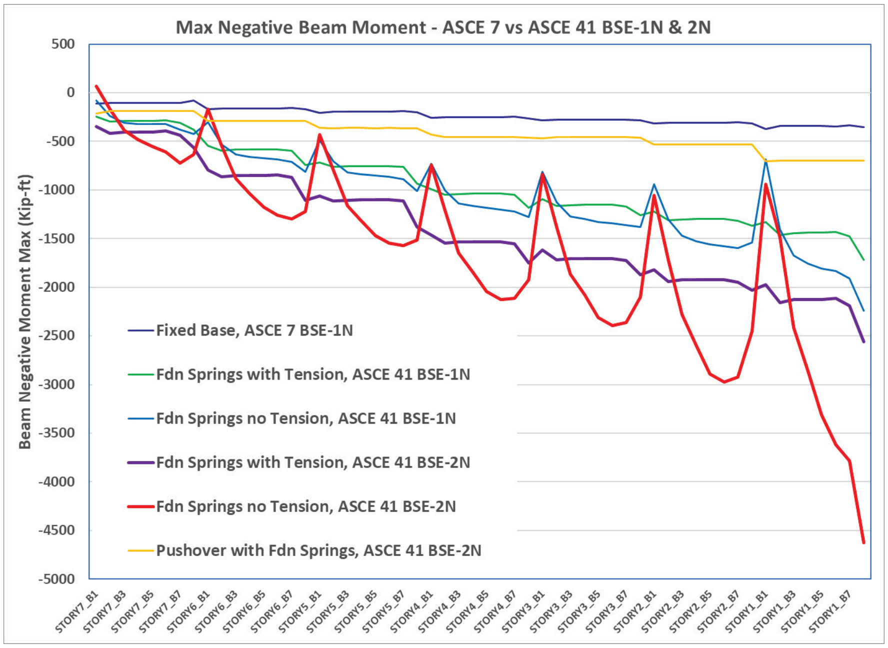


Figure C-14 Beam negative moments

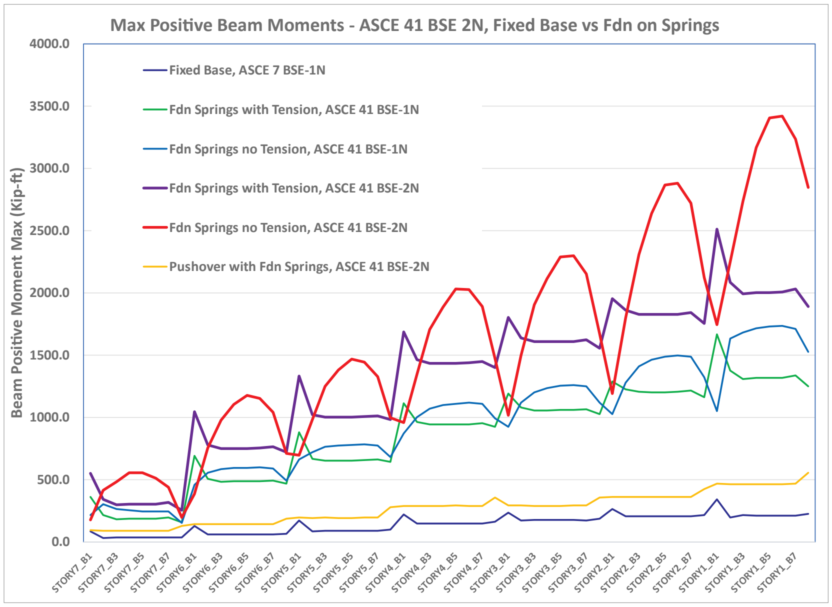


Figure C-15 Beam positive moments

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,

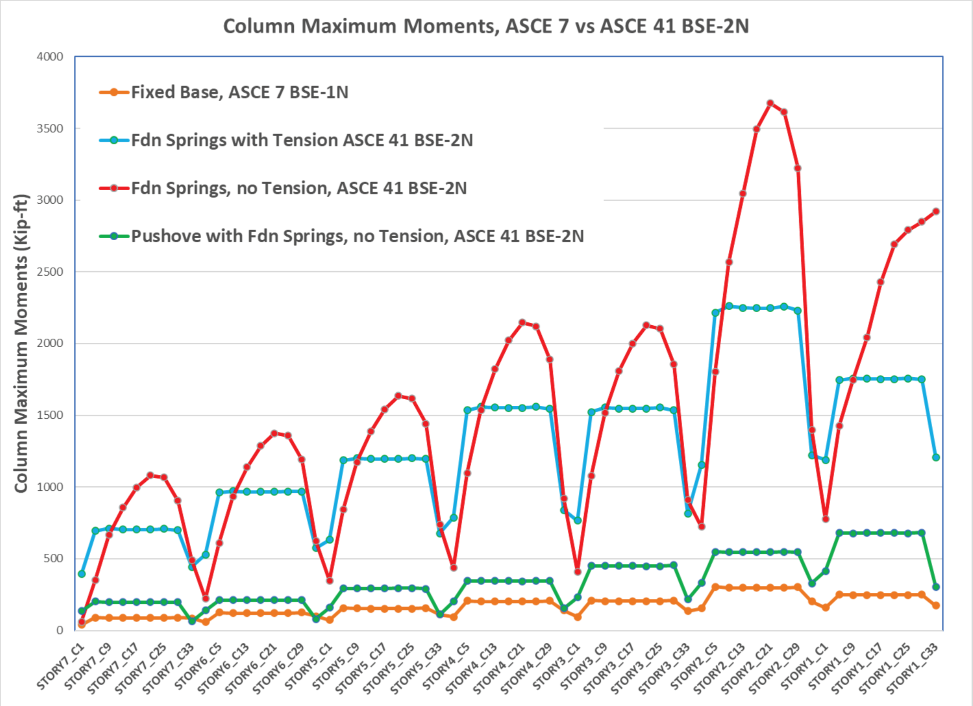


Figure C-16 Column moments

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,

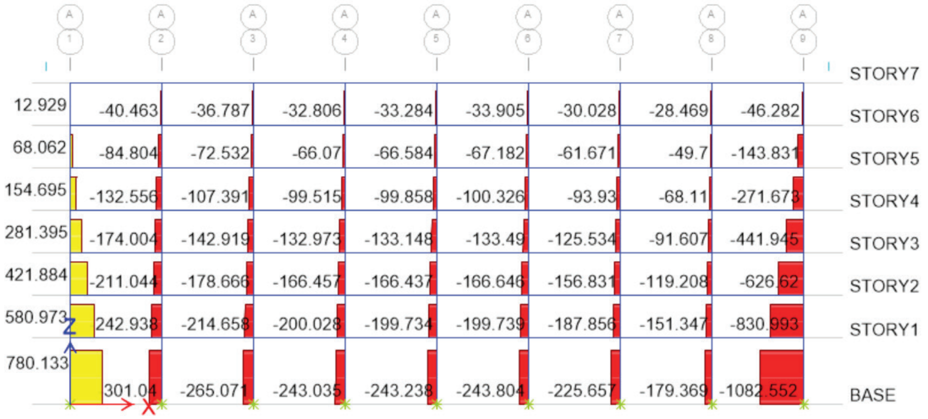


Figure C-19 **Column Axial Load, LC – 0.9D + E (BSE-2N), Soil resists Tension, k_{sv} = 0.1 kci,**

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.

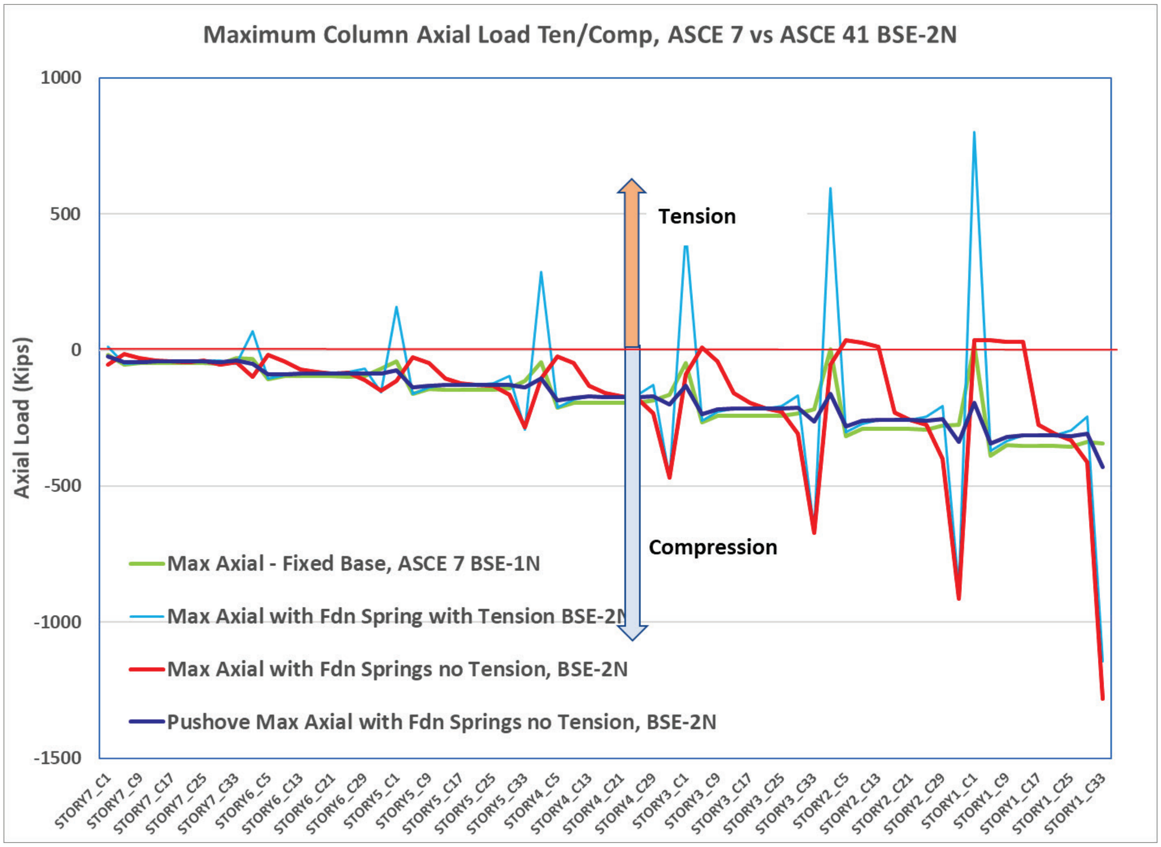


Figure C-22 Column axial loads

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)

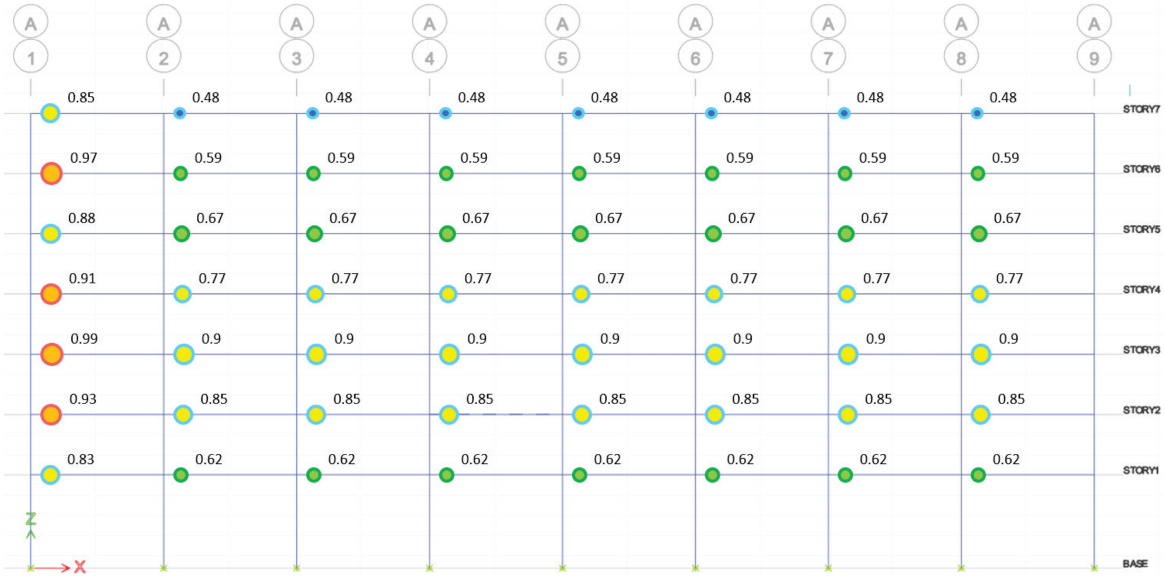


Figure C-27 ARs Beam Positive Moment ASCE 7: BSE-1N – Baseline

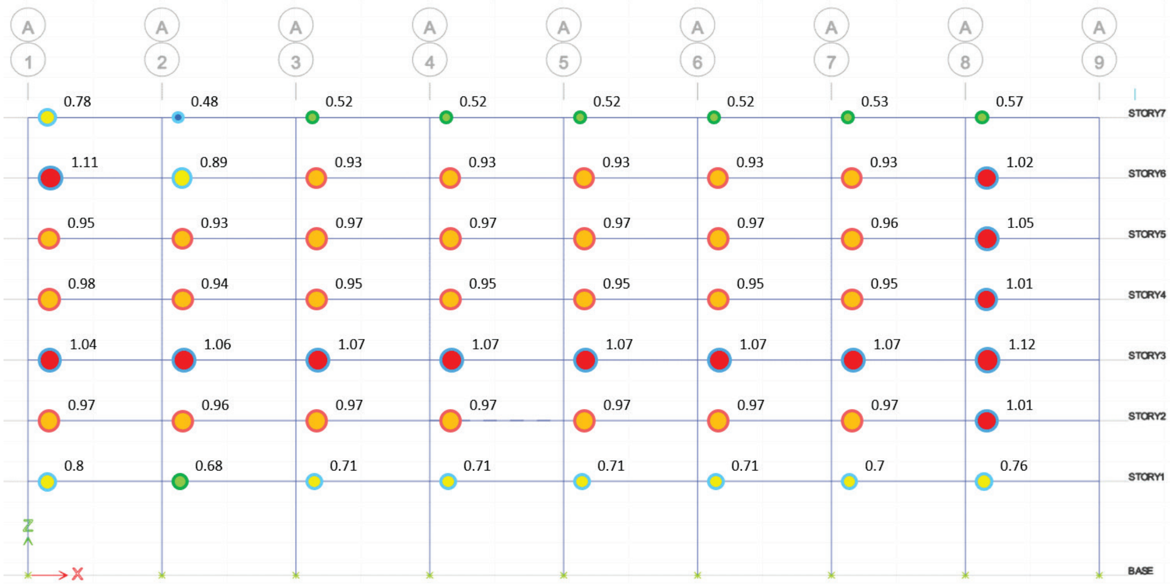


Figure C-28 ARs Beam Positive Moment ASCE 41: BSE-2N – Fixed-Base (CP)

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)

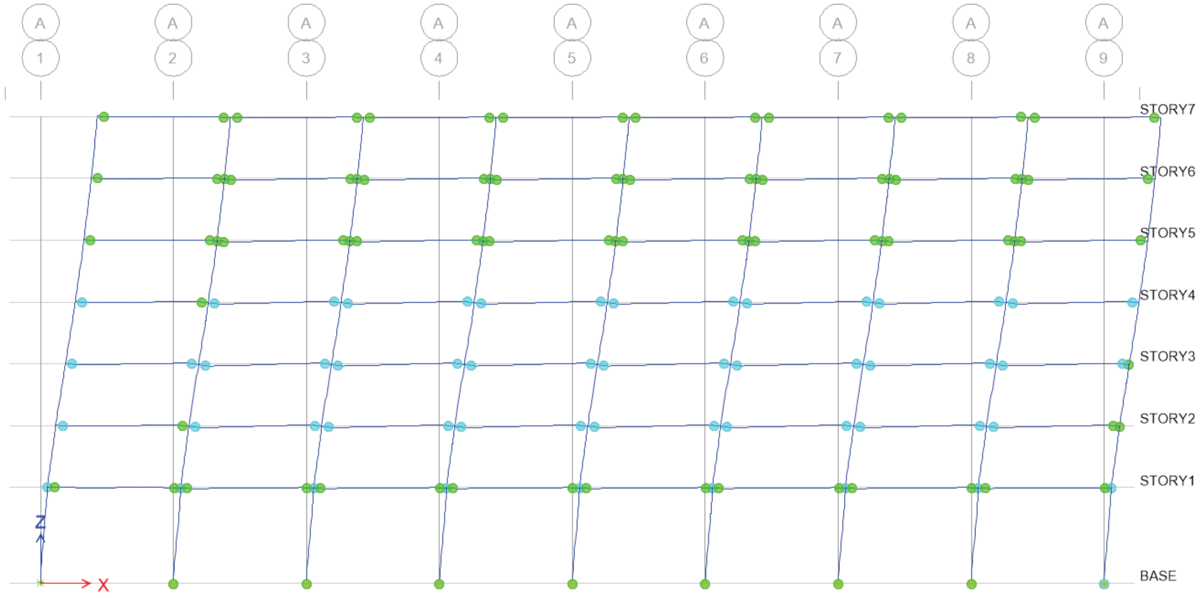


Figure C-31 Hinge pattern at Target Displacement – BSE-2N of 32 inches

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.

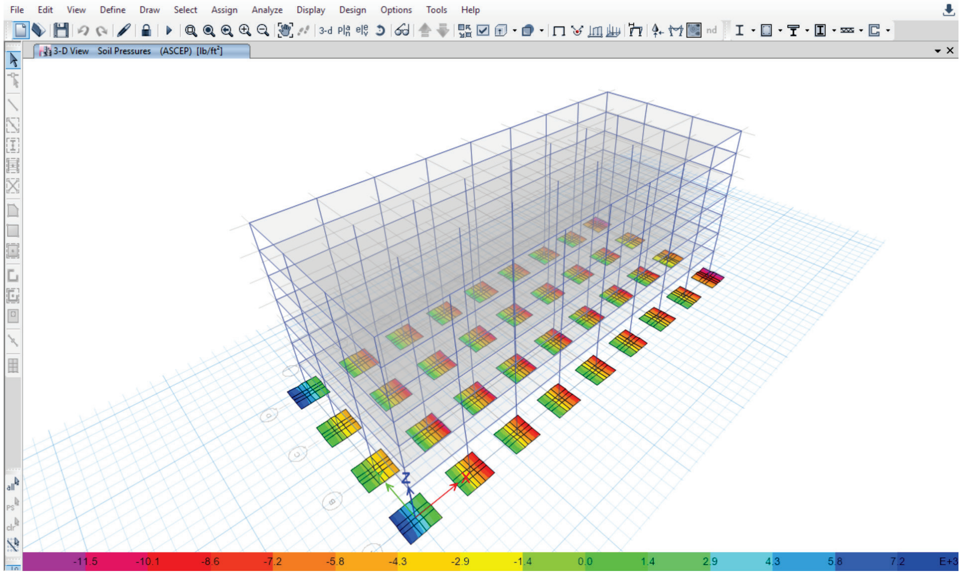


Figure C-32 Soil takes Tension Eq Hazard level BSE-1N Max pressure = 12.6 ksf

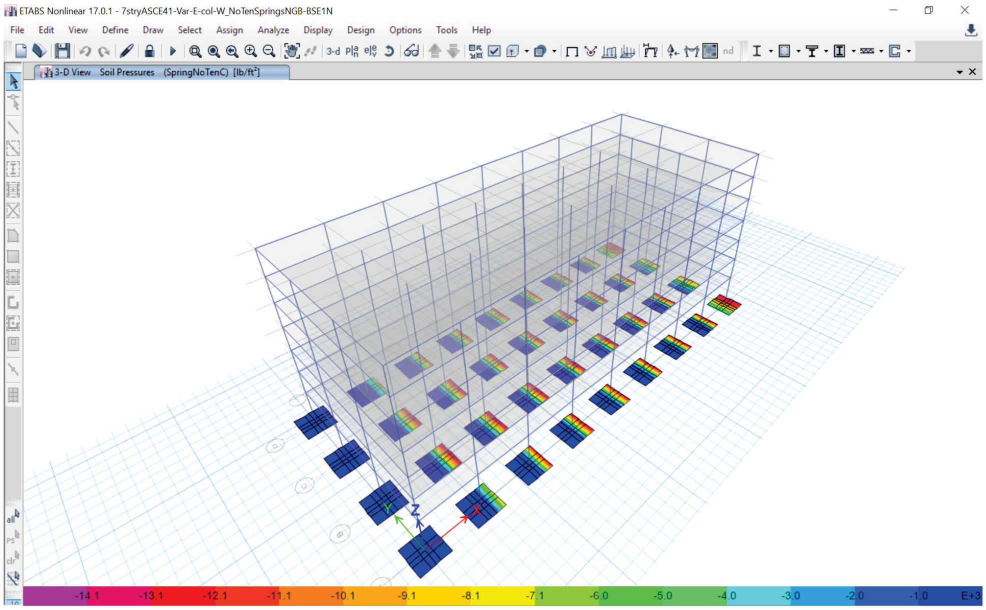
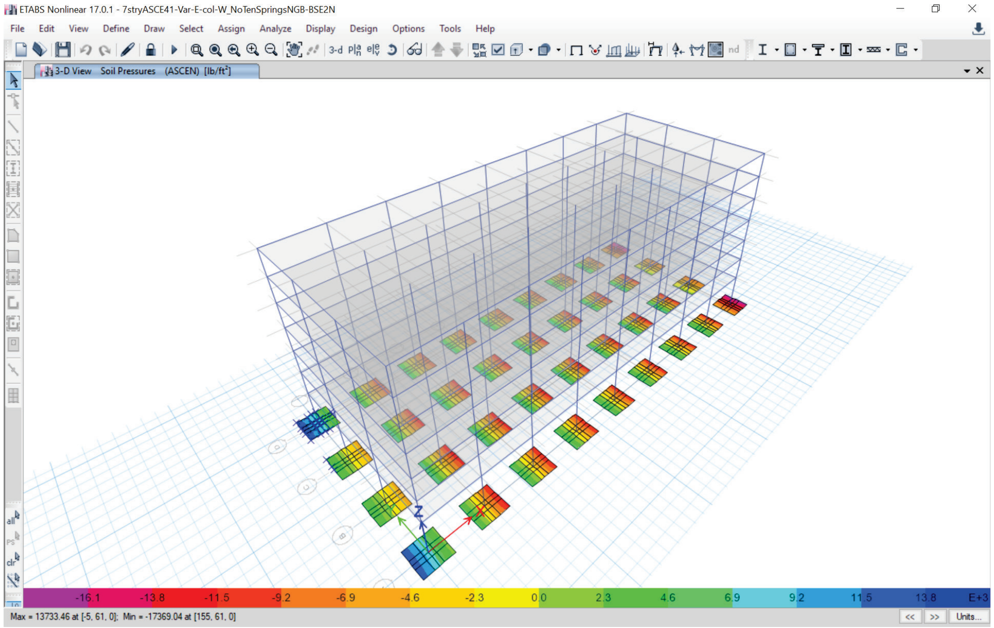


Figure C-33 Soil does not take Tension Eq Hazard Level BSE-1N Max pressure = 15.7 ksf

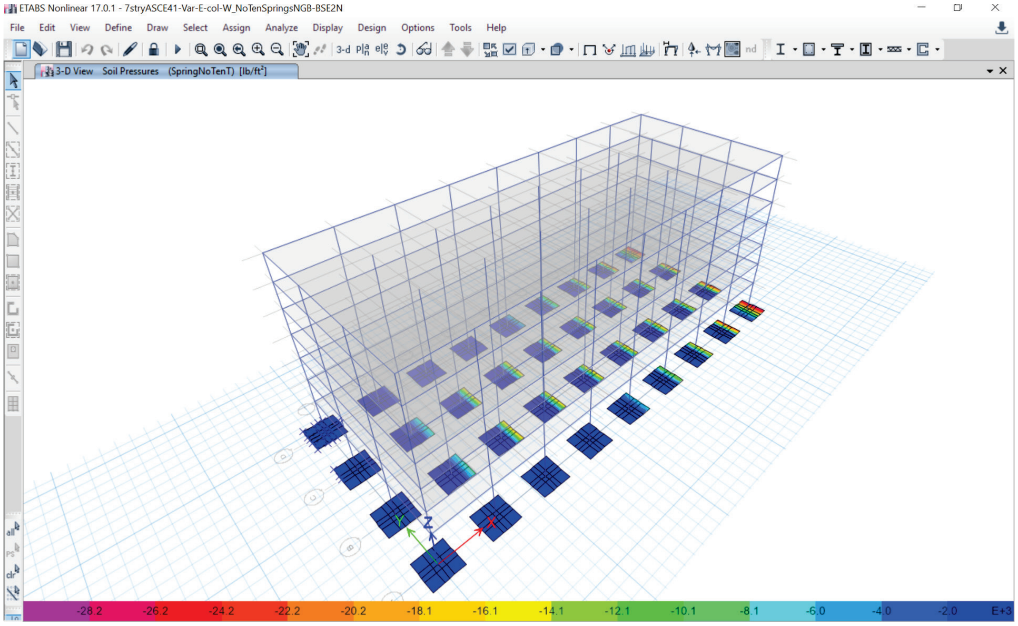


ETABS Nonlinear 17.0.1 - 7stryASCE41-Var-E-col-W_NoTenSpringsNGB-BSE2N

3-D View Soil Pressures (ASCEN) [lb/ft²]

Max = 13733.46 at [-5, 61, 0]; Min = -17369.04 at [155, 61, 0]

Figure C-34 Soil takes Tension Eq Hazard level BSE-2N Max pressure = 17.8 ksf.



ETABS Nonlinear 17.0.1 - 7stryASCE41-Var-E-col-W_NoTenSpringsNGB-BSE2N

3-D View Soil Pressures (SpringNoTenT) [lb/ft²]

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.

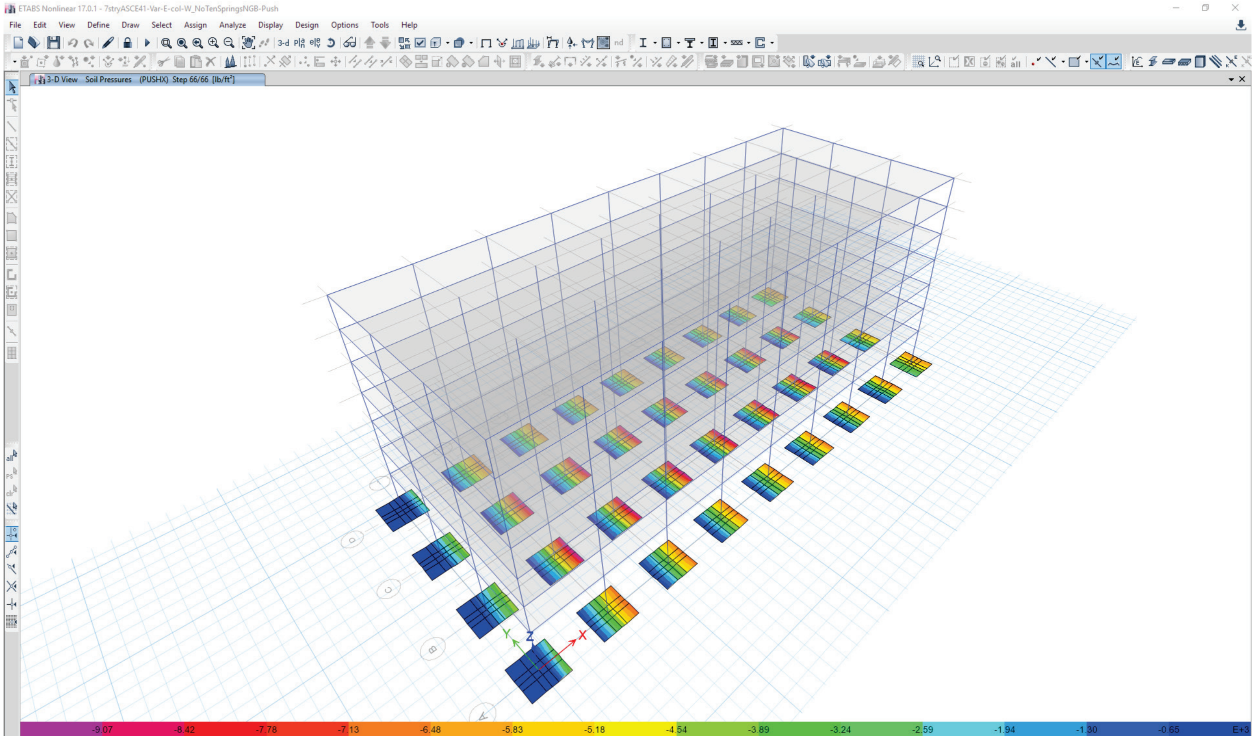


Figure C-36 Soil does not take Tension Eq Hazard Level BSE-2N NSP Max pressure = 9.24 ksf

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 = $(1 + .14S_{DS})D + 0.5L + 0.7*0.75Q_E$

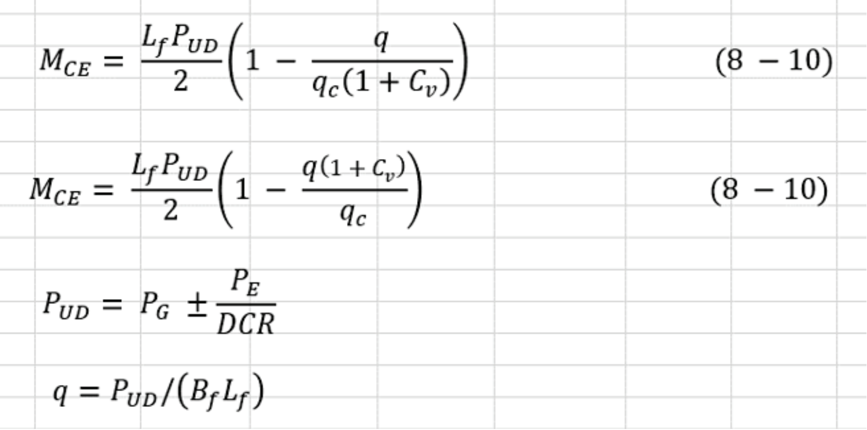
q allowable = 4.65 ksf with 1/3 increase for seismic

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.

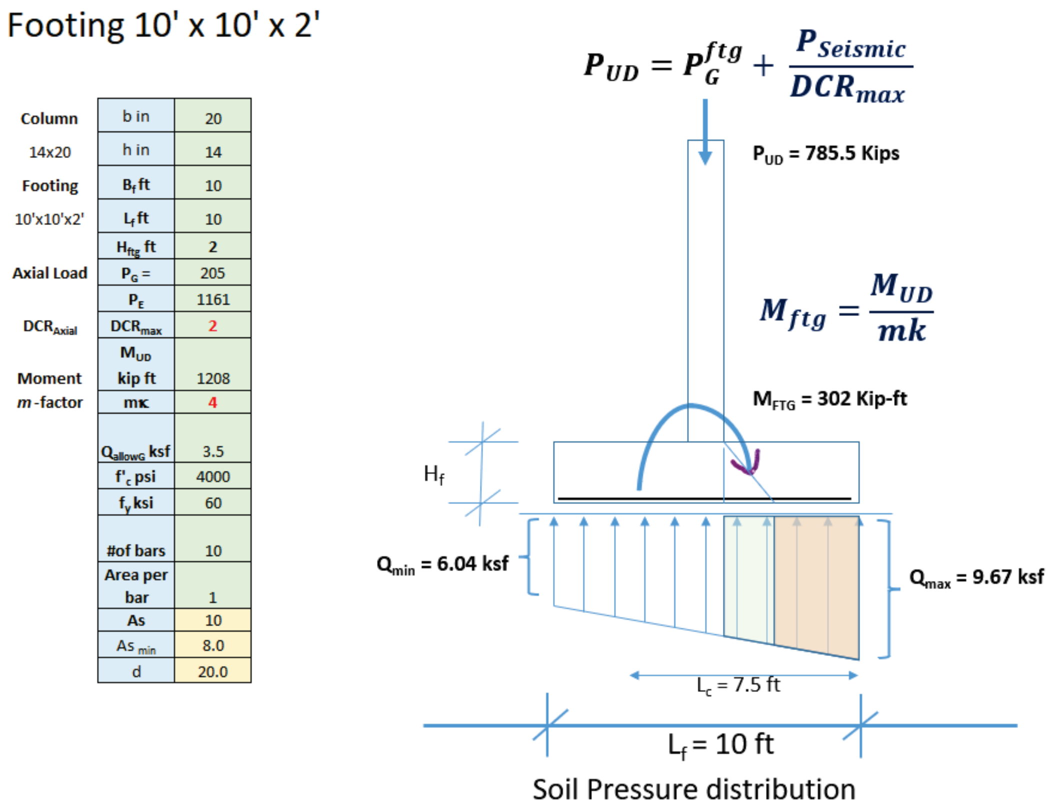
An alternate procedure is suggested in the commentary in ASCE/SEI 41-17 Section C8.4.2.3.2.1 for multiple isolated footings coupled by the superstructure above where the total area of the footing A_f is summed for all the footings, and the axial load P_{UD} is summed for all the axial loads. However, 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)$\{5 \text{ ft} - (14 \text{ in})/(2)/(12)\}^2/2$ + (9.67 ksf – 8.07 ksf)(10 ft)$\{5 \text{ ft} - (14 \text{ in})/(2)/(12)\}^2/3$

= 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

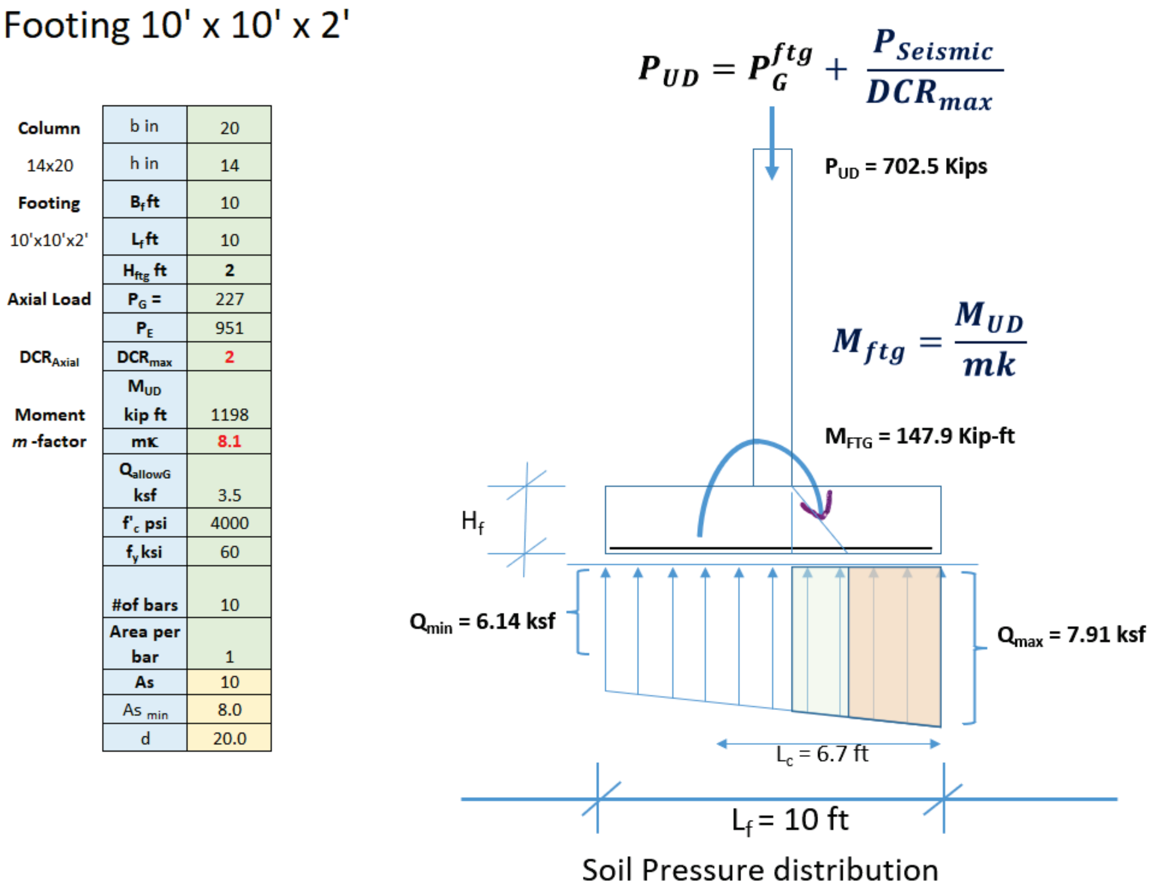
V_{UD} = (8.67 ksf)(10 ft){5 ft –(14 in)/(2)/(12) – (20 in)/12} + (9.67 ksf – 8.67 ksf)(10 ft){5 ft –(14 in)/(2)/12 – (20 in)/12}/2

=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**	**$m\kappa$**	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

$$M_{UD} = (7.13\text{ ksf})(10\text{ ft})\{5\text{ ft} - (14\text{ in})/(2)/(12)\}^2/2 + (7.91\text{ ksf} - 7.13\text{ ksf})(10\text{ ft})\{5\text{ ft} - (14\text{ in})/(2)/(12)\}^2/3$$

$$= 746\text{ kip-ft}$$

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.

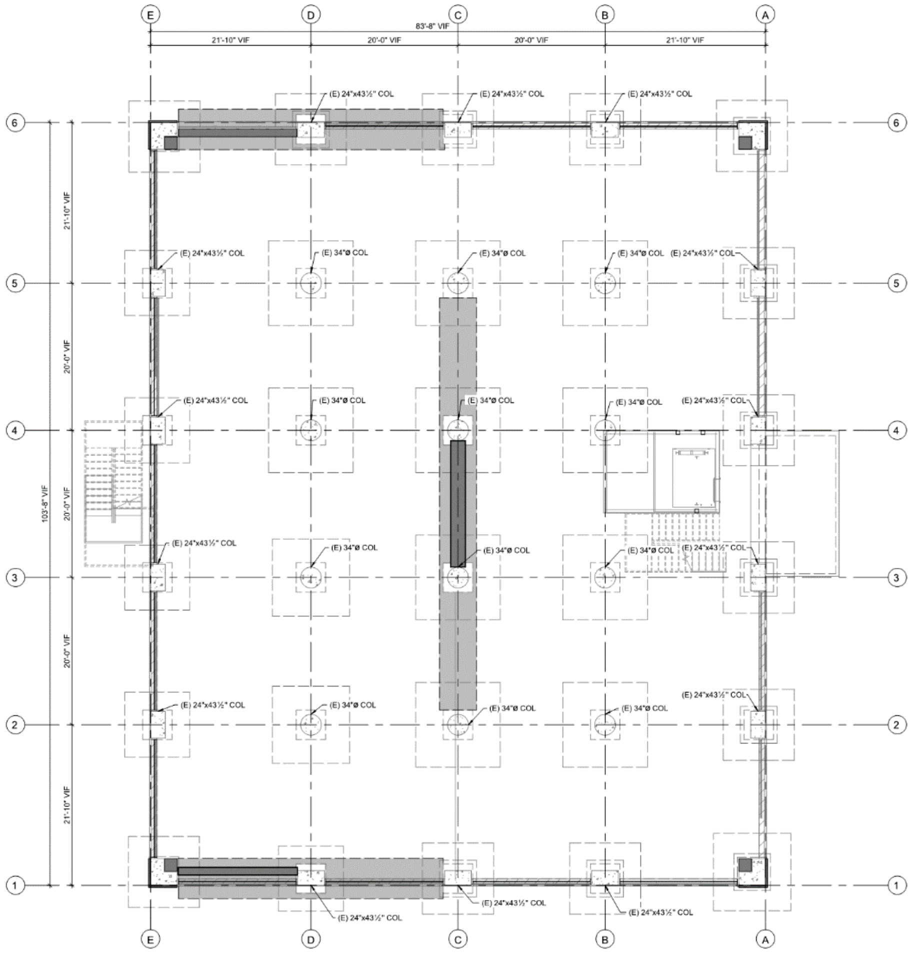


Figure C-39 Foundation Plan: With Proposed Foundation Retrofit

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.

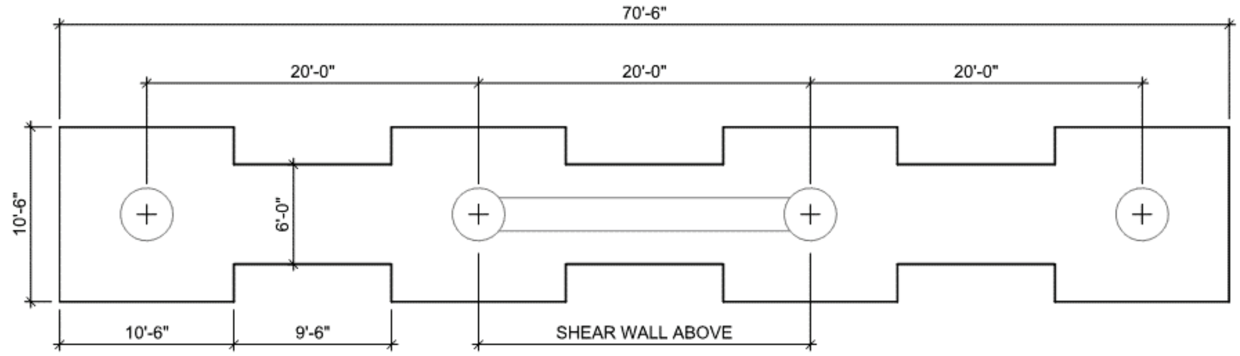


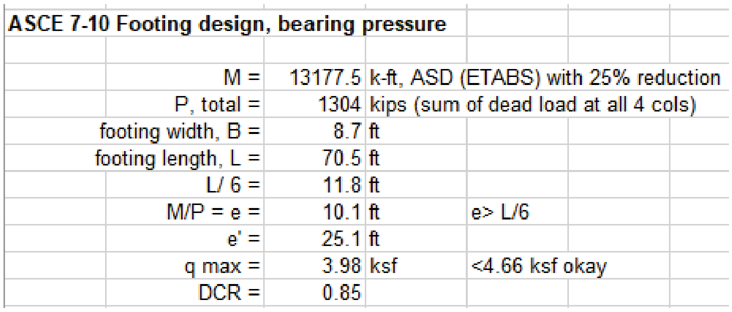
Figure C-40 Retrofit Footing Plan Layout with Dimensions

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.

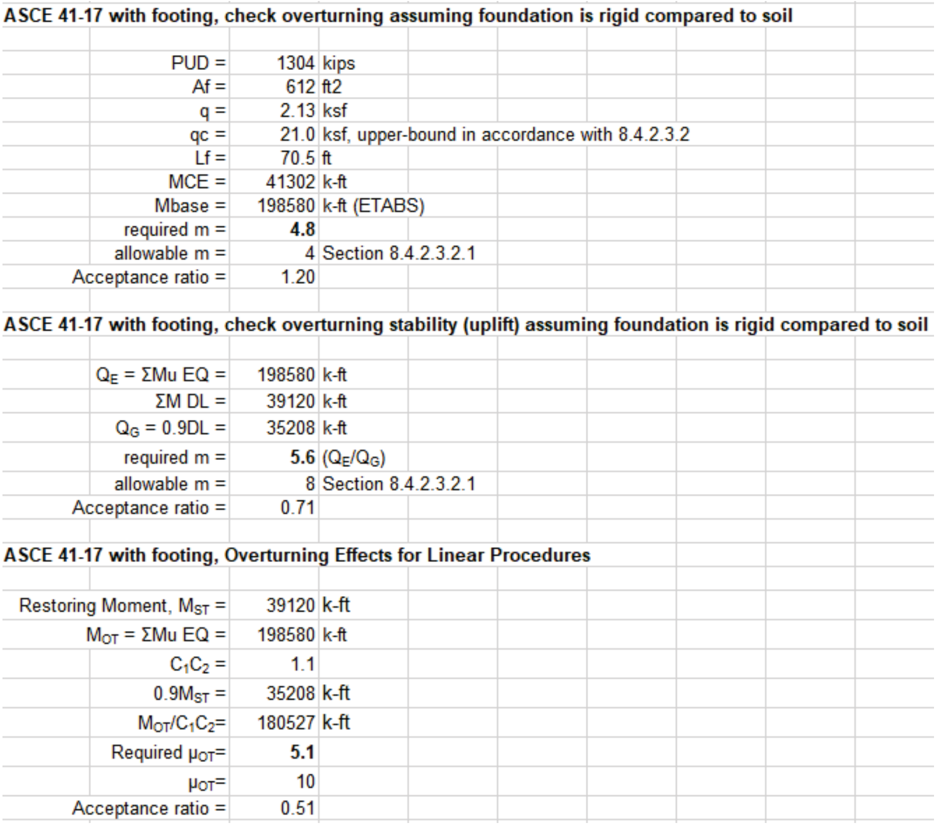


ASCE 7-10 Footing design, bearing pressure			
M =	13177.5	k-ft, ASD (ETABS) with 25% reduction	
P, total =	1304	kips (sum of dead load at all 4 cols)	
footing width, B =	8.7	ft	
footing length, L =	70.5	ft	
L/ 6 =	11.8	ft	
M/P = e =	10.1	ft	e> L/6
e' =	25.1	ft	
q max =	3.98	ksf	<4.66 ksf okay
DCR =	0.85		

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	

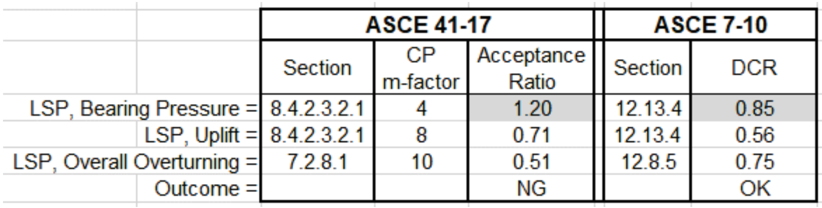
ASCE 41-17 with footing, Overturning Effects for Linear Procedures

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:

$$Acceptance\ Ratio = \frac{\boldsymbol{P_{D+L}}}{\boldsymbol{q_{allowG}A_G}} + \frac{\boldsymbol{P_{Seis}}}{\boldsymbol{mq_{ultimate}A_gDCR_{supA}}} + \frac{\frac{\boldsymbol{M_{Seis}L}}{\boldsymbol{2}}}{\boldsymbol{mq_{ultimate}I_gDCR_{supM}}} \qquad Eq. C-1$$

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:

$$Q_{max} = \frac{1}{A_g}\left(P_{D+L} + \frac{P_{Seis}}{mDCR_{supA}} + \frac{6M_{Seis}}{mDCR_{supM}L_f}\right) \qquad Eq.C-2$$

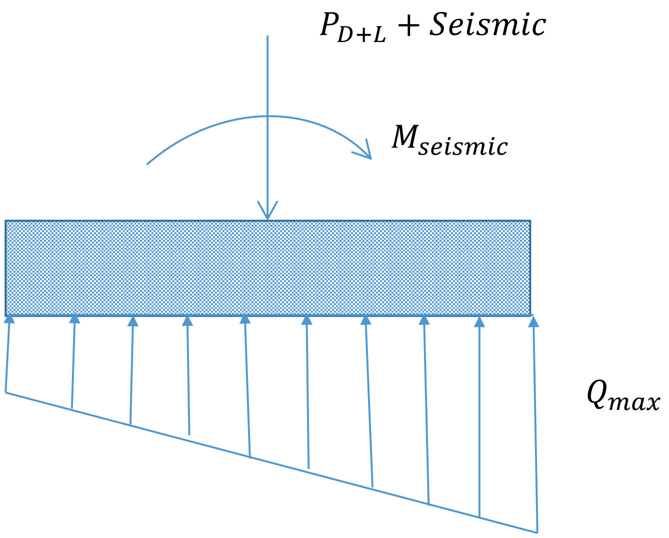


Figure C-44 Case 1, entire footing remains in contact with the soil

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.

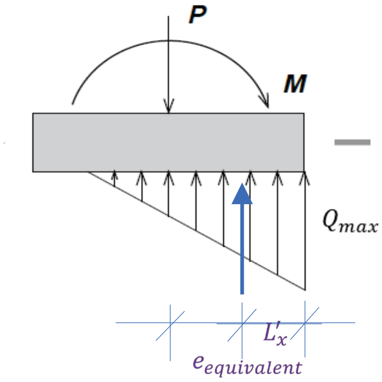


Figure C-45 Case 2, partial uplift of the footing and $Q_{max} < q_{ultimate}$.

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:

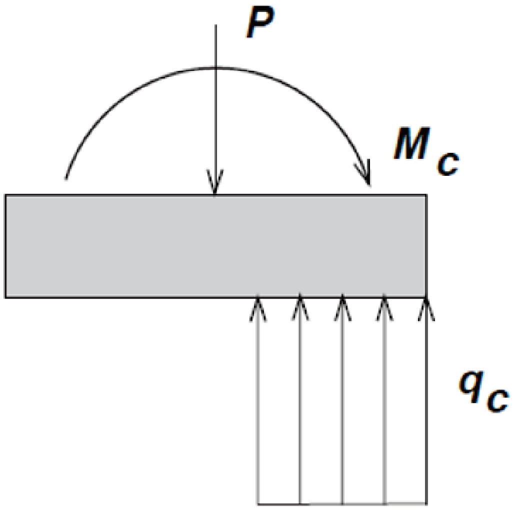


Figure C-46 Case 3, soil pressure calculated using a rectangular distribution.

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:

$$\boldsymbol{P_{equivalent} = P_{D+L} + \frac{P_{Seis}}{mDCR_{supA}};}$$

$$\boldsymbol{M_{equivalent} = \frac{M_{Seis}}{mDCR_{supM}};}$$

and

$$\boldsymbol{e_{equivalent} = \frac{M_{equvialnt}}{P_{equvialnt}};}$$

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.

Case 1: $\boldsymbol{Q_{max} = \frac{P_{equivalent}}{A_g}\left(1 + \frac{6e_{equivalent}}{L_f}\right)}$; $e \leq \boldsymbol{L_f/6}$

Case 2: $\boldsymbol{Q_{max} = \frac{2P_{equivalent}}{3L_y\left(\frac{L_f}{2} - e_{equivalent}\right)}}$ - **Triangular, e > L_f/6**

Case 3: $\boldsymbol{Q_{max} = \frac{P_{equivalent}}{2L_y\left(\frac{L_f}{2} - e_{equivalent}\right)}}$ - **Rectangular**

Moment Capacity: $\boldsymbol{M_{CE} = \frac{L_f P_{UD}}{2}\left(1 - \frac{q}{q_c}\right)}$ - **Eq 8-10**

A summary of the equations used for the proposed acceptance ratio is given below.

$$P_{equivalent} = P_{D+L} + \frac{P_{Seis}}{3mDCR_{supA}}$$

$$M_{equivalent} = \frac{M_{Seis}}{3mDCR_{supM}}$$

$$e_{equivalent\_AC} \leq \frac{L_f}{6}$$

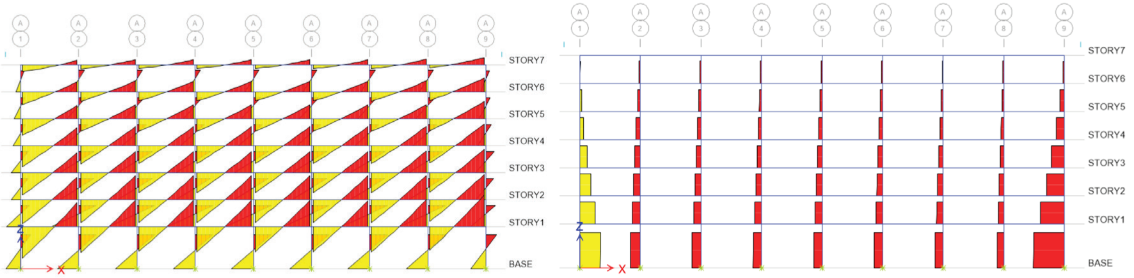
$$\text{Acceptance Ratio} = \frac{P_{equivalent\_AC}}{A_g Q_{allowG}}\left(1 + \frac{6e_{equivalent\_AC}}{L_f}\right)$$

$$e_{equivalent\_AC} > \frac{L_x}{6}$$

$$\text{Acceptance Ratio} = \frac{2P_{equivalent\_AC}}{3B_f Q_{allowG}\left(\frac{L_x}{2} - e_{equivalent\_AC}\right)}$$

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. $q_c = 3 \times q_{allow}$.



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**

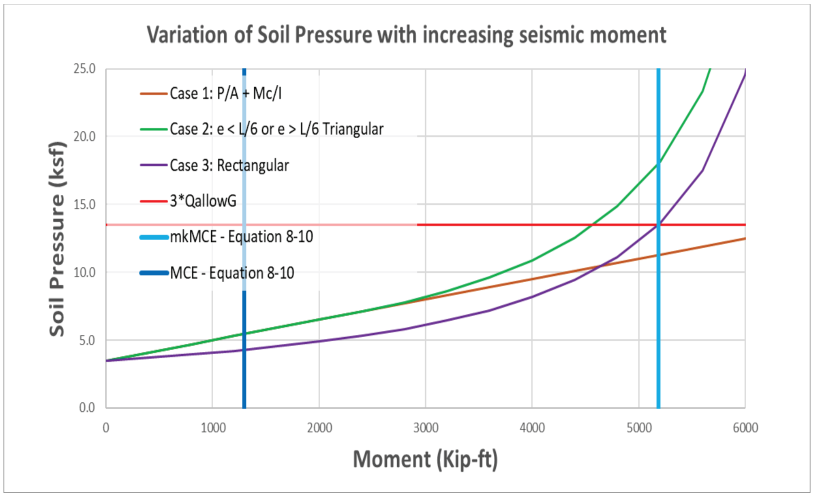


Figure C-48 Soil pressure variation with seismic overturning moment.

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**

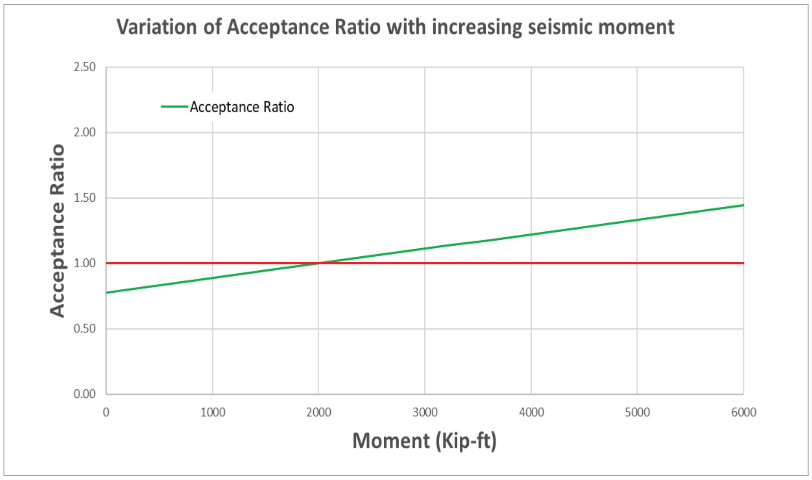


Figure C-49 Variation in acceptance ratio with increasing seismic overturning moment.

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**

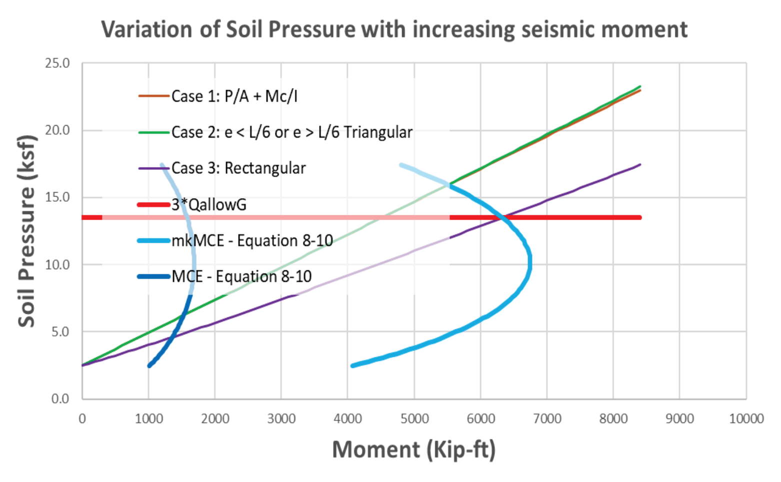


Figure C-50 Soil pressure variation with seismic overturning moment, end bay.

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**

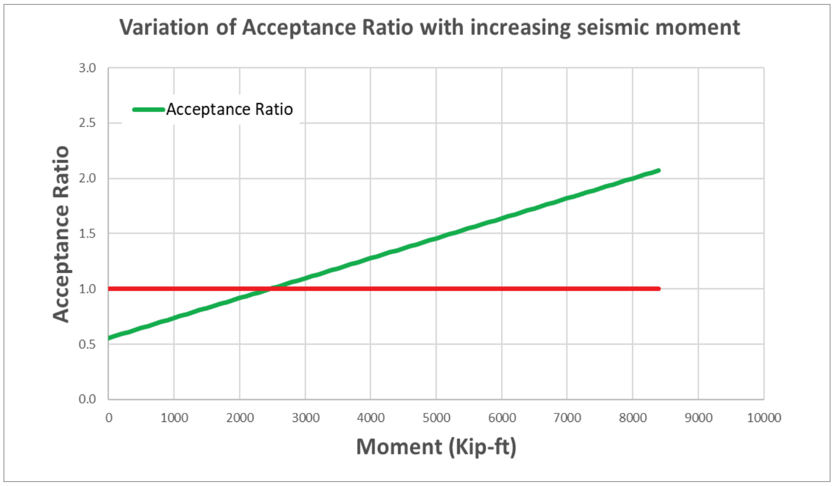


Figure C-51 Variation in acceptance ratio with increasing seismic overturning moment, end bay.

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	
$m_{Axail\text{ -}}$ Factor	4	CP
M_{seis}	0	kip-ft
M_{seism}_inc	100	kip-ft
DCR_{mom}	2	
$m_{mom\text{ -}}$ Factor	4	CP
Q_{allowG}	4.5	ksf

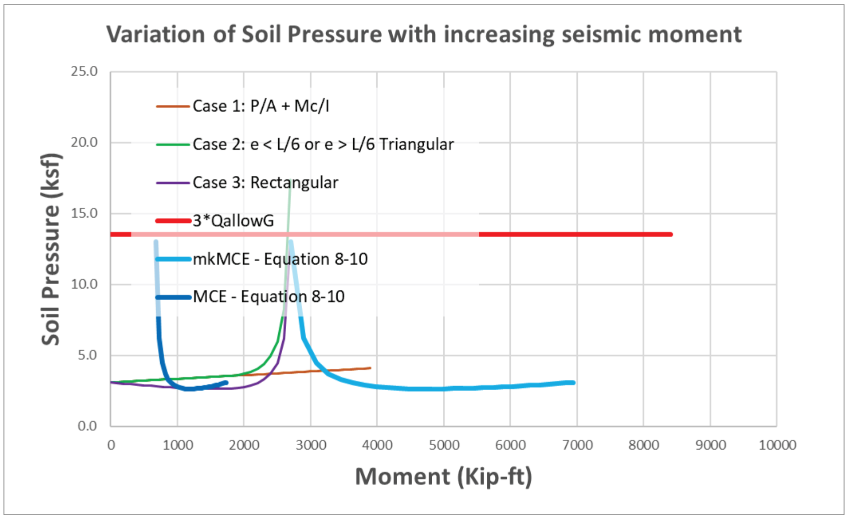


Figure C-52 **Soil pressure variation with seismic overturning moment, end bay, seismic demand subtracts from gravity.**

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	
$m_{Axail\text{ -}}$ Factor	4	CP
M_{seis}	0	kip-ft
M_{seism}_inc	100	kip-ft
DCR_{mom}	2	
$m_{mom\text{ -}}$ Factor	4	CP
Q_{allowG}	4.5	ksf

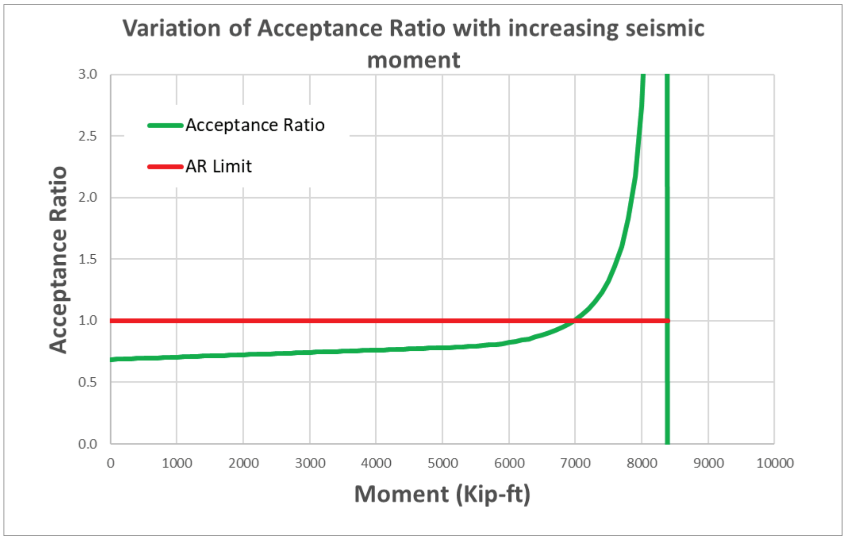


Figure C-53 **Soil pressure variation with seismic overturning moment, end bay, seismic demand subtracts from gravity.**

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.

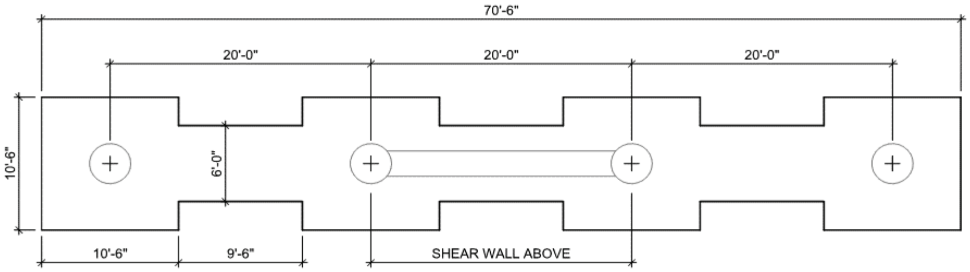


Figure C-54 Footing plan under new shear wall, Archetype 1

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**

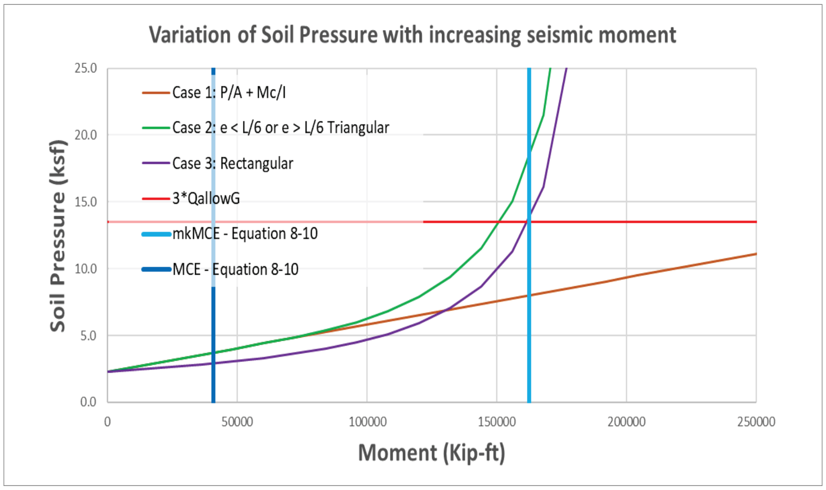


Figure C-55 Soil pressure variation with seismic overturning moment.

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**

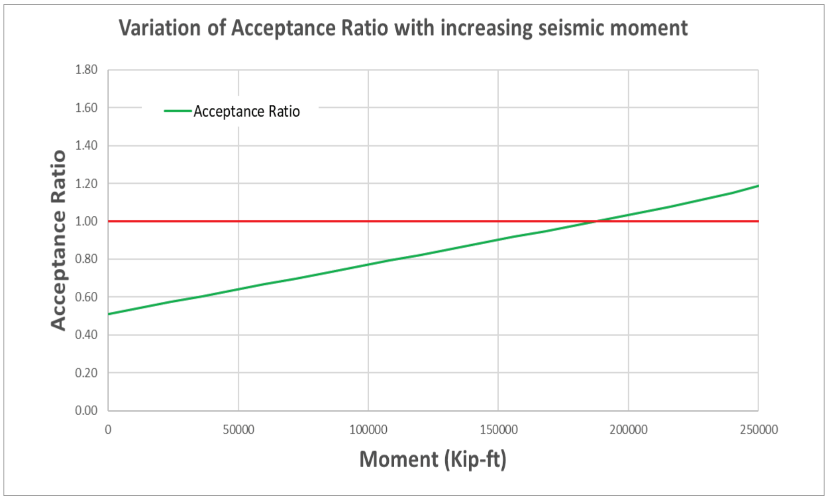


Figure C-56 Variation of acceptance ratio with seismic overturning moment.

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)

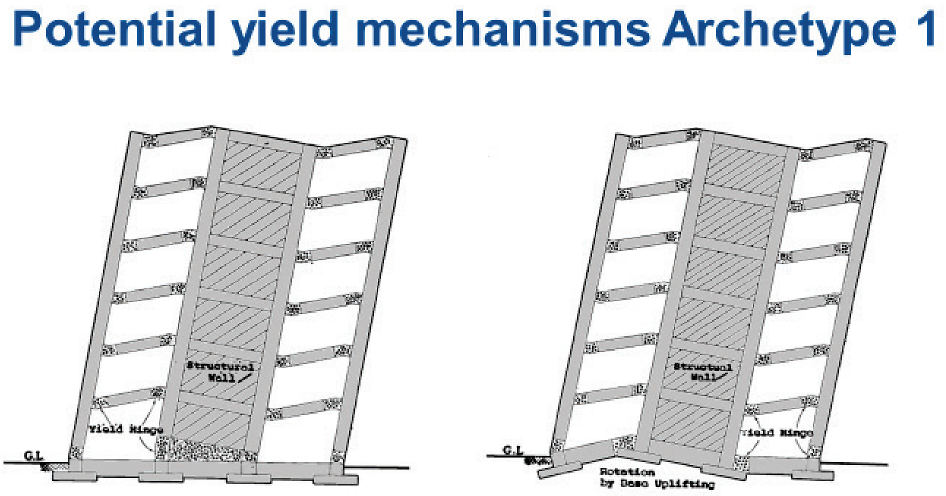


Figure C-57 Soil pressure variation with seismic overturning moment.

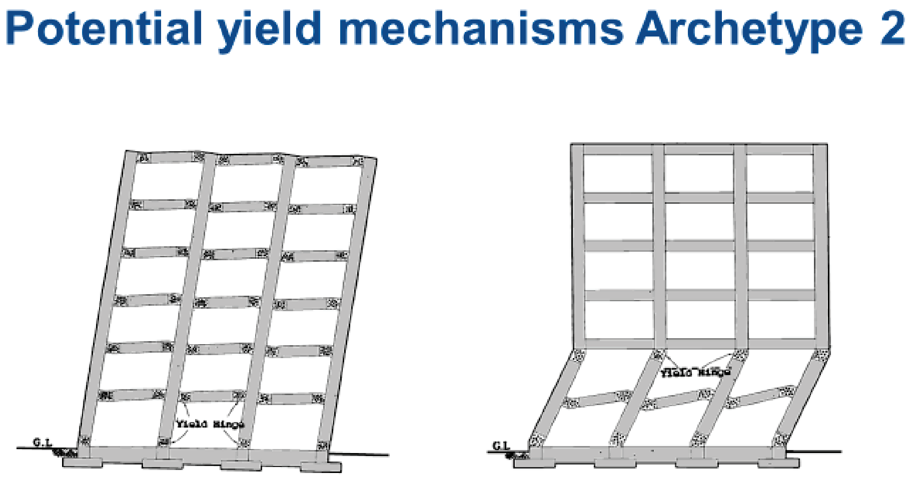


Figure C-58 Soil pressure variation with seismic overturning moment.

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.

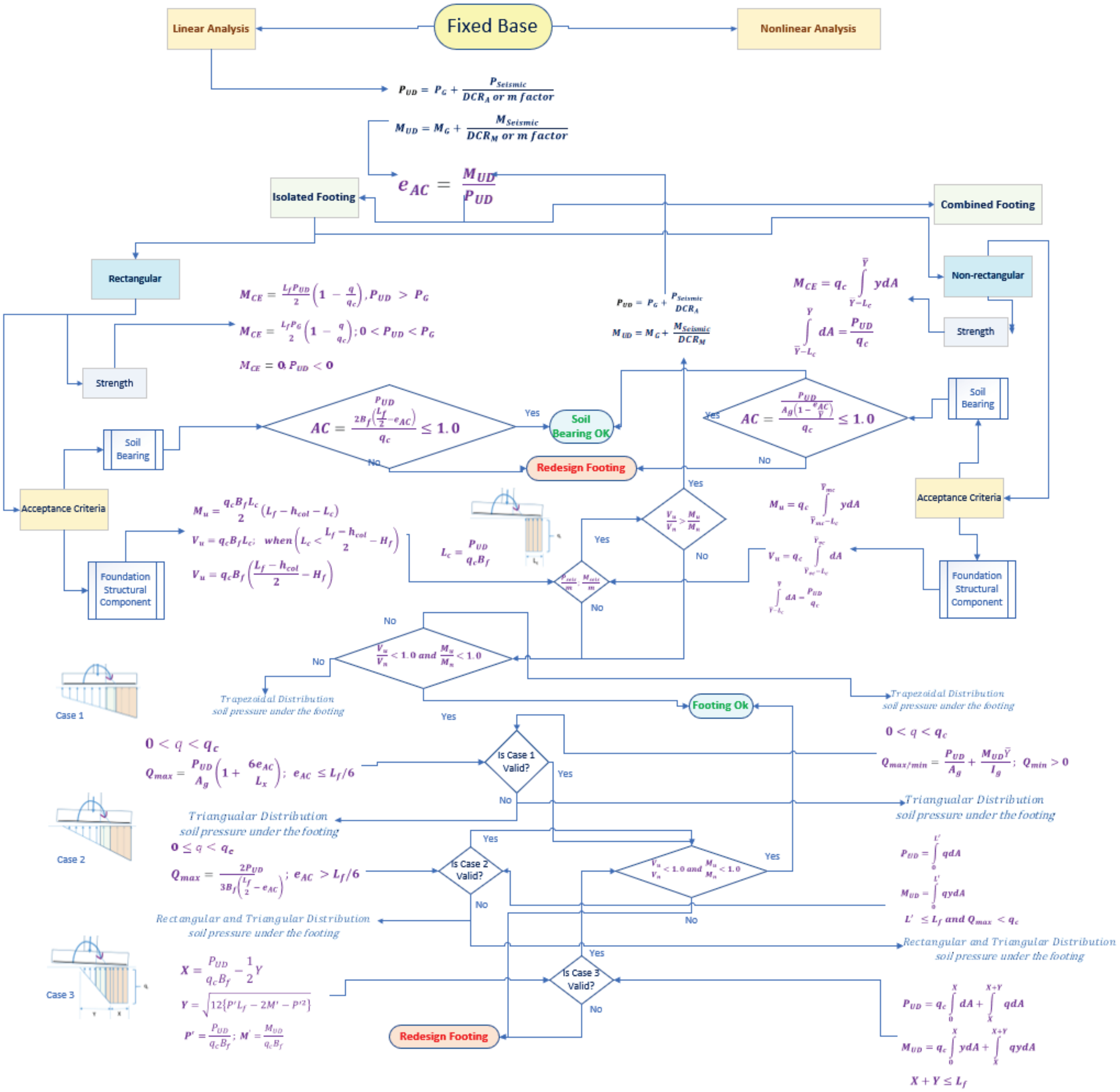


Figure C-59 **Flow chart of Proposed acceptance criteria methodology for soil bearing and the structural footing**

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

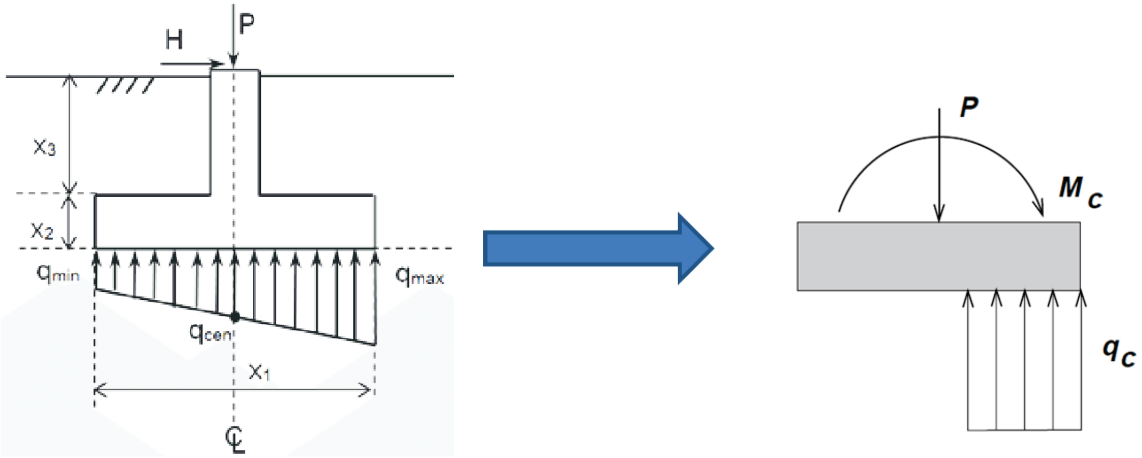


Figure C-60 **Proposed method for evaluating soil bearing and footing acceptance based on anticipated soil pressure distribution under the footing.**

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.

$$M_{CE} = \frac{L_f P_{UD}}{2}\left(1 - \frac{q}{q_c}\right) \qquad \text{ASCE/SEI Eq. (8-10)}$$

Where, $\mathrm{q} = \frac{\mathrm{P_{UD}}}{\mathrm{B_f L_f}}$ 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:

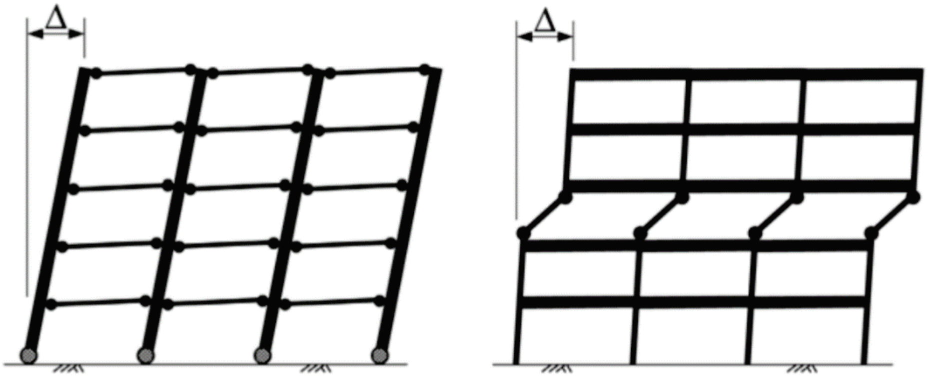


Figure C-61 Superstructure yield mechanisms limiting demands on the foundation

$$P_{UD} = P_{D+L} + \frac{P_{Seis}}{DCR_A}$$

$$M_{UD} = \frac{M_{Seis}}{DCR_M}$$

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:

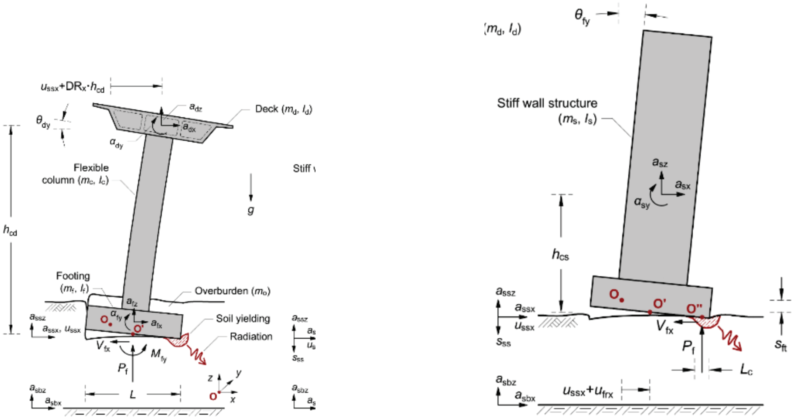


Figure C-62 Soil yielding governs the mechanisms limiting demands on the foundation

$$P_{UD} = P_{D+L} + \frac{P_{Seis}}{m}$$

$$M_{UD} = \frac{M_{Seis}}{m}$$

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

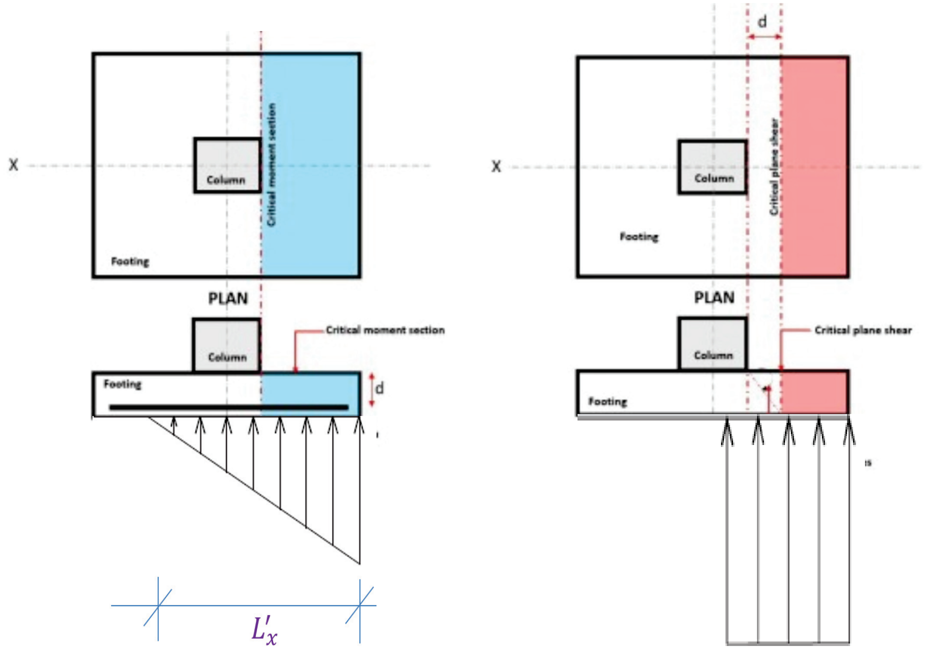


Figure C-63 Evaluation of the structural footing at each critical section for moment and shear

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.

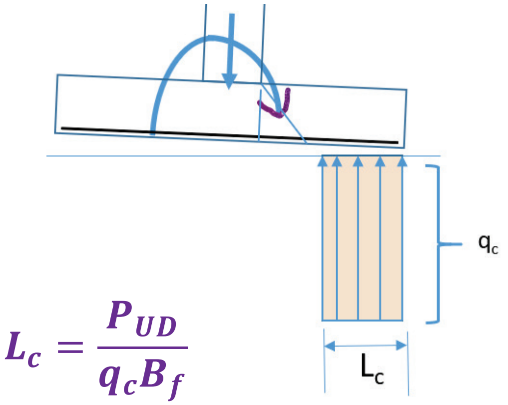


Figure C-64 **Soil pressure distribution for evaluation of the structural footing**

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.

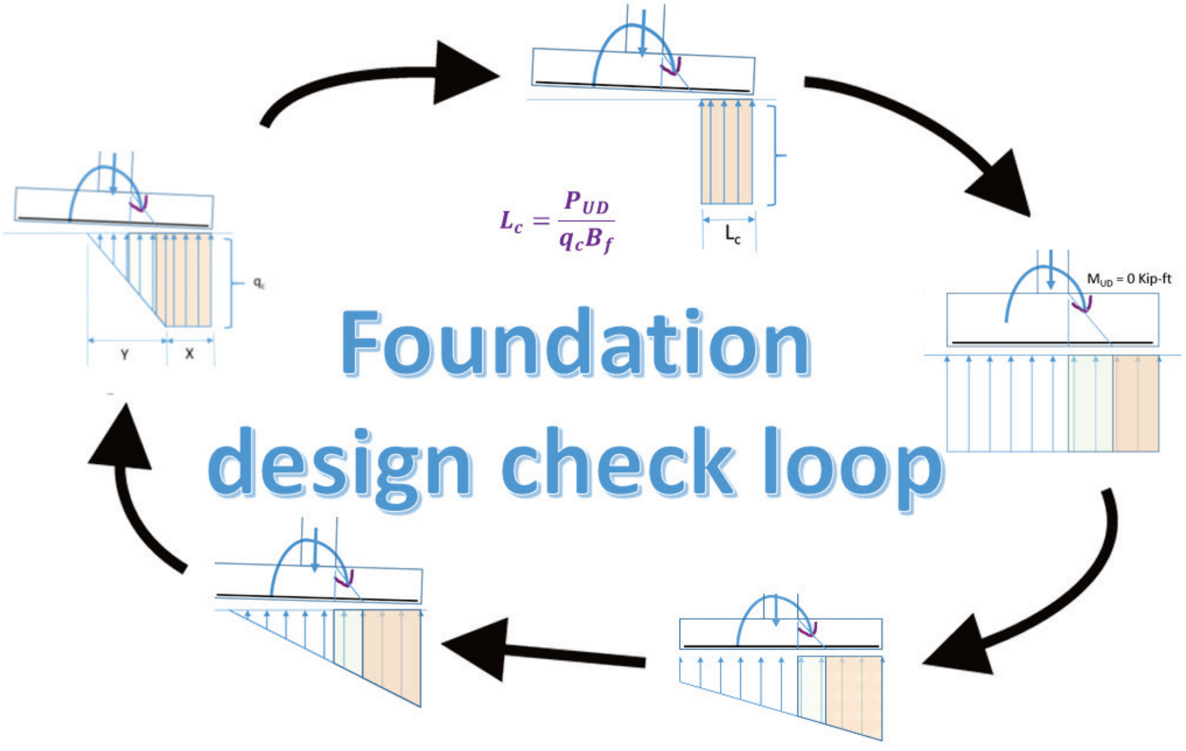


Figure C-65 **Alternative soil pressure distribution for evaluation of the structural footing**

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 $0 \leq Q_{min} < q < q_c$, where:

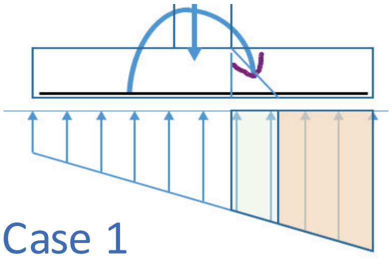


Figure C-66 Trapezoidal soil pressure distribution

$$Q_{max/min} = \frac{P_{UD}}{A_g}\left(1 \pm \frac{6e_{AC}}{L_f}\right); when\ e_{AC} \leq L_f/6 \qquad \text{C-17}$$

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 $0 \leq q < q_c$, where:

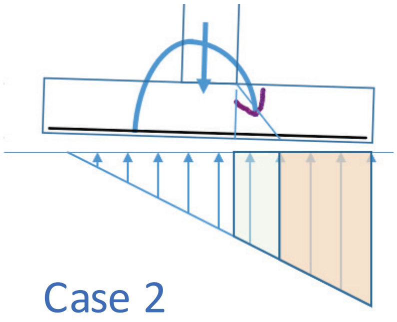


Figure C-67 Triangular soil pressure

$$Q_{max} = \frac{2P_{UD}}{3B_f\left(\frac{L_f}{2} - e_{AC}\right)};\ when\ \frac{L_f}{6} \leq e_{AC} \leq \frac{L_f}{2} \qquad \text{C-18}$$

$$Q_{min} = 0\ at\ L' = 3\left(\frac{L_f}{2} - e_{AC}\right) \leq L_f$$

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.

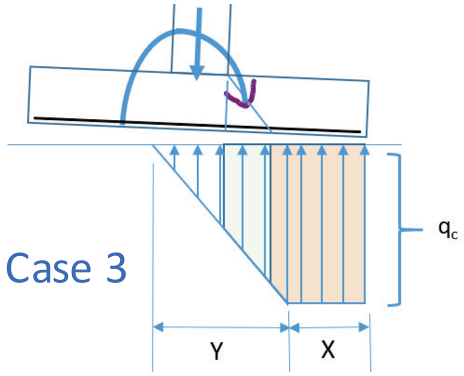


Figure C-68 Rectangular and triangular soil pressure distribution

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:

$$P' = \frac{P_{UD}}{q_c B_f}$$

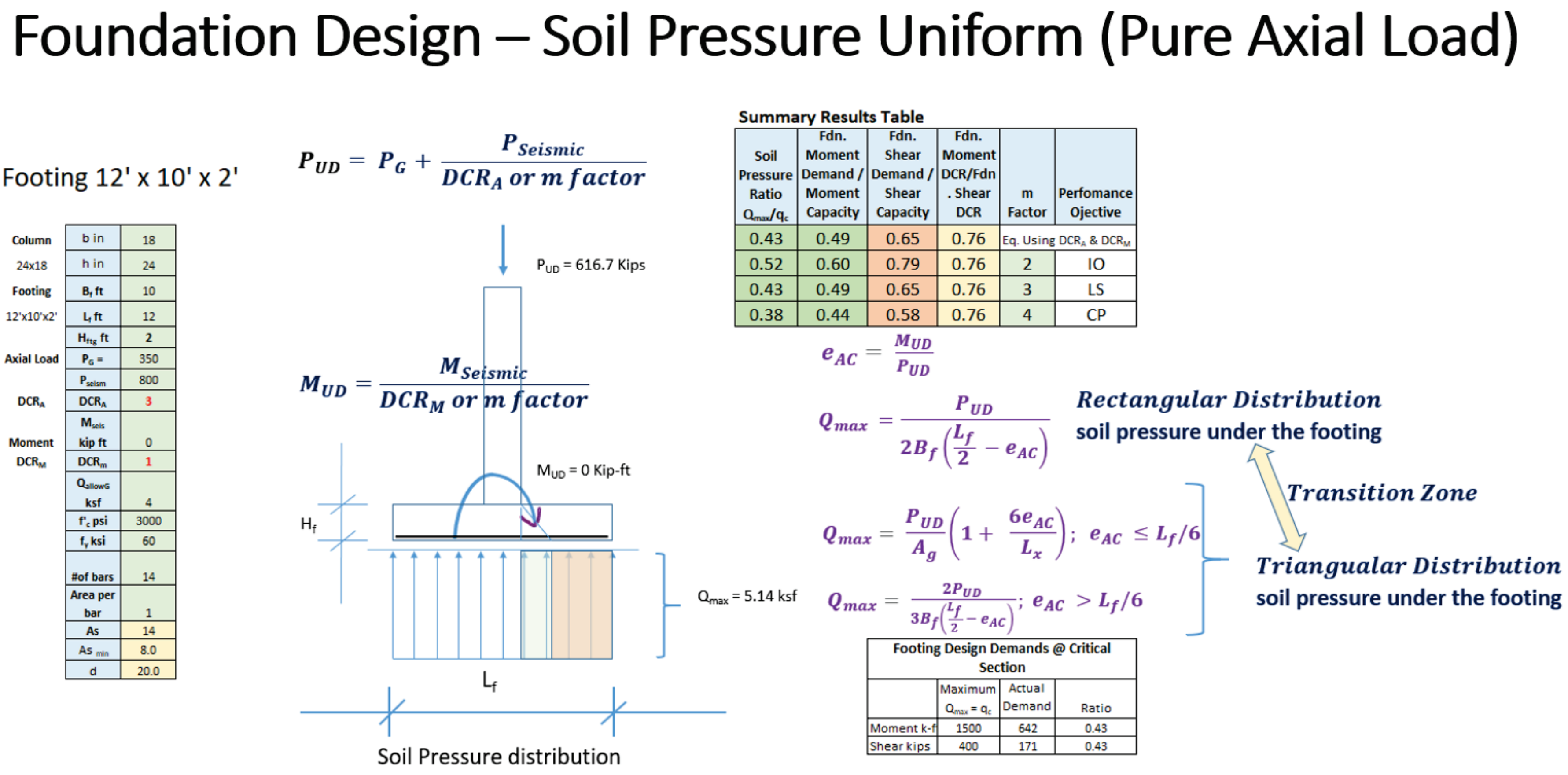
and

$$M' = \frac{M_{UD}}{q_c B_f}$$

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

$$P_{UD} = P_G + \frac{P_{Seismic}}{DCR_A \text{ or } m\ factor}$$

$$M_{UD} = \frac{M_{Seismic}}{DCR_M \text{ or } m\ factor}$$

Soil Pressure distribution

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

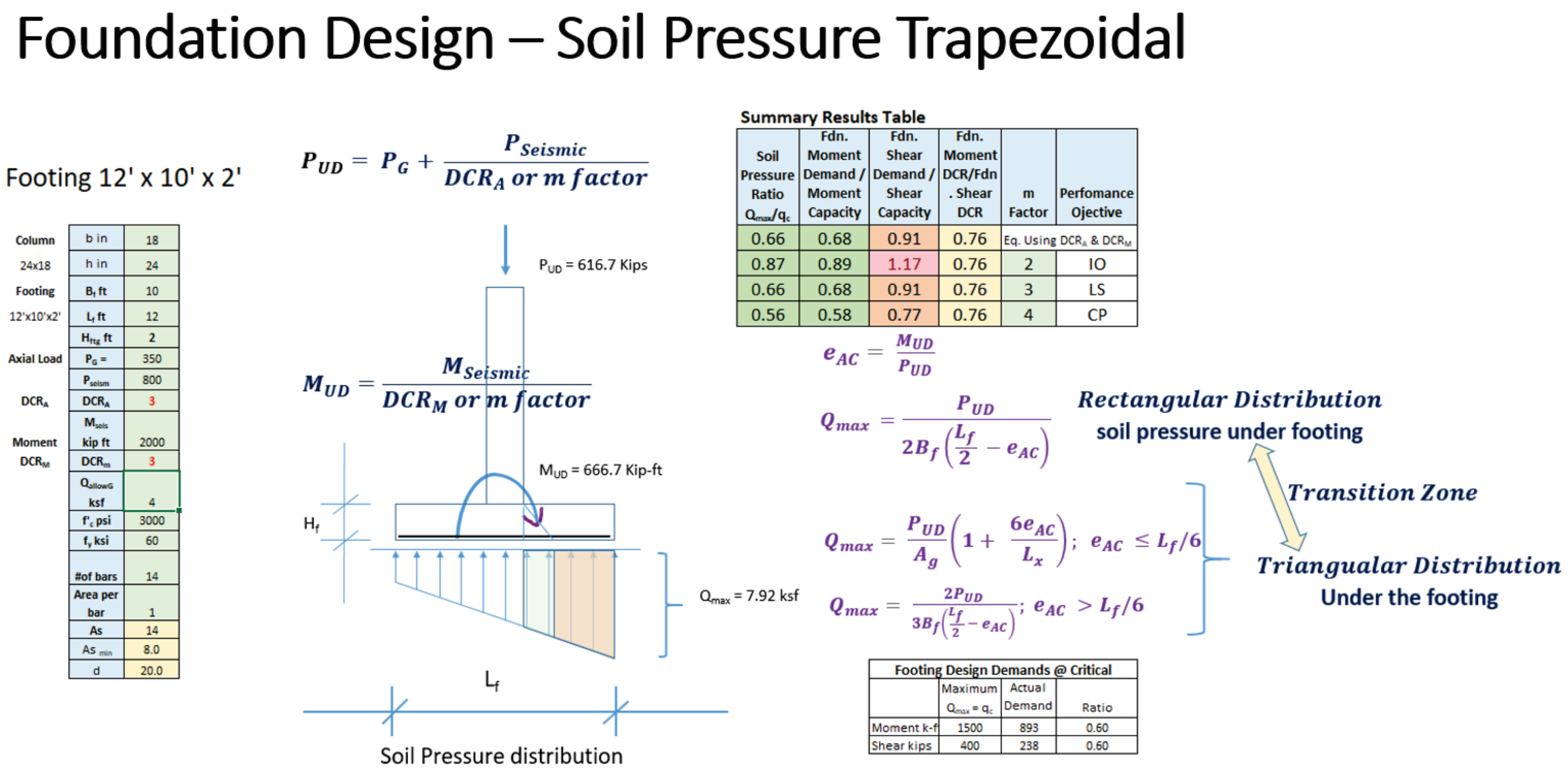
$$Q_{max} = \frac{P_{UD}}{A_g}\left(1 + \frac{6e_{AC}}{L_x}\right);\ e_{AC} \le L_f/6$$

$$Q_{max} = \frac{2P_{UD}}{3B_f\left(\frac{L_f}{2} - e_{AC}\right)};\ e_{AC} > L_f/6$$

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

$$P_{UD} = P_G + \frac{P_{Seismic}}{DCR_A\ or\ m\ factor}$$

P_{UD} = 616.7 Kips

$$M_{UD} = \frac{M_{Seismic}}{DCR_M\ or\ m\ factor}$$

M_{UD} = 666.7 Kip-ft

Q_{max} = 7.92 ksf

Soil Pressure distribution

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

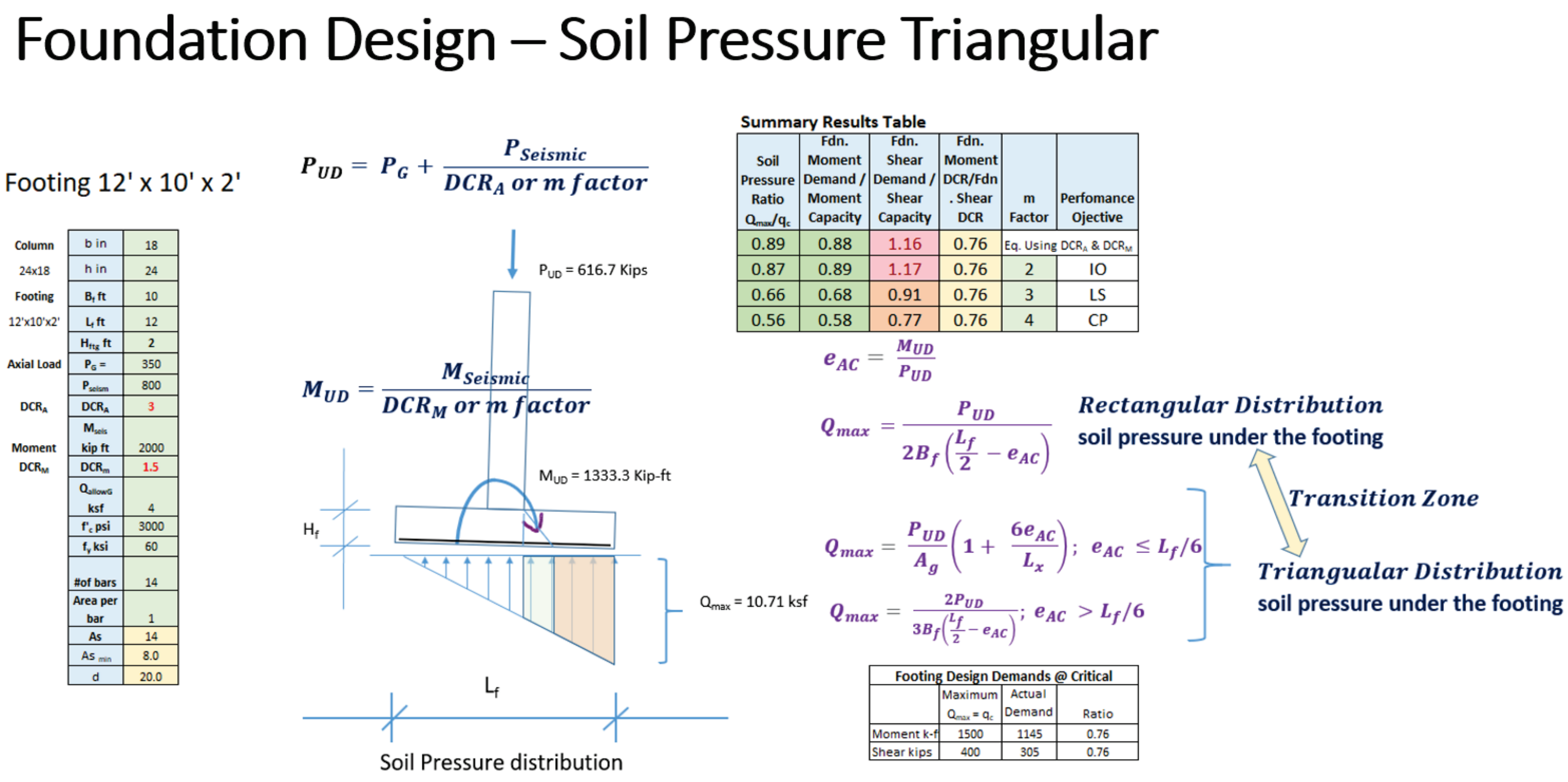
$$Q_{max} = \frac{P_{UD}}{A_g}\left(1 + \frac{6e_{AC}}{L_x}\right);\ e_{AC} \leq L_f/6$$

$$Q_{max} = \frac{2P_{UD}}{3B_f\left(\frac{L_f}{2} - e_{AC}\right)};\ e_{AC} > L_f/6$$

Triangular 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

$$P_{UD} = P_G + \frac{P_{Seismic}}{DCR_A \; or \; m \; factor}$$

P_{UD} = 616.7 Kips

$$M_{UD} = \frac{M_{Seismic}}{DCR_M \; or \; m \; factor}$$

M_{UD} = 1333.3 Kip-ft

H_f

Q_{max} = 10.71 ksf

L_f

Soil Pressure distribution

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

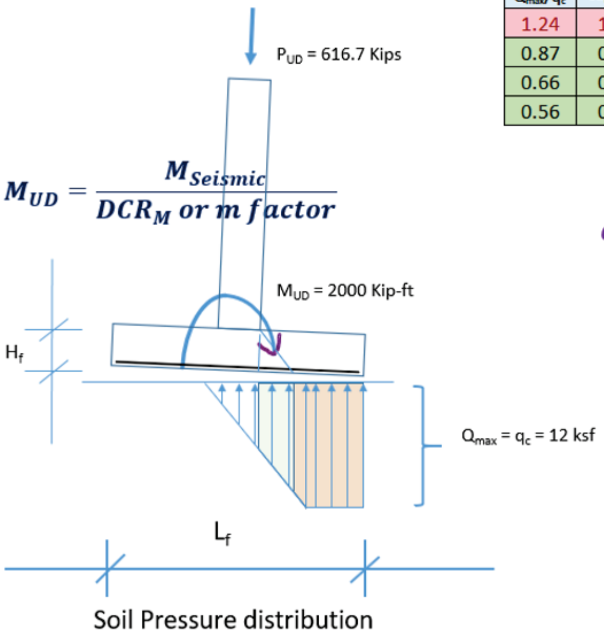
Foundation Design – Soil Pressure in Transition Zone

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-f	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

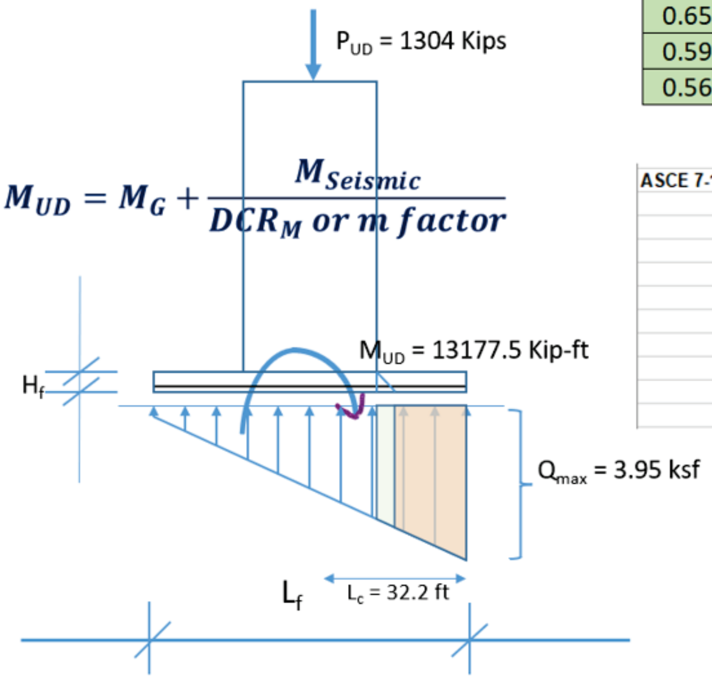
Results for Archetype Building 1 – ASCE/SEI 7-10

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

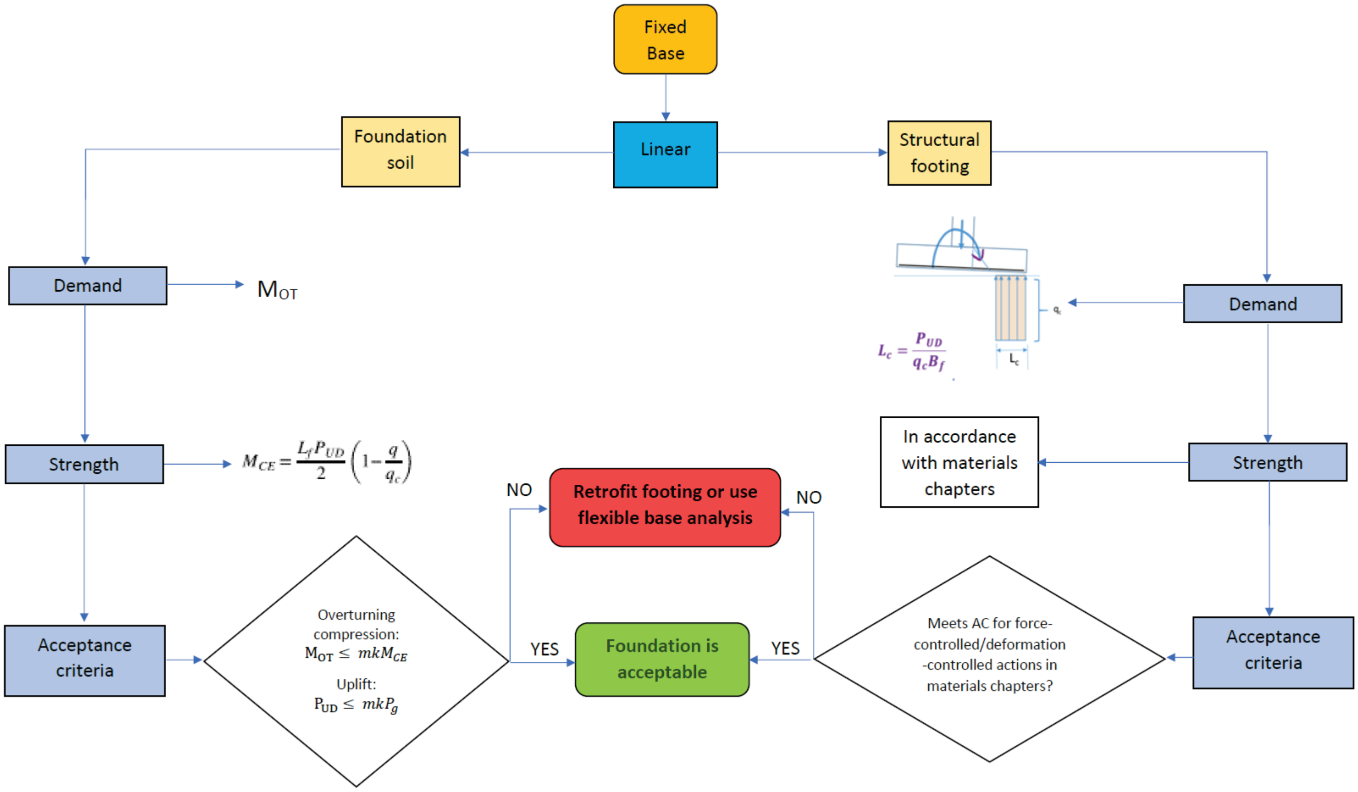


Figure C-74 Flowchart for Proposal B

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:

$$M_{CE} = \frac{L_f P_{UD}}{2}\left(1 - \frac{q}{q_c}\right)$$ 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:

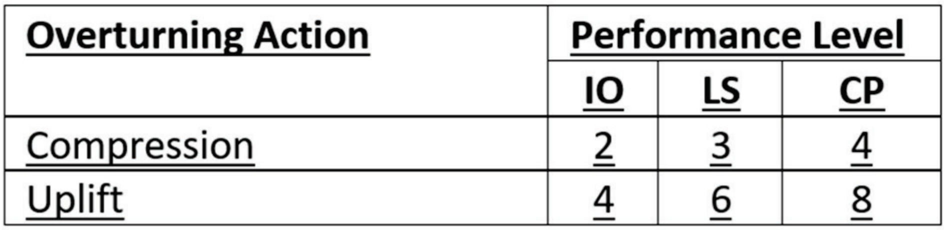
$$P_{UD} \leq m\kappa P_g$$

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

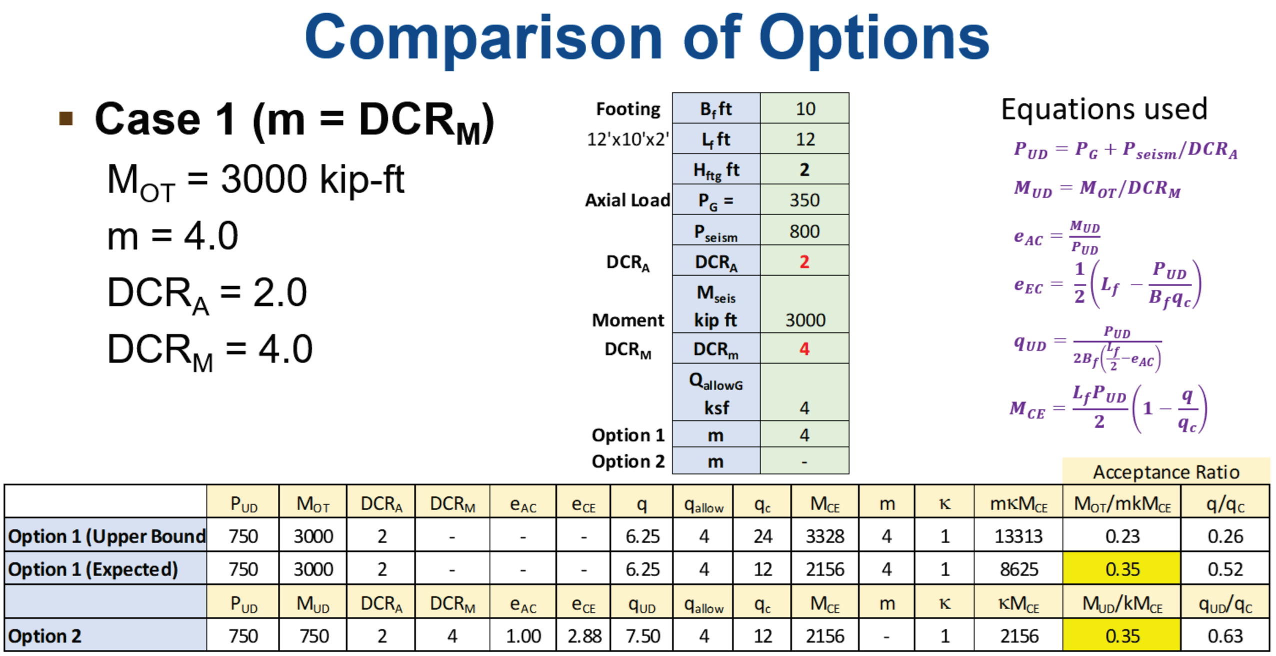
To decide between the two options, a spreadsheet was created to show the similarities and differences between the options. In addition, an alternate procedure similar to Option 2 was proposed, and the outcomes from a different moment frame example than Archetype 2 was also used to compare the results 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

$$P_{UD} = P_G + P_{seism}/DCR_A$$

$$M_{UD} = M_{OT}/DCR_M$$

$$e_{AC} = \frac{M_{UD}}{P_{UD}}$$

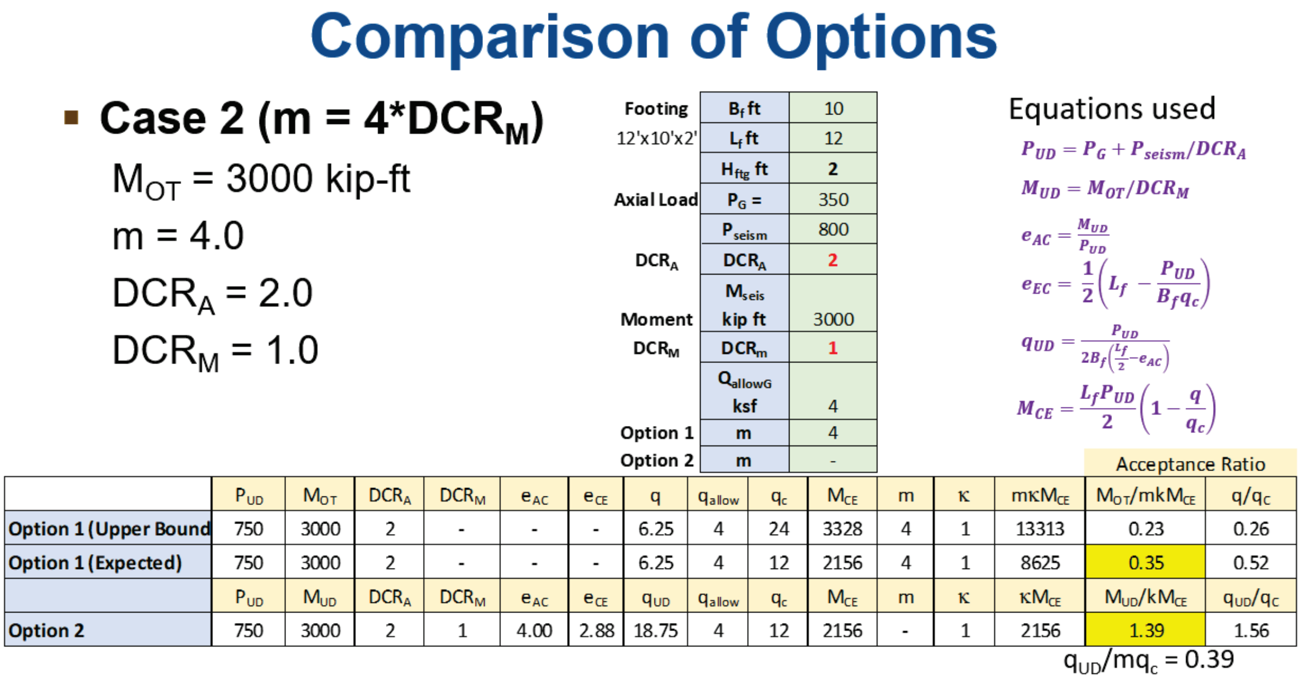
$$e_{EC} = \frac{1}{2}\left(L_f - \frac{P_{UD}}{B_f q_c}\right)$$

$$q_{UD} = \frac{P_{UD}}{2B_f\left(\frac{L_f}{2} - e_{AC}\right)}$$

$$M_{CE} = \frac{L_f P_{UD}}{2}\left(1 - \frac{q}{q_c}\right)$$

														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

$$P_{UD} = P_G + P_{seism}/DCR_A$$

$$M_{UD} = M_{OT}/DCR_M$$

$$e_{AC} = \frac{M_{UD}}{P_{UD}}$$

$$e_{EC} = \frac{1}{2}\left(L_f - \frac{P_{UD}}{B_f q_c}\right)$$

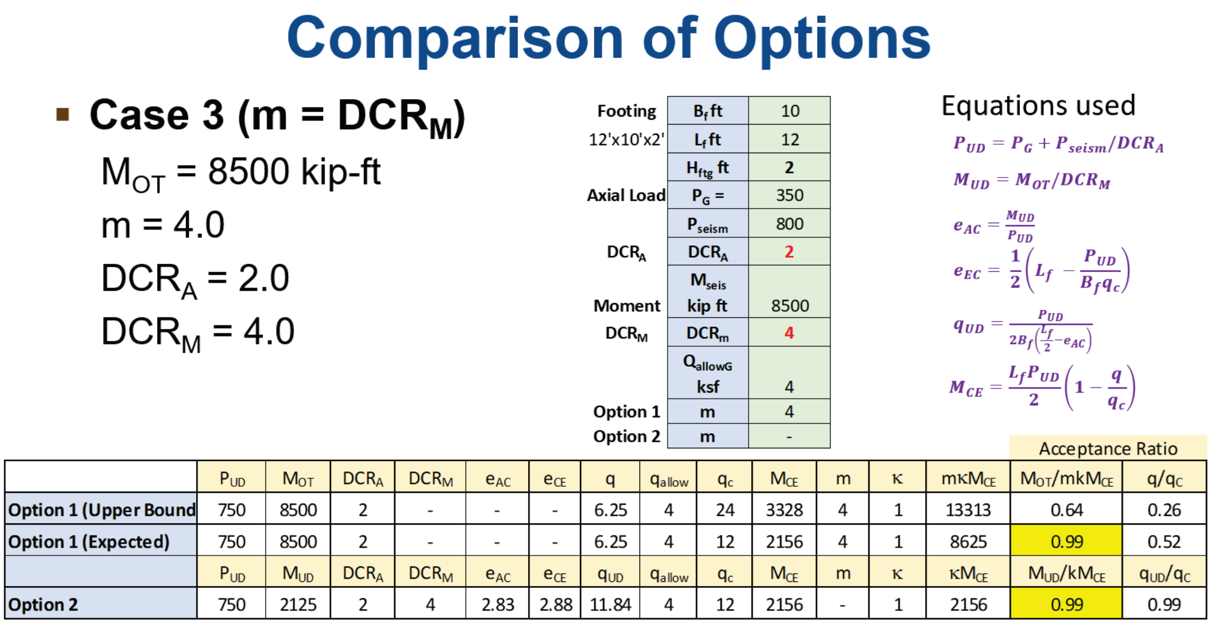
$$q_{UD} = \frac{P_{UD}}{2B_f\left(\frac{L_f}{2} - e_{AC}\right)}$$

$$M_{CE} = \frac{L_f P_{UD}}{2}\left(1 - \frac{q}{q_c}\right)$$

														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	**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**	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.

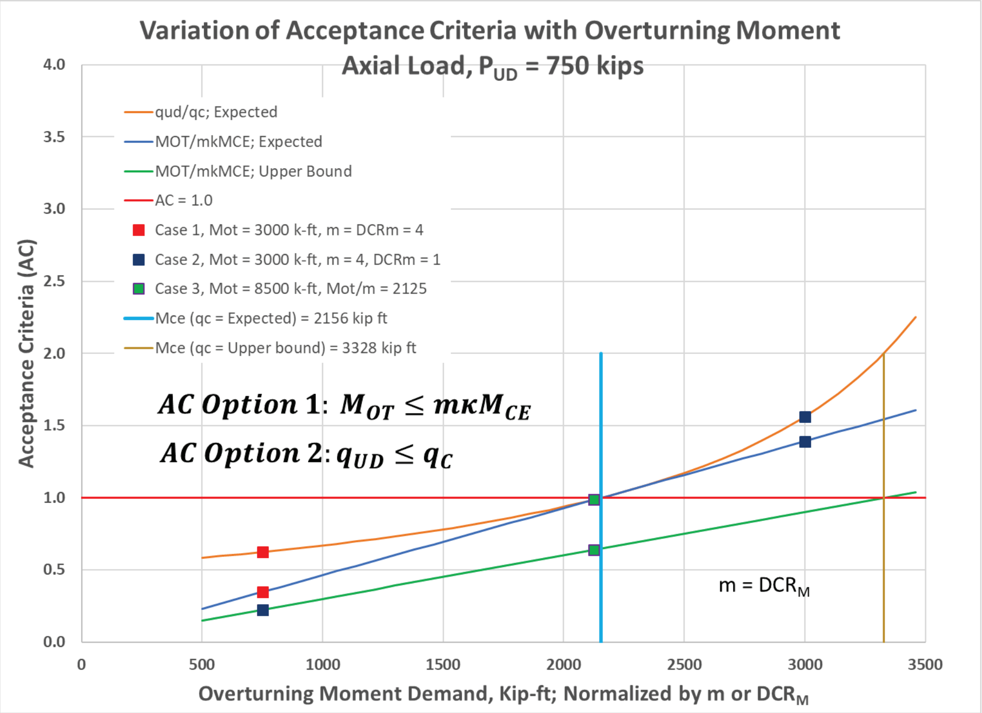


Figure C-78 Variation of acceptance criteria between Option 1 and Option 2.

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.

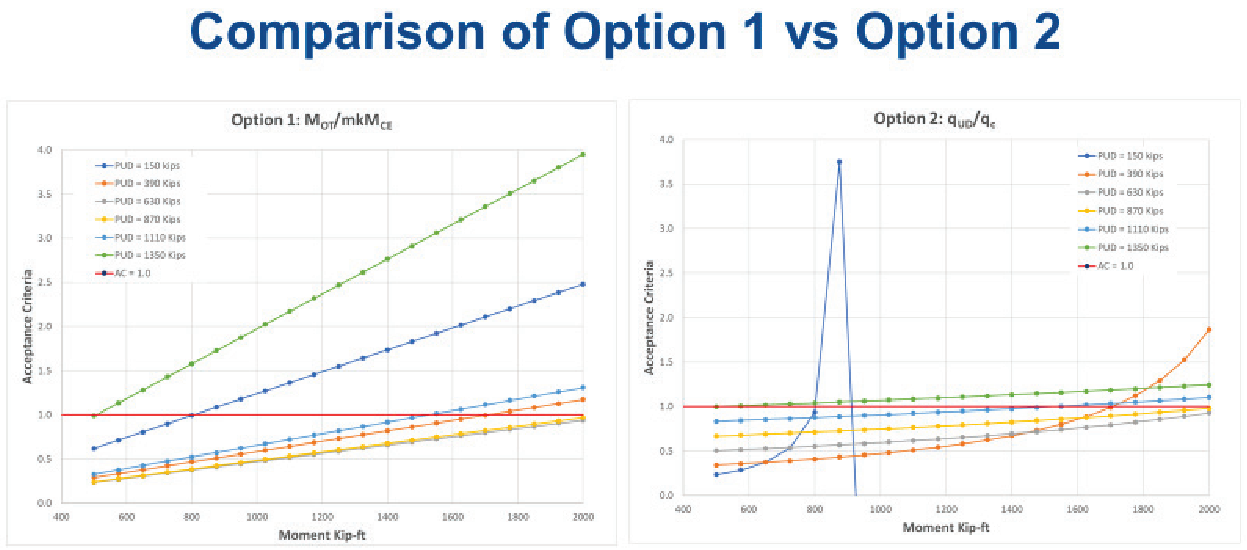


Figure C-79 Acceptance Ratios for the two options with varying axial load

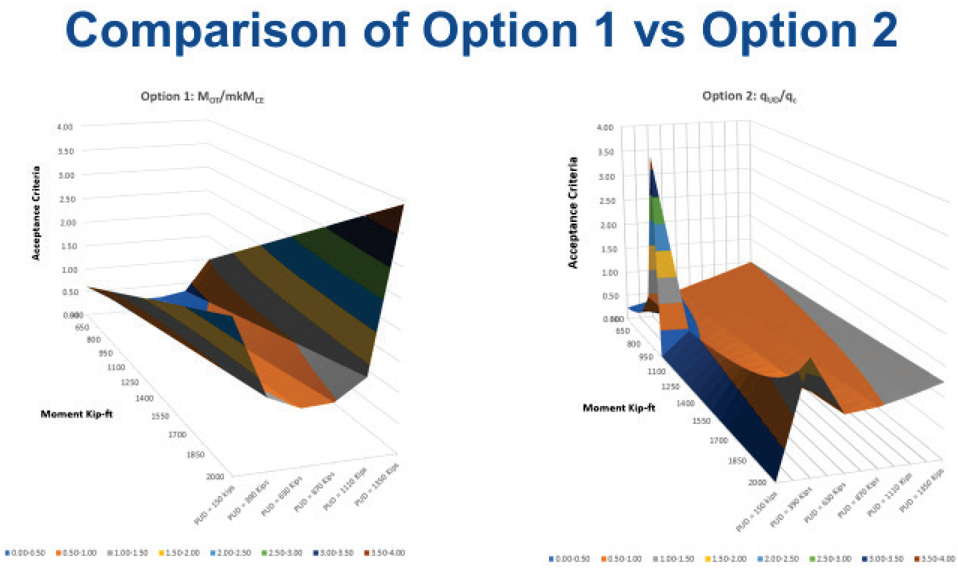
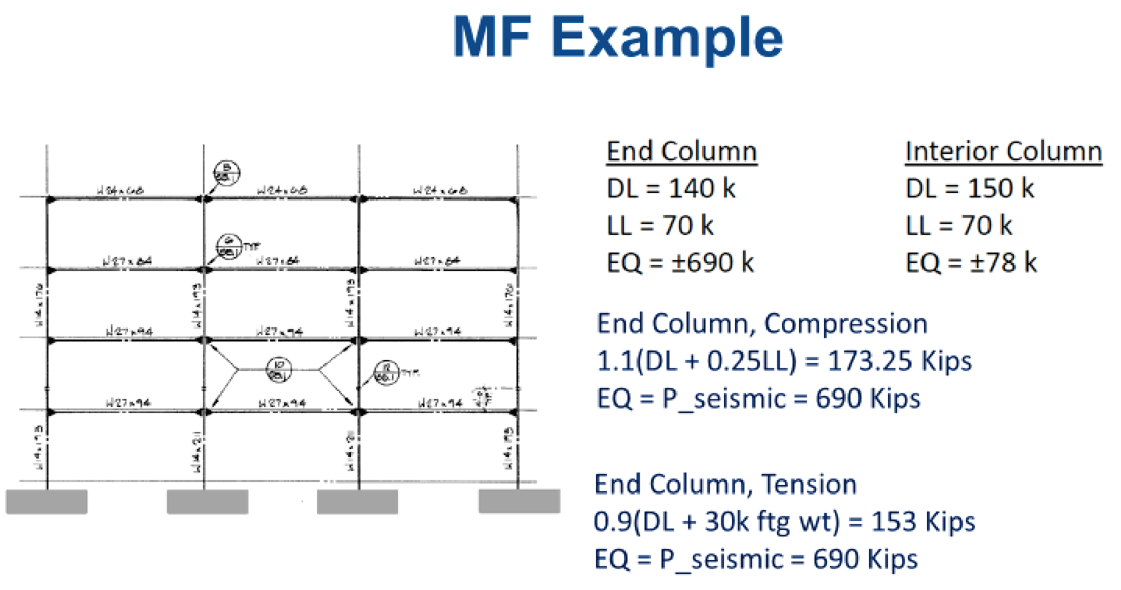


Figure C-80 Three-dimension representation showing the comparison between the two options with varying axial load.

Example 2: Three Bay Moment Frame



MF Example

End Column
DL = 140 k
LL = 70 k
EQ = ±690 k

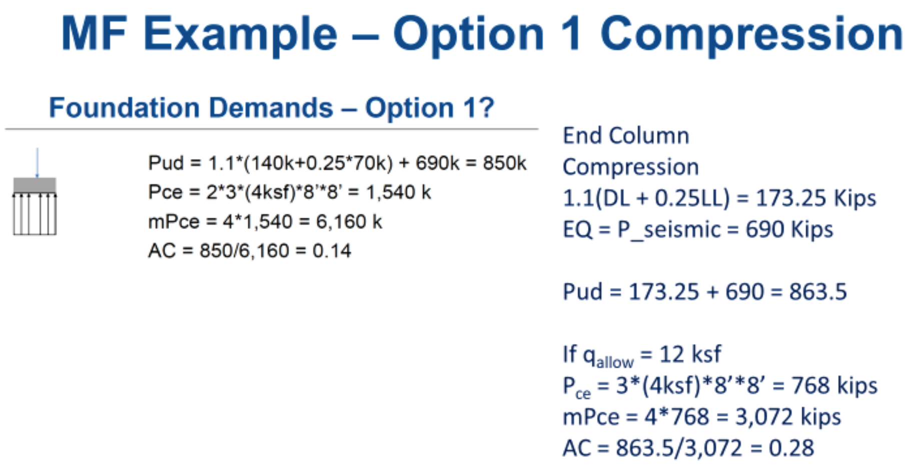
Interior Column
DL = 150 k
LL = 70 k
EQ = ±78 k

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?

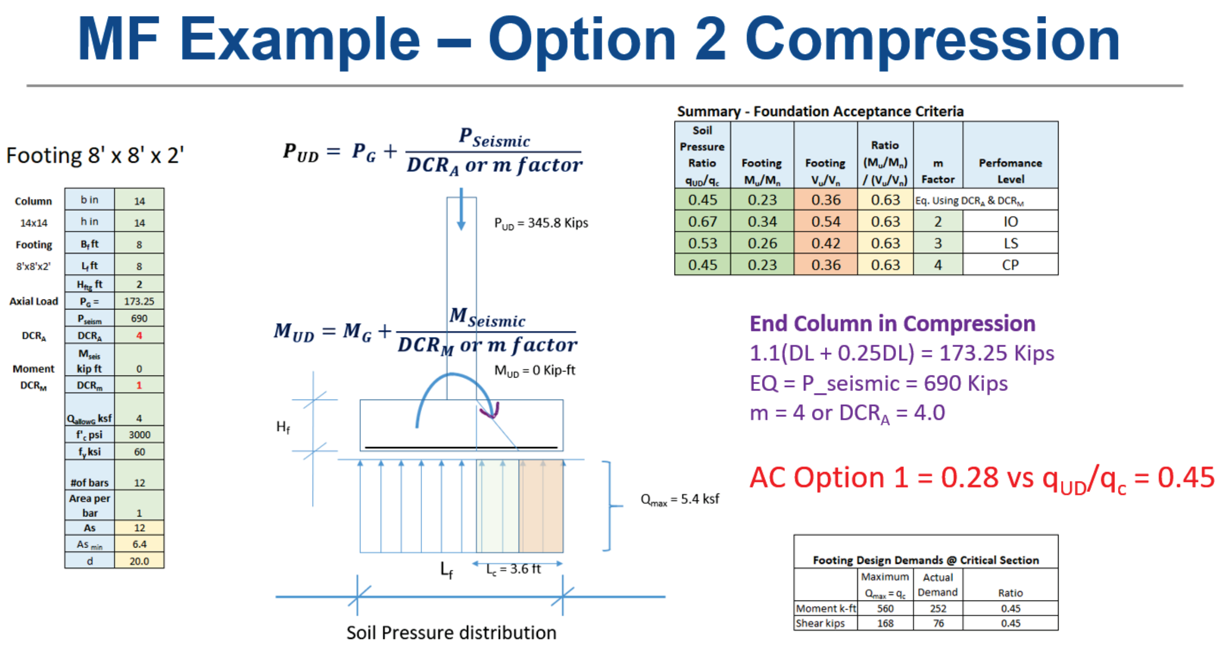
Pud = 1.1*(140k+0.25*70k) + 690k = 850k
Pce = 2*3*(4ksf)*8'*8' = 1,540 k
mPce = 4*1,540 = 6,160 k
AC = 850/6,160 = 0.14

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

$$P_{UD} = P_G + \frac{P_{Seismic}}{DCR_A \text{ or } m\ factor}$$

$$M_{UD} = M_G + \frac{M_{Seismic}}{DCR_M \text{ or } m\ factor}$$

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

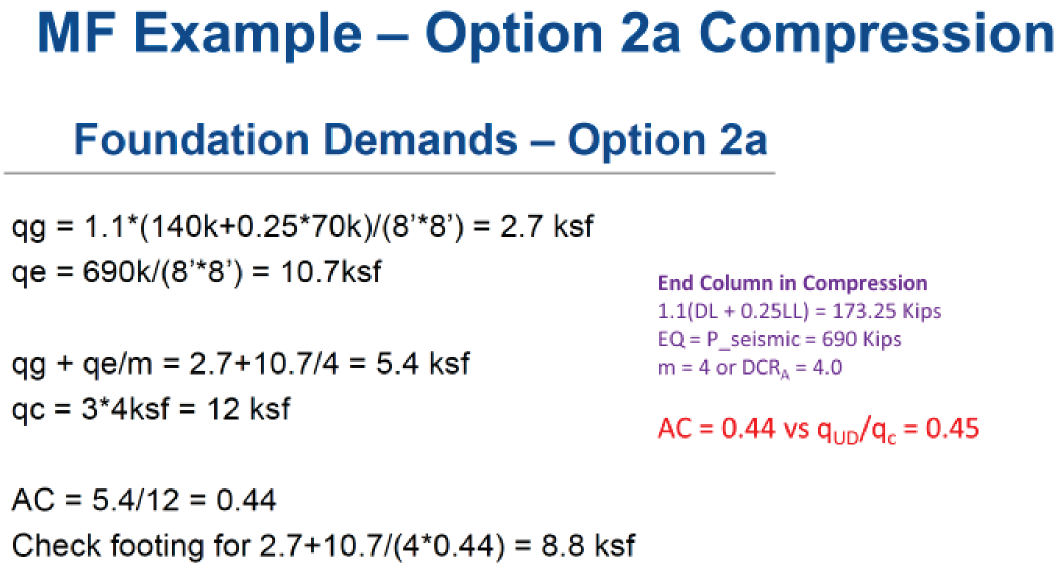
EQ = P_seismic = 690 Kips

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

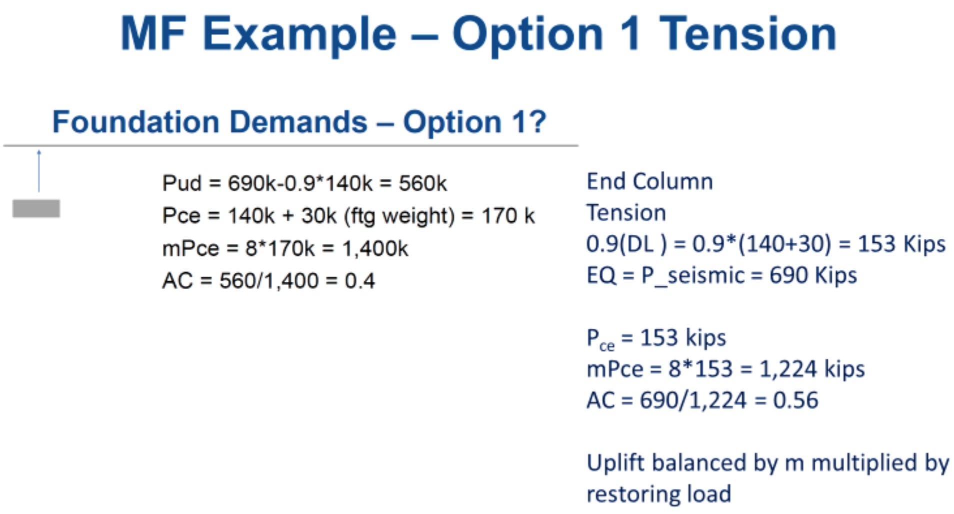
m = 4 or DCR_A = 4.0

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.



MF Example – Option 1 Tension

Foundation Demands – Option 1?

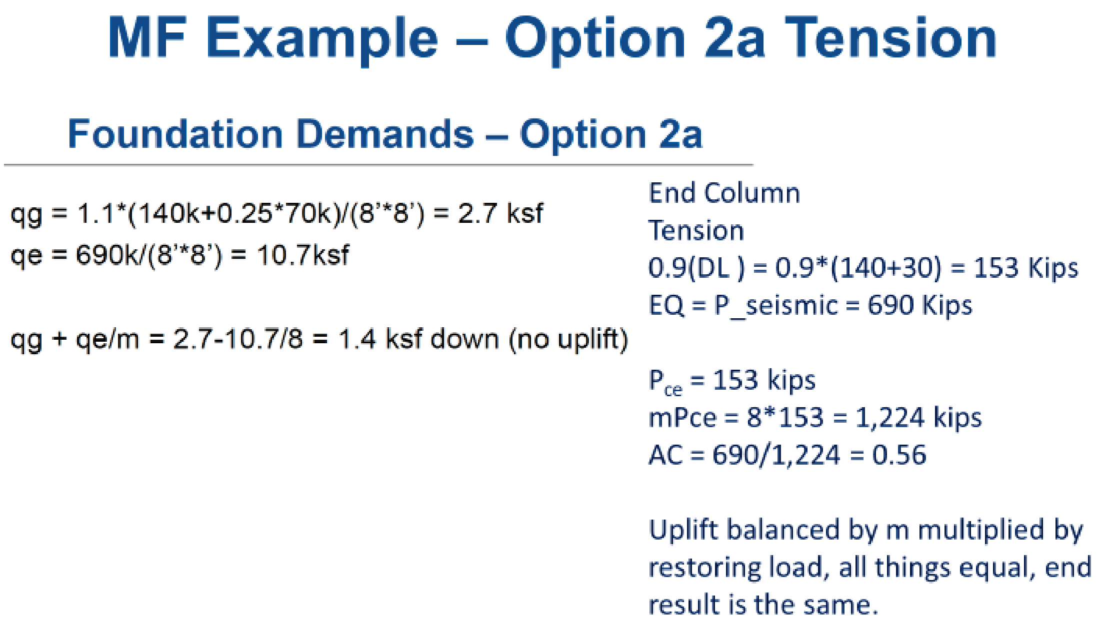
Pud = 690k-0.9*140k = 560k
Pce = 140k + 30k (ftg weight) = 170 k
mPce = 8*170k = 1,400k
AC = 560/1,400 = 0.4

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

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.

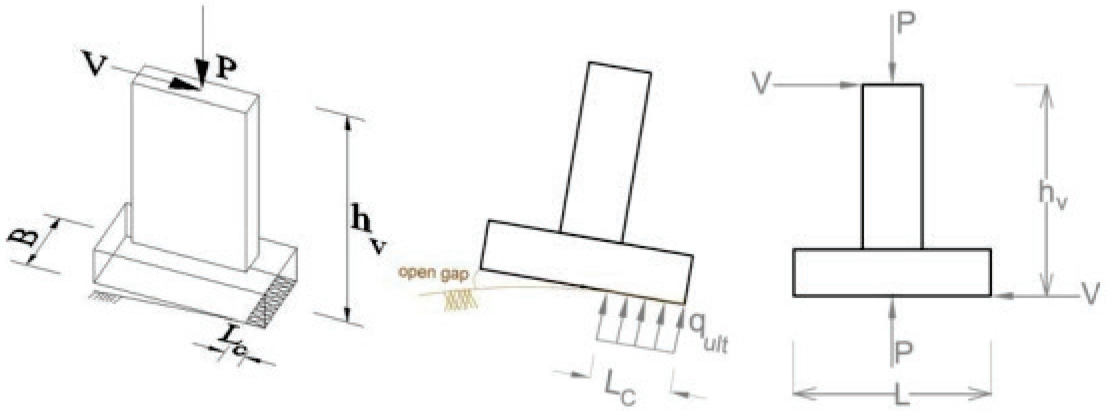


Figure C-87 Rocking on an isolated footing

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.

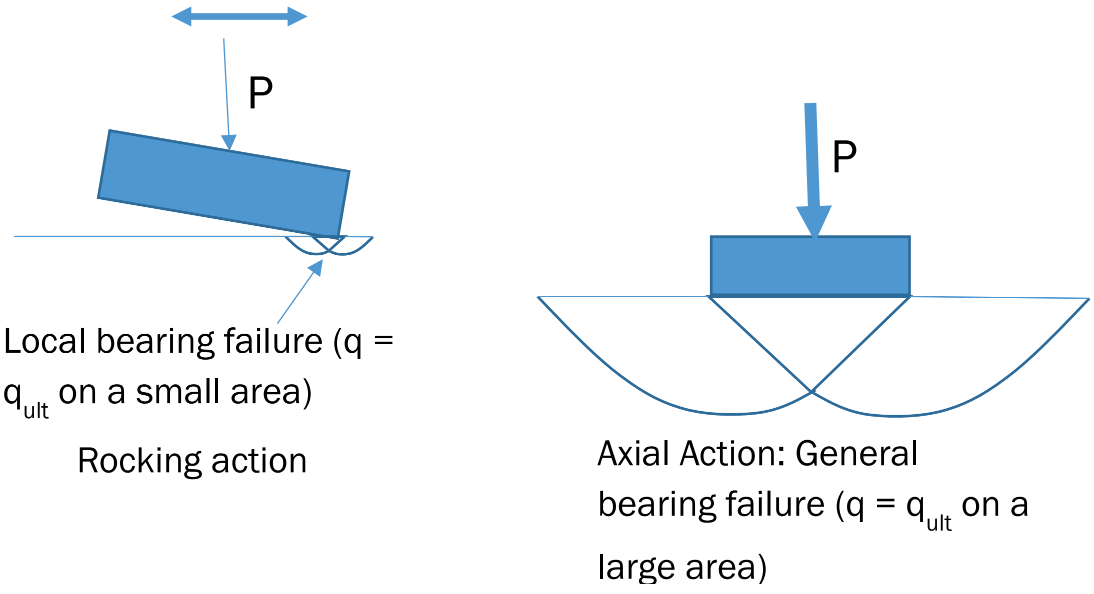


Figure C-88 Comparison of soil bearing failure from rocking and axial actions.

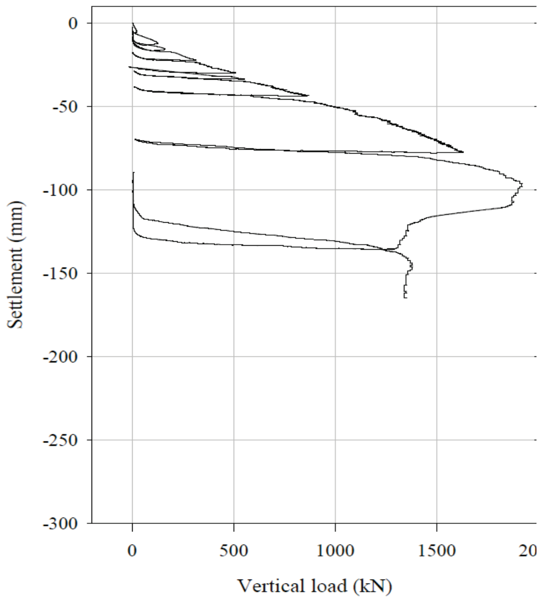


Figure C-89 Soil force deformation for cyclic axial compression action.

C.4.9 Decision on Option Selection

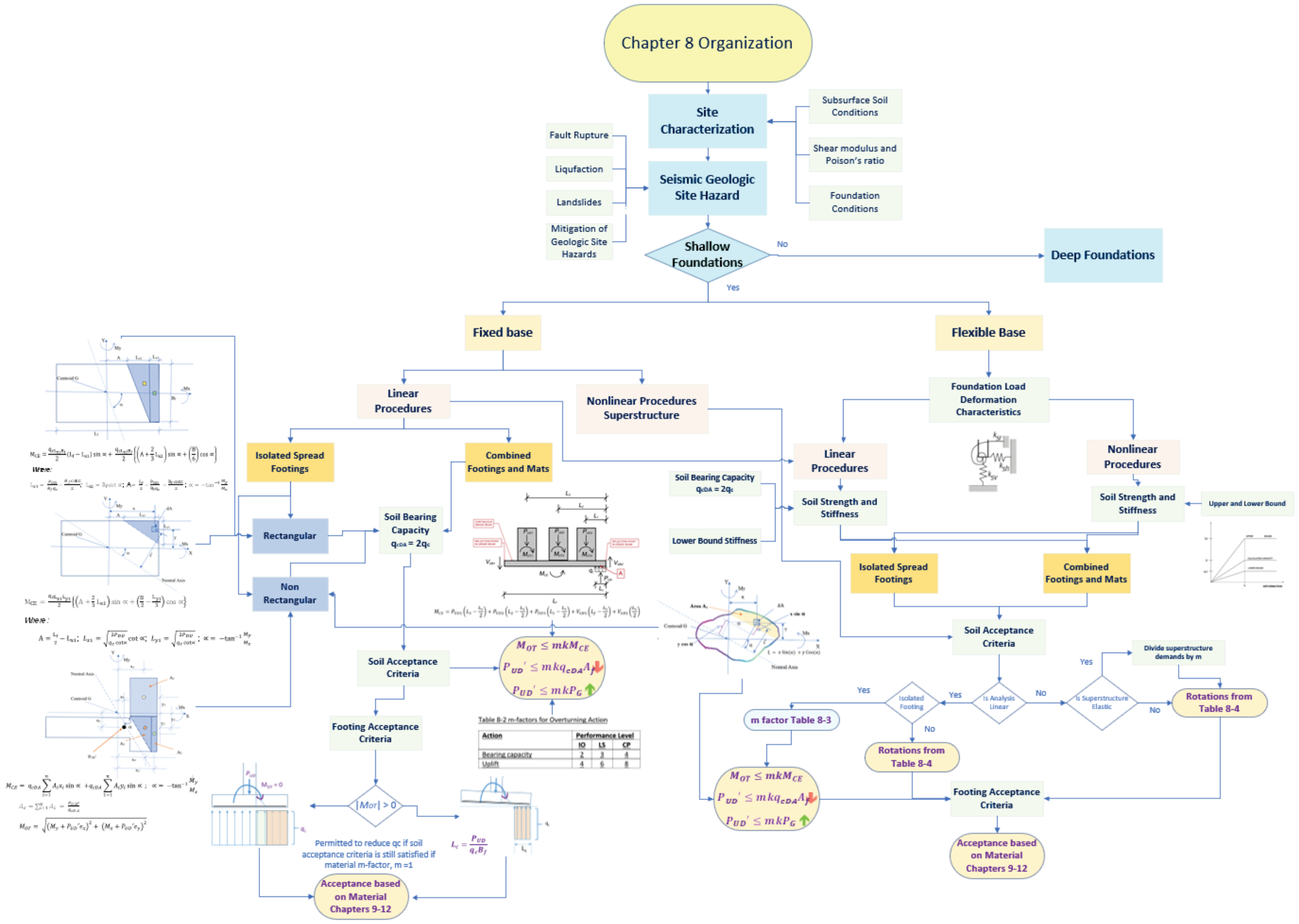
From the results of the comparison between the two options, the following was decided.

- 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.



Permitted to reduce qc if soil acceptance criteria is still satisfied if material m-factor, m =1

Table 8-2 m-factors for Overturning Action

Action	Performance Level		
	IO	LS	CP
Bearing capacity	2	3	4
Uplift	4	6	8

Figure C-90 Flowchart of proposed restructure of Chapter 8, for shallow foundations

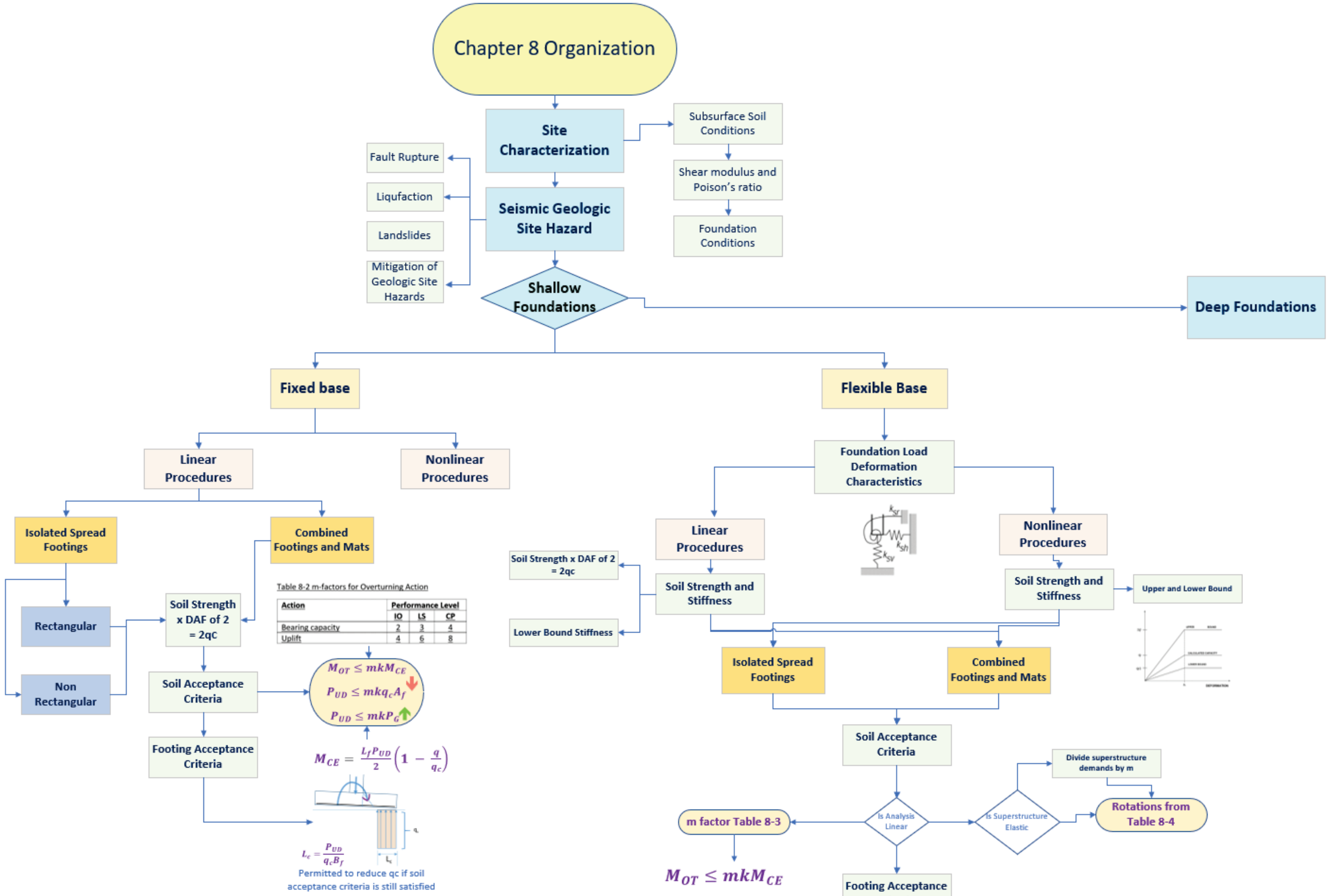


Table 8-2 m-factors for Overturning Action

Action	Performance Level		
	IO	LS	CP
Bearing capacity	2	3	4
Uplift	4	6	8

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.

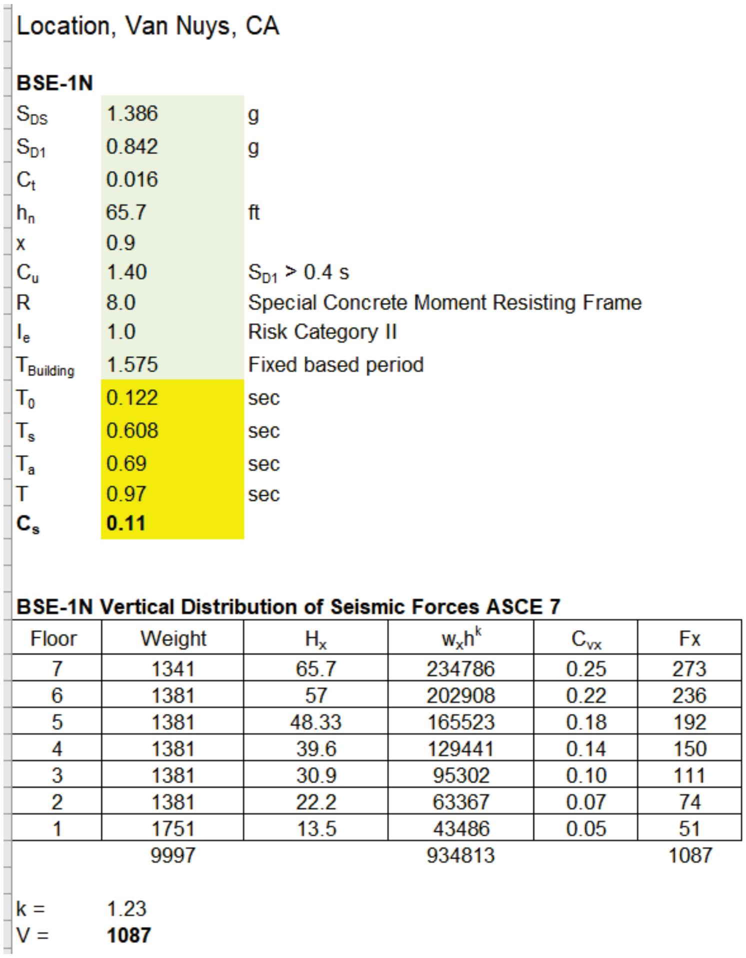
Appendix C1 -Base Shear Calculations:

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**	

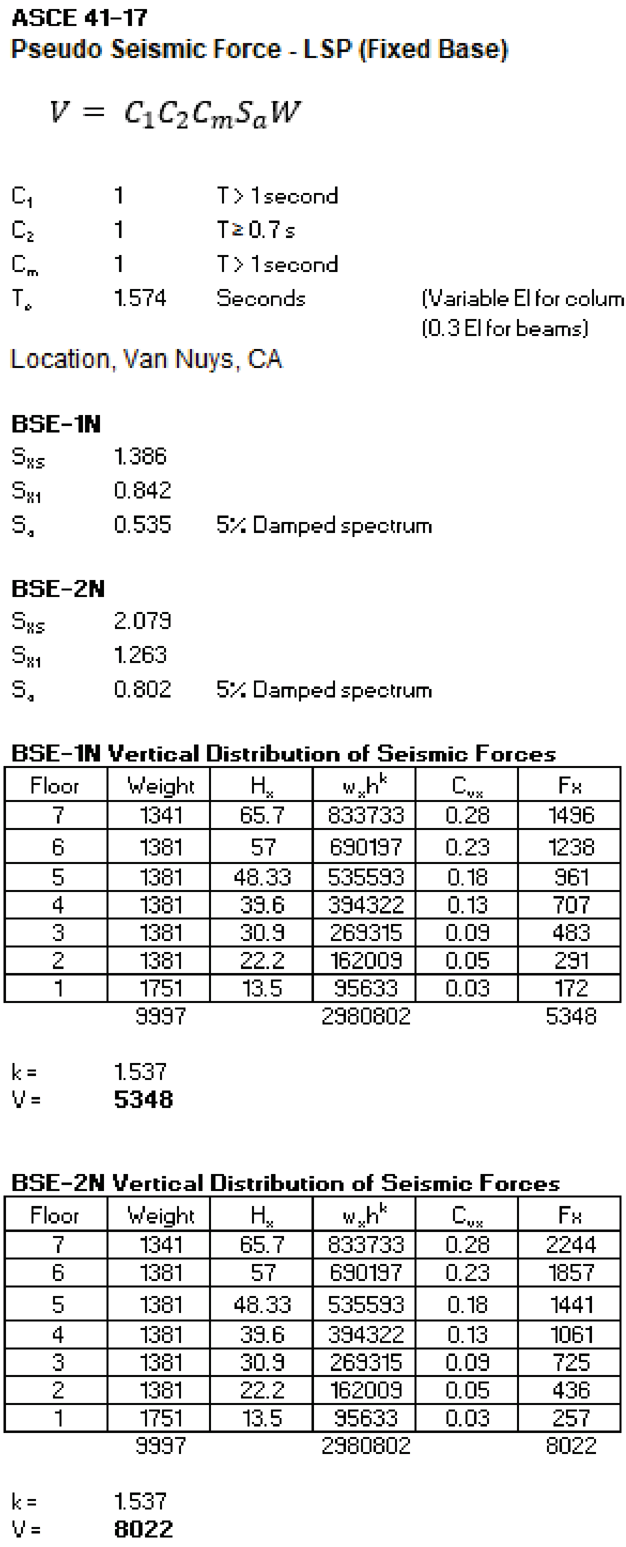
BSE-1N Vertical Distribution of Seismic Forces ASCE 7

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**

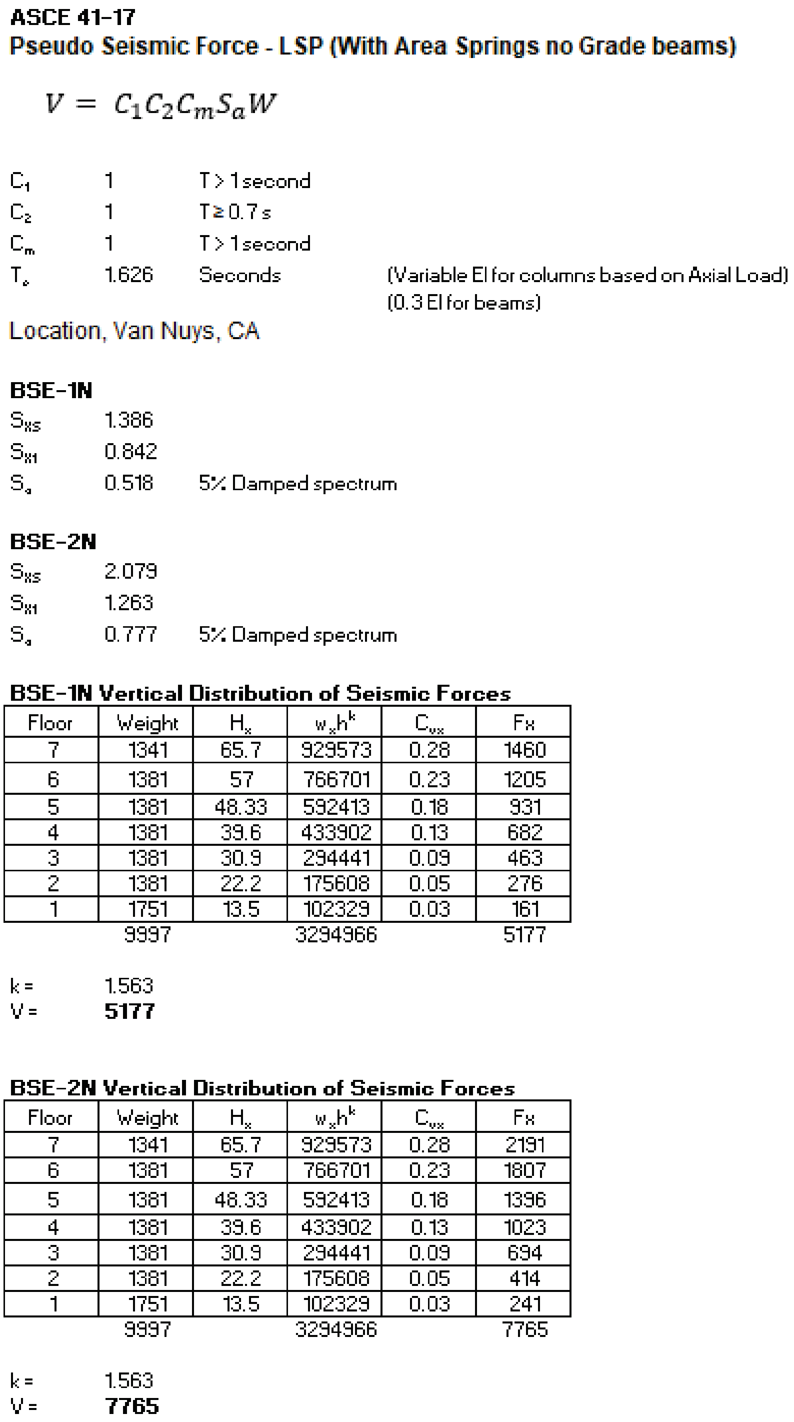
BSE-2N Vertical Distribution of Seismic Forces

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**

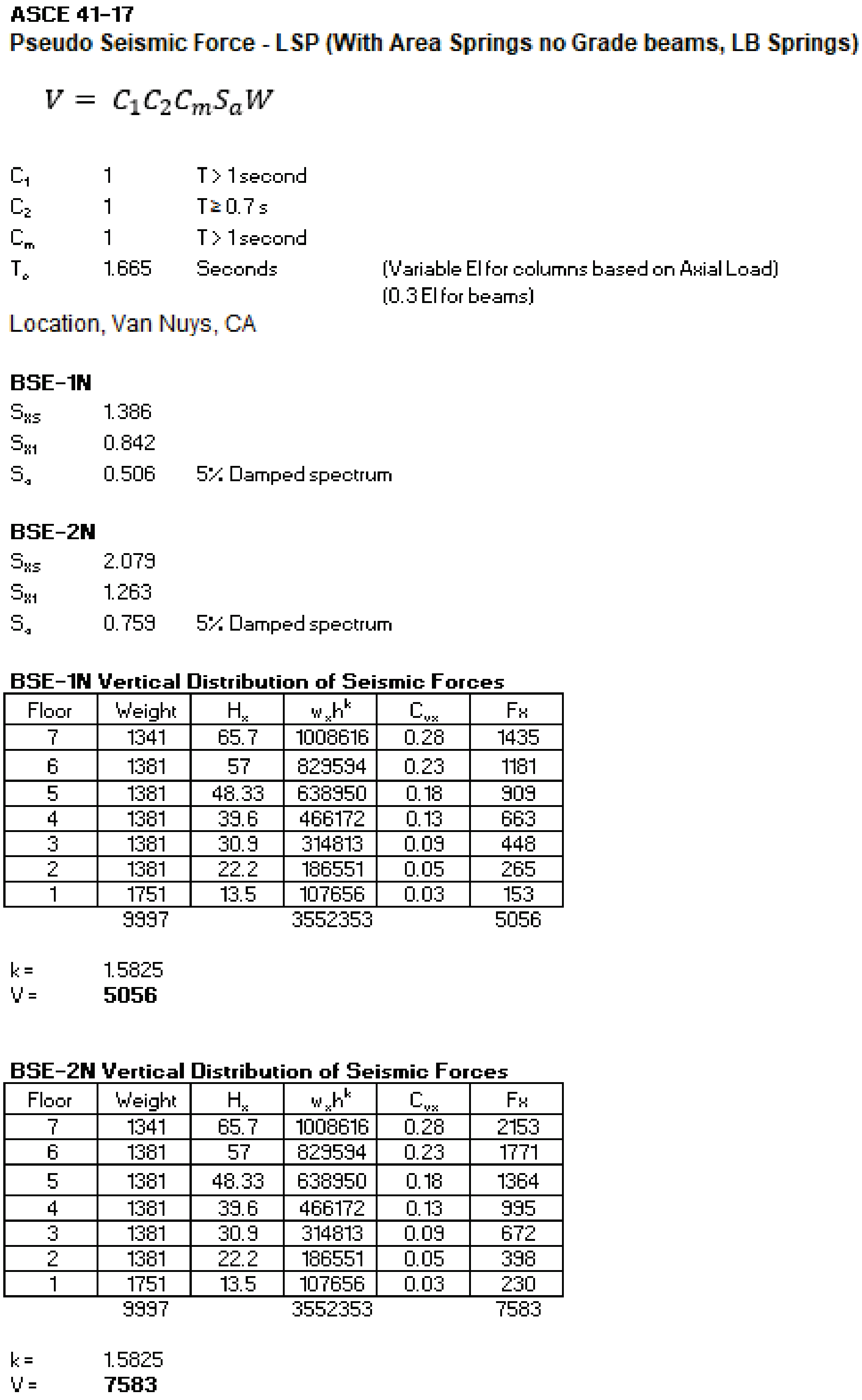
BSE-2N Vertical Distribution of Seismic Forces

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**

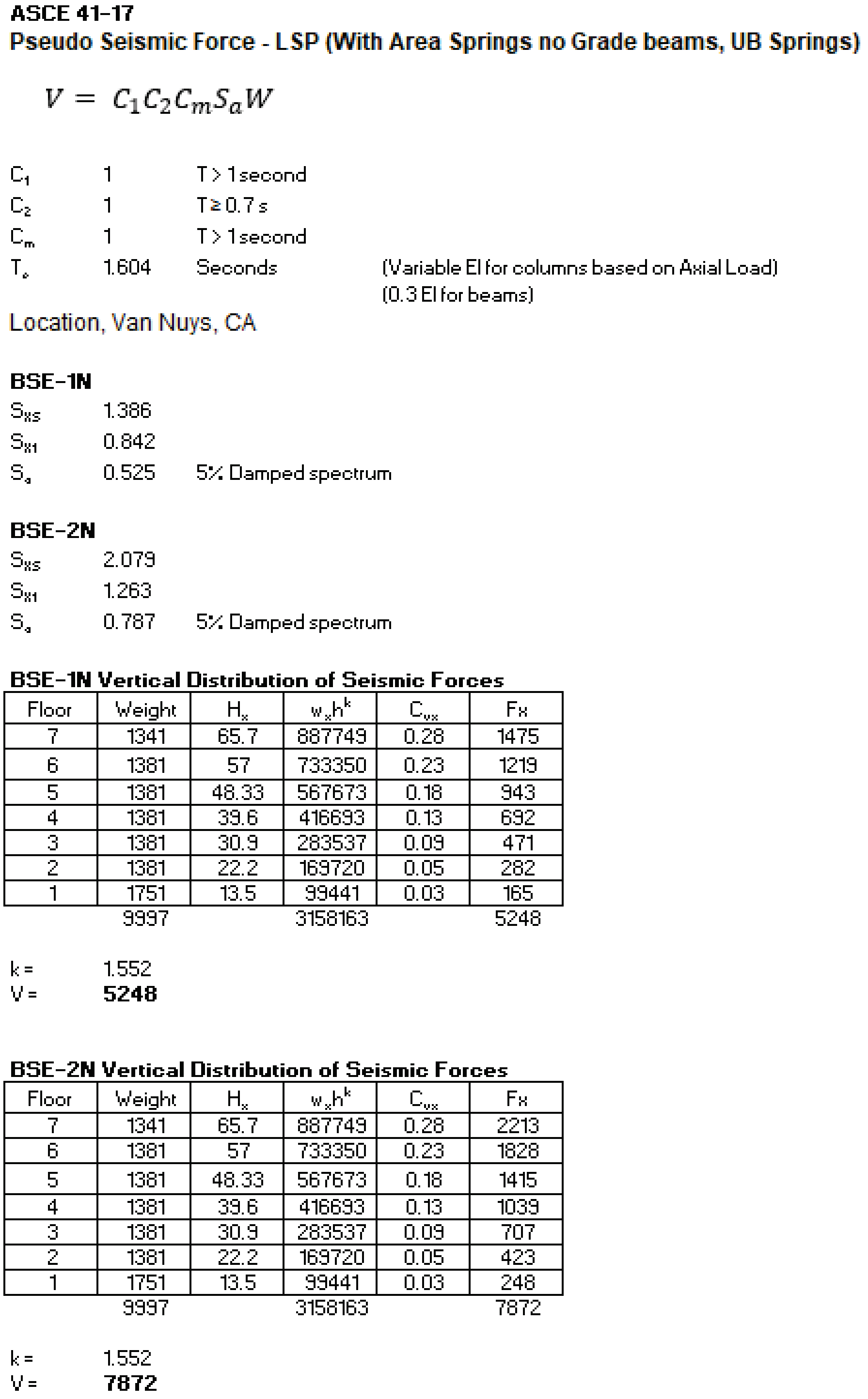
BSE-2N Vertical Distribution of Seismic Forces

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.

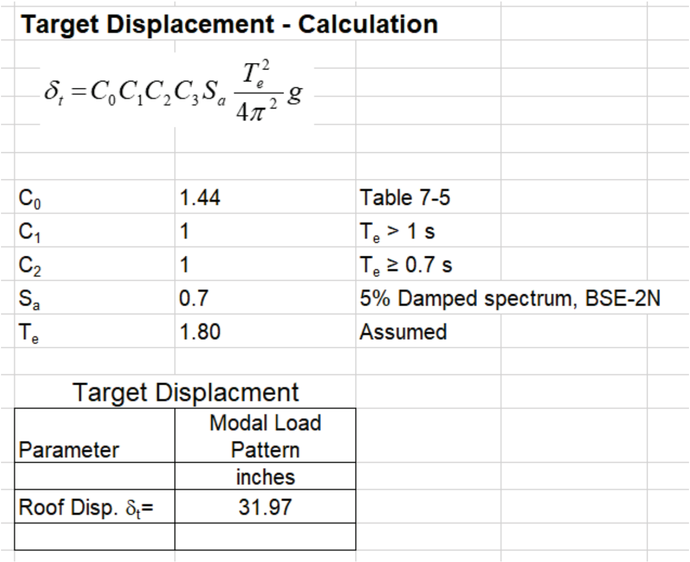
K_i = Elastic lateral stiffness of the building in the direction under consideration.

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

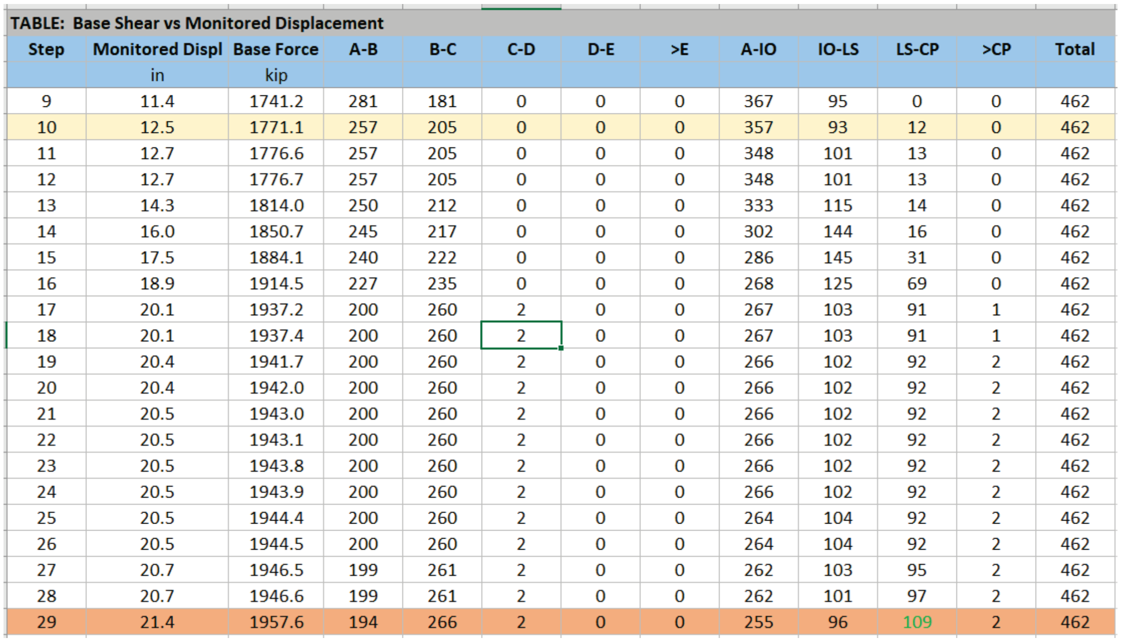


TABLE: Base Shear vs Monitored Displacement

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

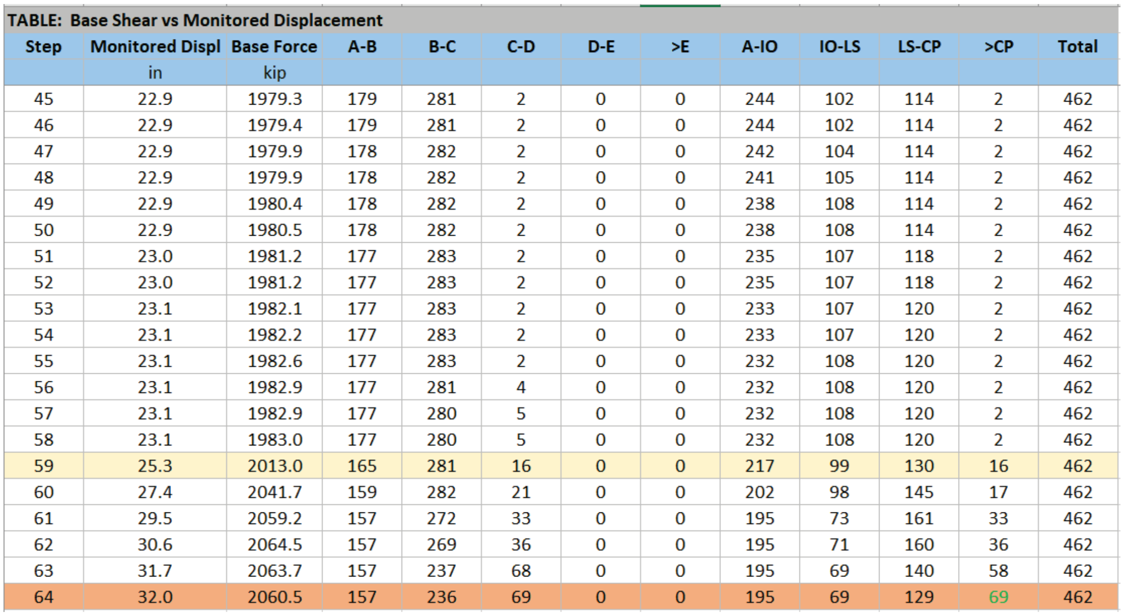


TABLE: Base Shear vs Monitored Displacement

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”