Part 3, Appendix D: Design Examples

D.1 Motivation

Chapter 8 of ASCE/SEI 41-17 has been completely rewritten and restructured for usability and technical content in ASCE/SEI 41-23, the next version of the standard. Given the extensive changes made to this chapter, three separate design examples are developed to help users of ASCE/SEI 41 on how the new provisions are to be applied and to understand the impact of the changes. The first example checks the foundation acceptance for a single cantilevered shear wall on a strip footing. The second example checks the foundation acceptance for a single bay braced frame on isolated and combined footings. The third example is of a stair tower on a Mat foundation. Each example demonstrates use of a different provision in the Chapter, and how the new provisions compare with the provisions in ASCE/SEI 41-17. These design examples only look at the acceptance for overturning stability and soil bearing, not the acceptance of the foundation structural component. These design examples reflect the final version of the strikeout/underline provisions from ATC-140 WG-2 effort and section numbers and acceptance may differ from the final release version of ASCE/SEI 41-23.

D.2 Design Example – 1: Single Cantilevered Shear Wall

D.2.1 Problem Statement:

For the cantilevered shear wall shown in Figure D.2-1, determine the soil bearing acceptance ratio at the foundation soil interface and the foundation acceptance at the Collapse Prevention performance level for the applied pseudo force demands for the following analysis and modeling options:

1. Soil foundation interface is modeled as a fixed base.
2. Soil foundation interface is modeled as a flexible base.

CODE: ASCE 41-23 AND ASCE 41-17

SPECIFICATIONS:

Allowable Soil pressure (D + L): q_{allow} = 3 ksf

Existing Concrete Strength: f'_c = 3,000 psi.

Existing Steel Strength f_y = 40,000 psi.

GIVEN:

Footing dimensions:

B_f = 5.0 ft

L_f = 40 ft

$A_f = B_f L_f$ = (5)(40) = 200 ft^2

LOADING:

M_{OT} = 30,000 kip-ft

P_D = 300.0 kips (Dead, includes weight of the footing)

P_L = 50.0 kips (0.25 unreduced live load)

P_E = 0.0 kips

MAXIMUM AXIAL DEMAND CAPACITY RATIO:

DCR_{max} = 1.0 (No reduction in seismic axial load demands to the wall due to superstructure yielding)

KNOWLEDGE FACTOR:

κ = 1.0

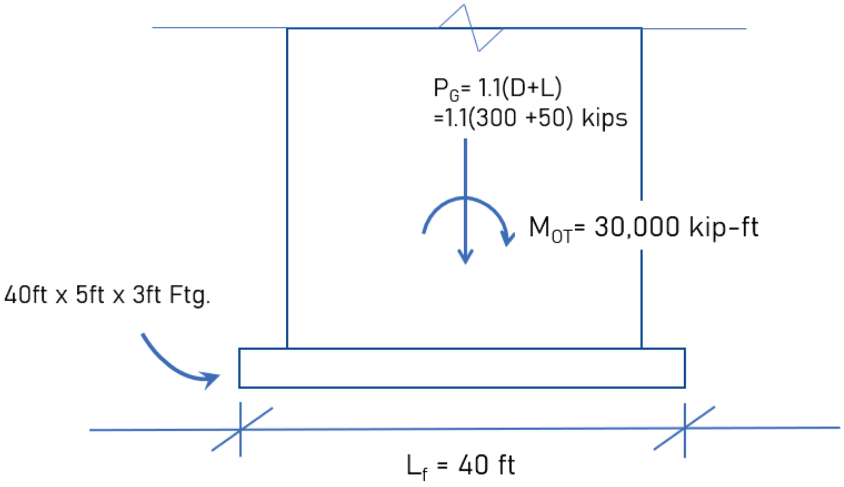


Figure D.2-1 Cantilever shear wall on a 40´x5´x3´ spread footing

D.2.2 CASE 1: Soil Foundation Interface Modeled as a Fixed Base

D.2.2.1 SOLUTION (PROPOSED PROVISIONS (ASCE/SEI 41-23)

D.2.2.1.1 Soil Strength for short term seismic loads:

$q_{cDA} = 2(q_c) = 2(3)(q_{allow}) = 6(3) = 18\text{ksf}$ (ASCE/SEI 41-23 *Eq.* 8-9)

D.2.2.1.2 Determination of Foundation Moment Capacity

$$M_{CE} = \frac{P_{UF}L_f}{2}\left(1 - \frac{q}{q_{cDA}}\right) kip - ft \qquad (\text{ASCE/SEI } 41 - 23 \;\; Eq. 8 - 12)$$

Where:

Axial load demand:

$$P_{UF} = P_G + \frac{P_E}{DCR_{max}} = \; 1.1(300 + 50) + \frac{0}{1.0} = 385\; kips \qquad (\text{ASCE/SEI } 41 - 23 \; Eq. 8 - 13)$$

Soil bearing pressure:

$$q = \frac{P_{UF}}{B_f L_f} = \frac{385}{200} = 1.925\; ksf$$

Foundation Moment Capacity

$$M_{CE} = \frac{P_{UF}L_f}{2}\left(1 - \frac{q}{q_{cDA}}\right) = \frac{385 * 40}{2}\left(1 - \frac{1.925}{18}\right) = 6876.5\; kip - ft$$

D.2.2.1.3 Acceptance Criteria Soil Bearing and Overturning, ASCE/SEI 41-23, Sec. 8.4.4.1.1.3.1

The component ductility or m-factors for soil bearing and overturning are given in ASCE/SEI 41-23 Table 8-4.

$m_{CP} = 4.0$

Acceptance Criteria, soil bearing, from ASCE/SEI 41 – 23 Eq. 8-21 is given as:

$$M_{OT} \leq m\kappa M_{CE} \qquad (\text{ASCE/SEI } 41 - 23 \;\; Eq. 8 - 21)$$

Rewriting Eq. 8-21 in terms of an Acceptance Ratio (AR)

$$AR = \frac{M_{OT}}{m\kappa M_{CE}} = \frac{30{,}000}{4 * 1 * 6876.5} = 1.091$$

D.2.2.2 SOLUTION (ASCE 41-17)

D.2.2.2.1 Soil Strength

Use of upper bound strenght is permitted per (ASCE/SEI 41-17 §8.4.2.3.1):

$q_c = (1+ C_v)(q_c) = (1+1)(3)(q_{allow}) = 6(3) = 18\text{ksf}$

D.2.2.2.2 Foundation Moment Capacity

$$M_{CE} = \frac{P_{UD}L_f}{2}\left(1 - \frac{q}{q_c}\right) kip - ft \qquad (\text{ASCE/SEI } 41 - 17\ Eq. 8 - 10)$$

D.2.2.2.3 Foundation Moment Capacity and Acceptance Criteria – When actions from seismic and gravity are additive

m-factor (overturning compression, Sec 8.4.2.3.2.1)

$m_{CP} = 4.0$

Axial load demand, using the compression load combiation (ASCE/SEI 41-23 Eq. 7-1):

$$P_{UD} = P_G + \frac{P_E}{DCR} = 1.1 * 350 + \frac{0}{1.0} = 385\ kips$$

Soil bearing pressure:

$$q = \frac{P_{UD}}{A_f} = \frac{385}{200} = 1.925\ ksf$$

$$M_{CE} = \frac{P_{UD}L_f}{2}\left(1 - \frac{q}{q_c}\right) = \frac{385 * 40}{2}\left(1 - \frac{1.925}{18}\right) = 6876.5\ kip - ft$$

Acceptance Criteria, soil bearing:

$$m\kappa Q_{CE} > Q_{UD} \qquad ASCE/SEI\ 41 - 17\ (Eq.\ 7 - 36)$$

$$Q_{CE} = M_{CE} \qquad ASCE/SEI\ 41 - 17\ (Sec. 8.4.2.3.2.1)$$

$$Q_{UD} = M_{UD} \qquad ASCE/SEI\ 41 - 17\ (Eq. 7 - 34)$$

Acceptance Ratio (AR)

$$AR = \frac{M_{UD}}{m\kappa M_{CE}} = \frac{30{,}000}{4 * 1 * 6876.5} = 1.091$$

D.2.2.2.4 Foundation Moment Capacity and Acceptance Criteria – When actions from seismic forces and gravity loads are counteracting

Axial load demand, when load combiation dead load is counteractive ASCE/SEI 41-17 (Eq. 7-2):

$$P_{UD} = P_G + \frac{P_E}{DCR} = 0.9 * 300 + \frac{0}{1.0} = 270\ kips$$

Soil bearing pressure:

$$q = \frac{P_{UD}}{A_f} = \frac{270}{200} = 1.35\ ksf$$

Moment Capacity:

$$M_{CE} = \frac{P_{UD} L_f}{2}\left(1 - \frac{q}{q_c}\right) = \frac{270 * 40}{2}\left(1 - \frac{1.35}{18}\right) = 4995\ kip - ft$$

Acceptance Criteria, soil bearing

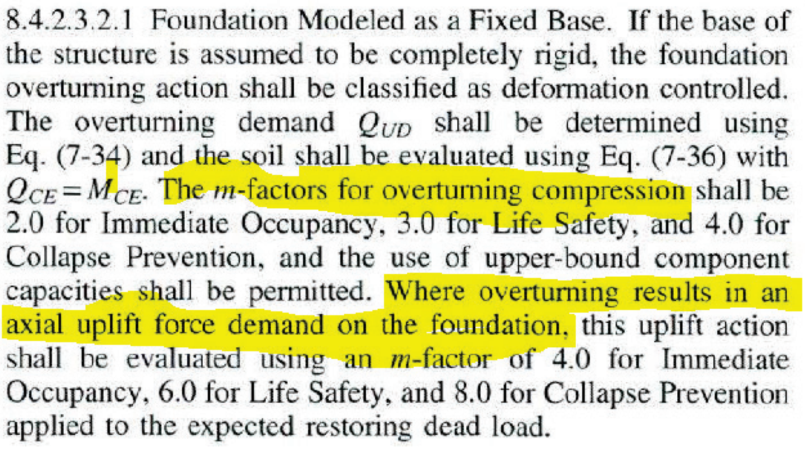
$m\kappa Q_{CE} > Q_{UD}$ $ASCE\ 41 - 17\ (Eq.\ 7 - 36)$

$Q_{CE} = M_{CE}$ $ASCE\ 41 - 17\ (Sec. 8.4.2.3.2.1)$

$Q_{UD} = M_{UD}$ v $ASCE\ 41 - 17\ (Eq. 7 - 34)$

Acceptance Ratio (AR)

The code (ASCE/SEI 41-17 §8.4.2.3.2.1) is not specific which m-factor is to be used in this case, therefore the AR is shown for two possible options. One using m-factor for compression, and the other, the m-factor for uplift.



8.4.2.3.2.1 Foundation Modeled as a Fixed Base. If the base of the structure is assumed to be completely rigid, the foundation overturning action shall be classified as deformation controlled. The overturning demand Q_{UD} shall be determined using Eq. (7-34) and the soil shall be evaluated using Eq. (7-36) with $Q_{CE} = M_{CE}$. The *m*-factors for overturning compression shall be 2.0 for Immediate Occupancy, 3.0 for Life Safety, and 4.0 for Collapse Prevention, and the use of upper-bound component capacities shall be permitted. Where overturning results in an axial uplift force demand on the foundation, this uplift action shall be evaluated using an *m*-factor of 4.0 for Immediate Occupancy, 6.0 for Life Safety, and 8.0 for Collapse Prevention applied to the expected restoring dead load.

Option 1: m_{CP} = 4.0, overturning compression

$$AR = \frac{M_{UD}}{m\kappa M_{CE}} = \frac{30{,}000}{4 * 1 * 4995} = 1.502$$

Option 2: m_{CP} = 8.0 when overturning results in axial uplift force demand

$$AR = \frac{M_{UD}}{m\kappa M_{CE}} = \frac{30{,}000}{8 * 1 * 4995} = 0.751$$

D.2.2.3 RESULTS COMPARISON:

Acceptance Ratio: Soil Foundation Interface Modeled as a Fixed Base

Load Combination	ASCE/SEI 41-23 (Proposed)	ASCE/SEI 41-17
Compression (Eq. 7-1)	1.091	1.091
Uplift (Eq. 7-2)	Not calculated	Option 1 – 1.502
	-	Option 2 – 0.751

D.2.3 CASE 2: Soil Foundation Interface Modeled as a Flexible Base

D.2.3.1 SOLUTION (PROPOSED PROVISIONS, ASCE/SEI 41-23)

D.2.3.1.1 Soil Strength for short term seismic loads:

$q_{cDA} = 2(q_c) = 2(3)(q_{allow}) = 6(3) = 18$ksf ASCE/SEI 41 – 23 (*Eq.* 8-9)

D.2.3.1.2 Determination of Foundation Moment Capacity

$$M_{CE} = \frac{P_{UF} L_f}{2}\left(1 - \frac{q}{q_{cDA}}\right) kip - ft \qquad (\text{ASCE/SEI } 41 - 23 \;\; Eq. 8 - 12)$$

Where:

Axial load demand:

Load combiation compression ASCE/SEI 41-23 (Eq. 7-1):

$$P_{UF} = P_G + \frac{P_E}{DCR_{max}} = 1.1(300 + 50) + \frac{0}{1.0} = 385\ kips \qquad (\text{ASCE/SEI } 41 - 23\ Eq. 8 - 13)$$

Soil bearing pressure:

$$q = \frac{P_{UF}}{B_f L_f} = \frac{385}{200} = 1.925\ ksf$$

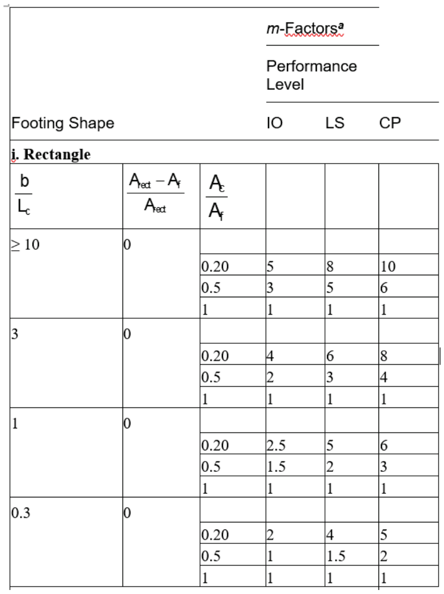
Foundation Moment Capacity

$$M_{CE} = \frac{P_{UF} L_f}{2}\left(1 - \frac{q}{q_{cDA}}\right) = \frac{385 * 40}{2}\left(1 - \frac{1.925}{18}\right) = 6876.5\ kip - ft$$

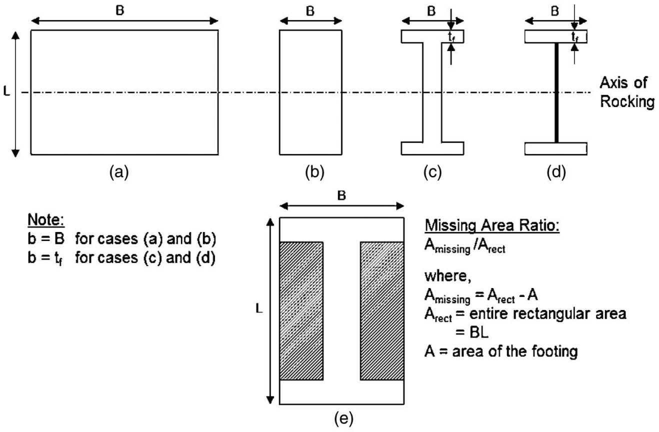
D.2.3.1.3 Foundation Acceptance

Determination of m-factor Table 8-7

m-factor (overturning compression) are obtained from ASCE/SEI Table 8-7, Figure D.2-1.



Footing Shape			*m*-Factors[a] Performance Level IO	LS	CP
i. Rectangle					
$\frac{b}{L_c}$	$\frac{A_{rect} - A_f}{A_{rect}}$	$\frac{A_c}{A_f}$			
≥ 10	0				
		0.20	5	8	10
		0.5	3	5	6
		1	1	1	1
3	0				
		0.20	4	6	8
		0.5	2	3	4
		1	1	1	1
1	0				
		0.20	2.5	5	6
		0.5	1.5	2	3
		1	1	1	1
0.3	0				
		0.20	2	4	5
		0.5	1	1.5	2
		1	1	1	1



Note:
b = B for cases (a) and (b)
b = t_f for cases (c) and (d)

Missing Area Ratio:
$A_{missing}/A_{rect}$
where,
$A_{missing} = A_{rect} - A$
A_{rect} = entire rectangular area = BL
A = area of the footing

Figure D.2-1 m-Factors from ASCE/SEI 41-23 Table 8-7

$b = B_f = 5$ ft

$$L_c = \frac{P_{UF}}{B_f q_{cDA}} = 4.278\ ft$$

$$A_c = \frac{P_{UF}}{q_{cDA}} = 21.389;\ ft^2$$

$$A_{rect} = A_f = 200;\ ft^2$$

$$A_{miss} = \frac{A_{rect} - A_f}{A_{rect}} = 0;\ ft^2$$

$$b_{ratio} = \frac{b}{L_c} = 1.169;$$

$$A_{c\_ratio} = \frac{A_c}{A_f} = \frac{21.389}{200} = 0.107;$$

Interpolating the m-factor from Table 8-7

$$m_{CP} = 6 + (8-6)\frac{(b_{ratio} - 1.0)}{(3.0 - 1.0)} = 6.169;$$

$m_{CP} = 6.169$

Acceptance Criteria, soil bearing Eq. 8-21

$$M_{OT} \leq m\kappa M_{CE} \qquad \text{ASCE/SEI } 41-23;\ Eq.8-21$$

Acceptance Ratio (AR)

$$AR = \frac{M_{OT}}{m\kappa M_{CE}} = \frac{30{,}000}{6.169 * 1 * 6876.5} = 0.707$$

D.2.3.2 SOLUTION (ASCE/SEI 41-17)

D.2.3.2.1 Soil Strength

upper bound (ASCE/SEI 41-17 §8.4.2.3.2.2)

$q_c = (1 + C_v)(q_c) = (1+1)(3)(q_{allow}) = 6(3) = 18$ksf

D.2.3.2.2 Foundation Moment Capacity

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

D.2.3.2.3 Foundation Moment Capacity and Acceptance Criteria – When actions from seismic and gravity are additive

Axial load demand

Using the compression load combiation ASCE/SEI (Eq. 7-1)

$$P_{UD} = P_G + \frac{P_E}{DCR} = 1.1(300 + 50) + \frac{0}{1.0} = 385\ kips$$

Soil Bearing Pressure

$$q = \frac{P_{UD}}{A_f} = \frac{385}{200} = 1.925\ ksf$$

m-factor (overturning compression, §Sec 8.4.2.3.2.2)

m_{CP} = 6.169 from above since P_{UF} = P_{UD}.

$$M_{CE} = \frac{P_{UD}L_f}{2}\left(1 - \frac{q}{q_c}\right) = \frac{385 * 40}{2}\left(1 - \frac{1.925}{18}\right) = 6876.5\ kip - ft$$

Acceptance Criteria, soil bearing:

$$m\kappa Q_{CE} > Q_{UD} \qquad ASCE/SEI\ 41 - 17\ (Eq.\ 7 - 36)$$

Acceptance Ratio (AR)

$$AR = \frac{M_{UD}}{m\kappa M_{CE}} = \frac{30{,}000}{6.169 * 1 * 6876.5} = 0.707$$

D.2.3.2.4 Foundation Moment Capacity and Acceptance Criteria – When actions from seismic forces and gravity loads are counteracting

Axial load demand

When load combiation dead load is counteractive ASCE/sEI (Eq. 7-2) applies

$$P_{UD} = P_G + \frac{P_E}{DCR} = 0.9 * 300 + \frac{0}{1.0} = 270\ kips$$

Soil bearing pressure:

$$q = \frac{P_{UD}}{A_f} = \frac{270}{200} = 1.35\ ksf$$

Moment Capacity:

$$M_{CE} = \frac{P_{UD}L_f}{2}\left(1 - \frac{q}{q_c}\right) = \frac{270 * 40}{2}\left(1 - \frac{1.35}{18}\right) = 4995\ kip - ft$$

Acceptance Criteria, soil bearing

Acceptance Ratio (AR)

The code is not specific which m-factor is to be used in this case, therefore the following are options are considered:

Option 1: m_{CP} = 6.169, overturning compression

$$AR = \frac{M_{OT}}{m\kappa M_{CE}} = \frac{30{,}000}{6.169 * 1 * 4995} = 0.97$$

Option 2: m_{CP} = 10.0 when overturning results in axial uplift force demand

$$AR = \frac{M_{OT}}{m\kappa M_{CE}} = \frac{30{,}000}{10 * 1 * 4995} = 0.60$$

D.2.3.3 RESULTS COMPARISON:

Acceptance Ratio: Soil Foundation Interface Modeled as a Flexible Base

Load Combination	ASCE 41-23 (Proposed)	ASCE 41-17
Compression (Eq. 7-1)	0.707	0.707
Uplift (Eq. 7-2)	Not calculated	Option 1 – 0.97
	-	Option 2 – 0.60

D.2.4 Summary:

Results using ASCE/SEI 41-17 and more conservative than the results using ASCE/SEI 41-23 if the compression m-values are use when seismic loads and gravity are counteraction. ASCE/SEI 41-17 and 41-23 give identical results if only the load combination where seismic and gravity are additive is used.

Case 1: Acceptance Ratio: Soil Foundation Interface Modeled as a Fixed Base

Load Combination	ASCE 41-23 (Proposed)	ASCE 41-17
Compression (Eq. 7-1)	1.091	1.091
Uplift (Eq. 7-2)	Not calculated	Option 1 – 1.502
	-	Option 2 – 0.751

Case 2: Acceptance Ratio: Soil Foundation Interface Modeled as a Flexible Base

Load Combination	ASCE 41-23 (Proposed)	ASCE 41-17
Compression (Eq. 7-1)	0.707	0.707
Uplift (Eq. 7-2)	Not calculated	Option 1 – 0.97
	-	Option 2 – 0.60

D.3 Design Example – 2: Braced Frame on Isolated and Combined Footings

D.3.1 Problem Statement:

A steel braced frame (Figure D.3-1) forms the seismic lateral force resisting system of three-story office building. Foundation demands are the pseudo seismic forces from a Linear Static Procedure. Determine the soil bearing acceptance ratio at the foundation soil interface and the foundation acceptance at the Collapse Prevention performance level assuming the following modeling options for the analysis of the superstructure:

1. Isolated footings, foundation soil interface is modeled as a fixed base.
2. Isolated footings, Foundation soil interface is modeled as a flexible base.
3. Footings interconnected by a 3' wide x 3' deep grade beam with soil interface modeled as a fixed base.
4. Footings interconnected by a 3' wide x 3' deep grade beam with soil interface modeled as a flexible base.

CODE: ASCE 41-23 and ASCE 41-17

SPECIFICATIONS:

Allowable Soil pressure (D + L): q_{allow} = 3 ksf

Existing Concrete Strength: f'_c = 3,000 psi.

Existing Steel Strength f_y = 40,000 psi.

Unit weight of concrete 150 pcf

GIVEN:

Footing dimensions:

B_f = 10.0 ft

L_f = 10 ft

Loading:

M_{OT} = 30,000 kip-ft

$P_{D\_Superstructure}$ = 300.0 kips (Dead)

P_L = 50.0 kips (0.25 Unreduced Live)

Maximum Axial Demand Capacity Ratio:

DCR_{max} = 2.0 given

Knowledge Factor:

κ = 1.0

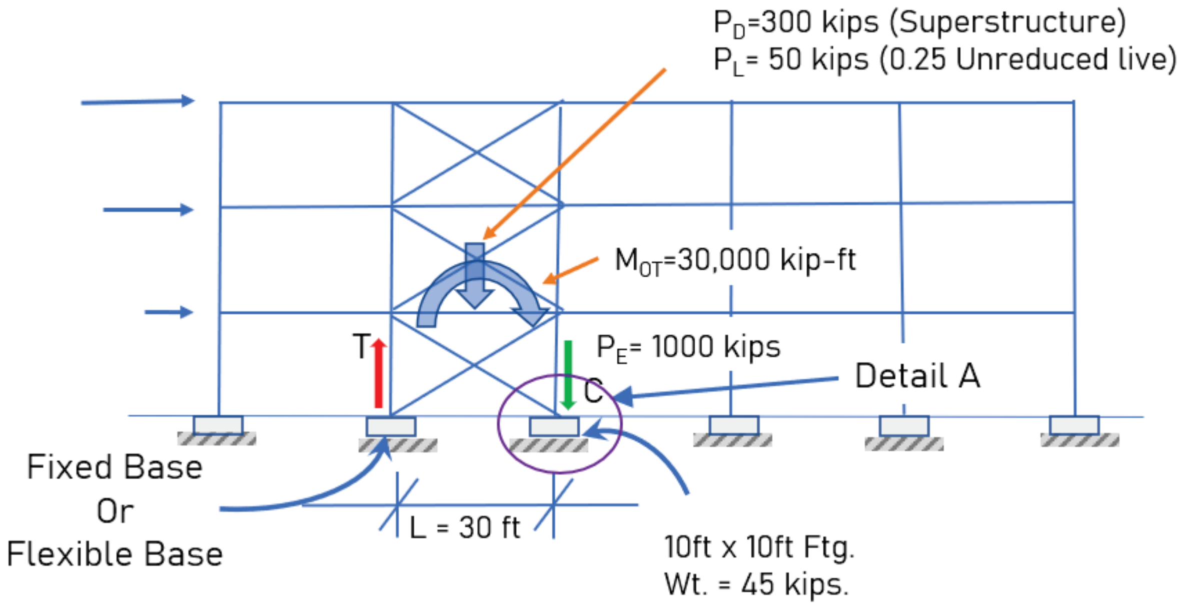


Figure D.3-2: Braced frame example.

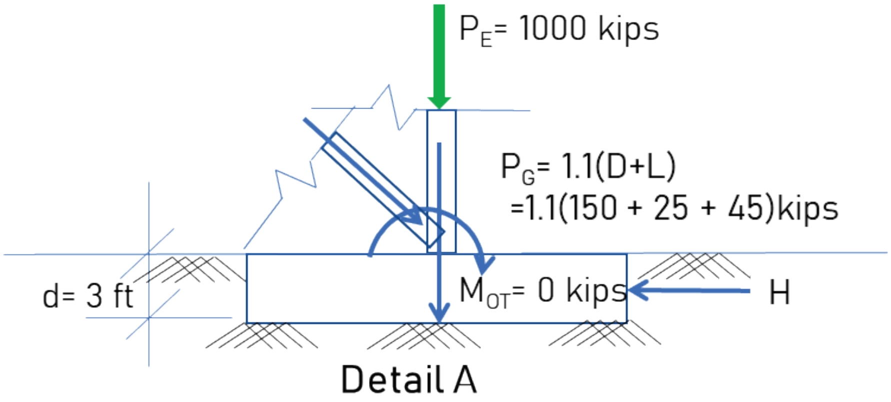


Figure D.3-2 Pseudo seismic demands on 10’x10 isolated footing under the braced frame.

D.3.2 CASE 1: Isolated Footings, Soil Foundation Interface Modeled as a Fixed Base

D.3.2.1 SOLUTION (PROPOSED PROVISIONS ASCE 41-23)

D.3.2.1.1 Soil Strength for short term seismic loads:

$q_{cDA} = 2(q_c) = 2(3)(q_{allow}) = 6(3) = 18\text{ksf}$ ASCE/SEI 41-23 (*Eq.* 8-9)

D.3.2.1.2 Determination of Axial load demand on the footing

Gravity load on each footing

$$P_{D\_superstructure} = 300\ kips$$

$$P_L = 50\ kips$$

$$P_{D\_Footing} = (10 \times 10\ \times\ 3)(0.150) =\ 45\ kips$$

$$P_G = \frac{300}{2} + \frac{50}{2} + 45 = 220\ kips$$

Seismic axial load (compression or uplift)

$$P_E = \frac{M_{OT}}{L'_f} = \frac{30{,}000}{30} = 1{,}000\ kips$$

Axial action compression load demand on the footing

$$P_{UF} = P_G + \frac{P_E}{DCR_{max}} = 1.1(220) + \frac{1000}{2} = 742\ kips \qquad \text{ASCE/SEI } 41-23\ (Eq. 8-13)$$

D.3.2.1.3 Acceptance criteria, axial compression:

Ductility m-factor multiplying soil bearing axial capacity

m_{CP} = 2.5 $\quad$ ASCE/SEI 41 – 23 Table 8-3

$$Acceptance\ Ratio\ (AR) = \frac{P_U}{m_{CP} q_{cDA} A_f} = \frac{742}{(2.5)(18)(100)} = 0.165$$

D.3.2.1.4 Acceptance criteria, axial action - uplift

m_{CP} = 8.0 $\quad$ ASCE/SEI 41 – 23 Table 8-3

$$P_D = \frac{300}{2} + 45 = 195\ kips$$

$$Acceptance\ Ratio\ (AR) = \frac{P_E}{0.9 m_{CP} P_D} = \frac{1000}{(0.9)(8)(195)} = 0.712$$

D.3.2.1.5 Governing Acceptance Ratio

Governing AR **=** 0.712

D.3.2.2 SOLUTION (ASCE/SEI 41-17)

hThere is non comparable solution in ASCE/SEI 41-17 when seismic axial loads and gravity are additiove for pure axial compression. There is only acceptance criteria for axial uplift and the acceptance ratios from ASCE/SEI 41-17 and from ASCE/SEI 41-23 is the same and is given as:

$$Acceptance\ Ratio\ (AR) = \frac{P_E}{0.9 m_{CP} P_D} = \frac{1000}{(0.9)(8)(195)} = 0.712 \quad \text{ASCE/SEI } 41-17\ \S 8.4.2.3.2.1$$

D.3.3 CASE 2: Isolated Footings, Foundation Interface Modeled as a Flexible Base

D.3.3.1 SOLUTION (PROPOSED PROVISIONS ASCE 41-23)

D.3.3.1.1 Acceptance criteria, Axial axial - compression:

Ductility m-factor multiplying soil bearing axial capacity

m_{CP} = 3.0 $\quad$ ASCE/SEI 41-23 Table 8-6

$$Acceptance\ Ratio\ (AR) = \frac{P_{UF}}{m_{CP} q_{cDA} A_f} = \frac{742}{(3.0)(18)(100)} = 0.137$$

D.3.3.1.2 Acceptance Criteria, Axial action - Uplift

m_{CP} = 10.0 ASCE/SEI 41-23 Table 8-6

$$P_D = \frac{300}{2} + 45 = 195\ kips$$

$$Acceptance\ Ratio\ (AR) = \frac{P_E}{0.9m_{CP}P_D} = \frac{1000}{(0.9)(10)(195)} = 0.57$$

D.3.3.1.3 Governing Acceptance Ratio

Governing AR **=** 0.57

D.3.3.2 SOLUTION (ASCE/SEI 41-17)

There is non comparable solution in ASCE/SEI 41-17 when seismic axial loads and gravity are additiove for pure axial compression. There is only acceptance criteria for axial uplift and the acceptance ratios from ASCE/SEI 41-17 and from ASCE/SEI 41-23 is the same and is given as:

$$Acceptance\ Ratio\ (AR) = \frac{P_E}{0.9m_{CP}P_D} = \frac{1000}{(0.9)(10)(195)} = 0.57 \quad \text{ASCE/SEI}\ \ 41-17\ \ \S 8.4.2.3.2.2$$

D.3.4 CASE 3: Footings interconnected by a 3’ x 3’ grade beam, Fixed Base

D.3.4.1 SOLUTION (PROPOSED PROVISIONS ASCE 41-23)

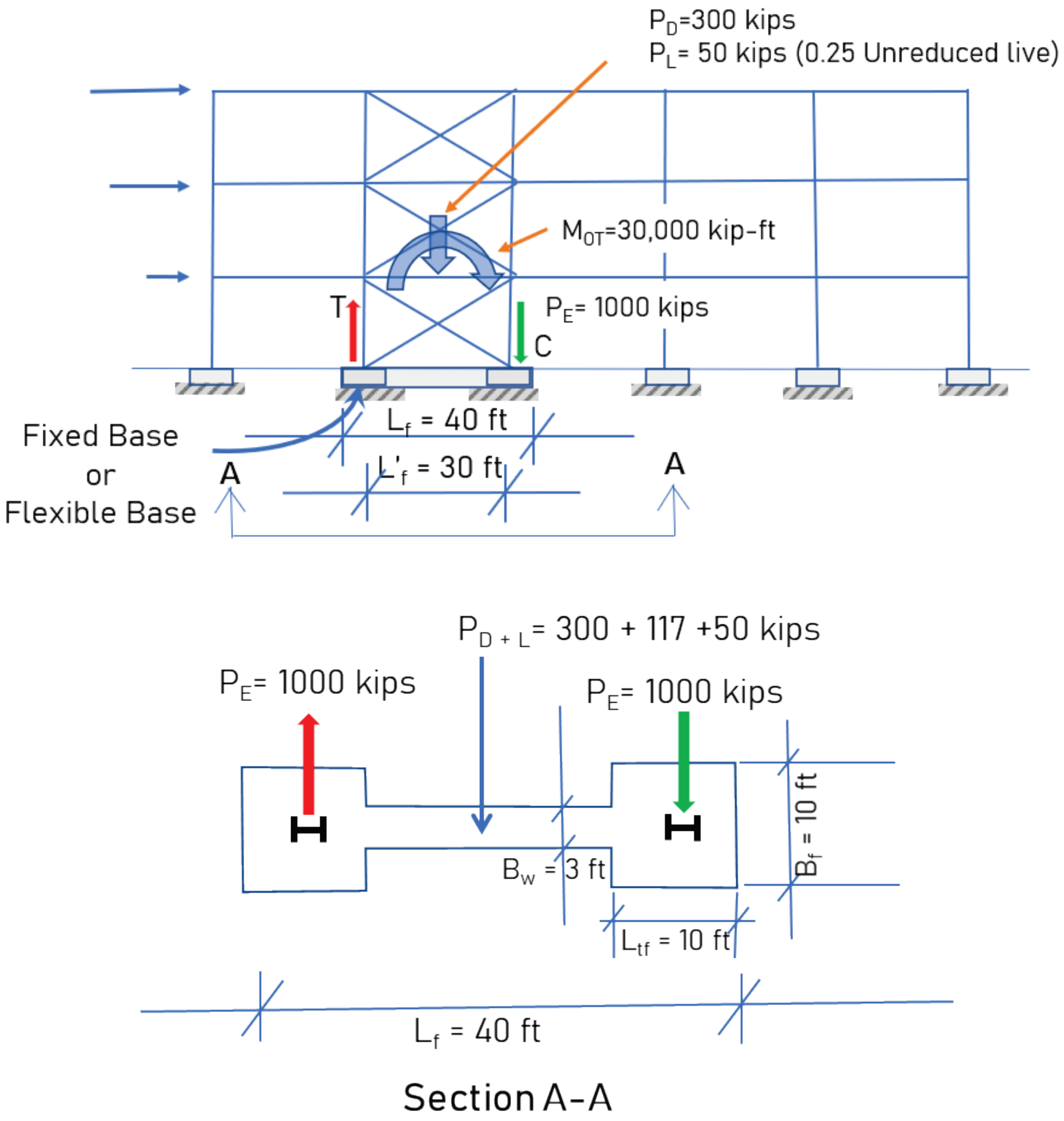


Figure D.3-3 **Pseudo seismic demands on 10’x10 isolated footings under the braced frame interconnected by a grade beam.**

D.3.4.1.1 Moment Capacity for Combined Axial and Moment action

Area of the combined footing

$$A_f = 2(B_f L_f) + (L_f - 2L_{tf})B_w = 260 ft^2$$

$$P_E = 0\ kips$$

$$B_w = 3\ ft$$

$$P_G = P_D + P_L + P_{Foundation}$$

$$P_{Foundation} = 2(45) + (3)(40 - 20)(3)(0.150) = 117\ \text{kips}$$

$$P_{D+L} = 300 + 50 + 117 = 467\ \text{kips}$$

Axial load demand:

$$P_{UF} = P_G + \frac{P_E}{DCR_{max}} = 1.1(467) + \frac{0}{2.0} = 513.7\ \text{kips} \qquad \text{ASCE/SEI 41} - 23\ \ (Eq. 8 - 13)$$

Soil bearing pressure:

$$q = \frac{P_{UF}}{A_f} = \frac{513.7}{260} = 1.976\ \text{ksf}$$

Foundation moment capacity

$$M_{CE} = \frac{P_{UF} L'_f}{2}\left(1 - \frac{q}{q_{cDA}}\right) = \frac{513.7 * 40}{2}\left(1 - \frac{1.976}{18}\right) = 9146.3\ \text{kip} - \text{ft}$$

D.3.4.1.2 Acceptance Criteria, soil bearing, from ASCE/SEI 41-23 Eq. 8-21 is given as:

$$M_{OT} \le m\kappa M_{CE} \qquad \text{ASCE/SEI 41} - 23\ (Eq. 8 - 21)$$

m_{CP} = 4.0 ASCE/SEI 41-23 (Table 8-4)

Rewriting Eq. 8-21 in terms of an Acceptance Ratio (AR)

$$AR = \frac{M_{OT}}{m\kappa M_{CE}} = \frac{30{,}000}{4 * 1 * 9146.3} = 0.82$$

D.3.4.2 SOLUTION ASCE 41-17: SOIL FOUNDATION INTERFACE MODELED AS A FIXED BASE

D.3.4.2.1 Soil strength upper bound

$q_c = (1 + C_v)(q_c) = (1+1)(3)(q_{allow}) = 6(3) = 18$ksf

D.3.4.2.2 Foundation Moment Capacity and Acceptance – When actions from seismic and gravity are additive

Axial load demand

Using the compression load combiation ASCE/SEI 41-17 (Eq. 7-1):

$$P_{UD} = P_G + \frac{P_E}{DCR} = 1.1 * 467 + \frac{0}{2.0} = 513.7\ kips$$

Soil bearing pressure

$$q = \frac{P_{UD}}{A_f} = \frac{513.7}{260} = 1.976\ ksf$$

Moment capacity

$$M_{CE} = \frac{P_{UD} L_f}{2}\left(1 - \frac{q}{q_c}\right) = \frac{513.7 * 40}{2}\left(1 - \frac{1.976}{18}\right) = 9146.3\ kip - ft$$

Acceptance Criteria, soil bearing:

$$m\kappa Q_{CE} > Q_{UD} \qquad ASCE\ 41 - 17\ (Eq.\ 7 - 36)$$

$$Q_{CE} = M_{CE} \qquad ASCE\ 41 - 17\ (Sec. 8.4.2.3.2.1)$$

$$Q_{UD} = M_{UD} \qquad ASCE\ 41 - 17\ (Eq. 7 - 34)$$

m-factor (overturning compression, Sec 8.4.2.3.2.1)

m_{CP} = 4.0

Acceptance Ratio (AR)

$$AR = \frac{M_{UD}}{m\kappa M_{CE}} = \frac{30{,}000}{4 * 1 * 9146.3} = 0.82$$

D.3.4.2.3 Foundation Moment Capacity and Acceptance – When actions from seismic forces and gravity loads are counteracting

Axial load demand

When load combiation dead load is counteractive ASCE/SEI 41-17 (Eq. 7-2)applies.

$$P_{UD} = P_G - \frac{P_E}{DCR} = 0.9 * (300 + 117) - \frac{0}{2.0} = 375.3\ kips$$

Soil bearing pressure:

$$q = \frac{P_{UD}}{A_f} = \frac{375.3}{260} = 1.443\ ksf$$

Moment Capacity:

$$M_{CE} = \frac{P_{UD}L_f}{2}\left(1 - \frac{q}{q_c}\right) = \frac{375.3 * 40}{2}\left(1 - \frac{1.443}{18}\right) = 6904.1\ kip - ft$$

Acceptance Criteria, soil bearing

$$m\kappa Q_{CE} > Q_{UD} \qquad ASCE/SEI\ 41-17\ (Eq.\ 7-36)$$

$$Q_{CE} = M_{CE} \qquad ASCE/SEI\ \ 41-17\ (Sec. 8.4.2.3.2.1)$$

$$Q_{UD} = M_{UD} \qquad ASCE/SEI\ 41-17\ (Eq. 7-34)$$

Acceptance Ratio (AR) The code is not specific which m-factor is to be used in this case, therefore the following are options are considered:

Option 1: m_{CP} = 4.0, overturning compression

$$AR = \frac{M_{OT}}{m\kappa M_{CE}} = \frac{30{,}000}{4 * 1 * 6904.1} = 1.086$$

Option 2: m_{CP} = 8.0 when overturning results in axial uplift force demand

$$AR = \frac{M_{OT}}{m\kappa M_{CE}} = \frac{30{,}000}{8 * 1 * 6904.1} = 0.543$$

D3.5 CASE 4: Footings interconnected by a 3’ x 3’ grade beam, Flexible Base

D.3.5.1 SOLUTION (PROPOSED PROVISIONS ASCE/SEI 41-23)

D.3.5.1.1 Acceptance Criteria Soil Bearing and Overturning, ASCE/SEI 41-23, Section 8.4.5.2.3.1.3

m-Factor for I-shaped footing

For the I-shaped footing, for the parameters and *m*-factors defined in Section 8.4.5.2.3.1

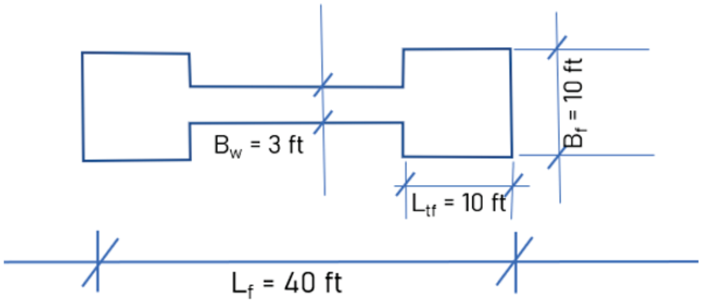
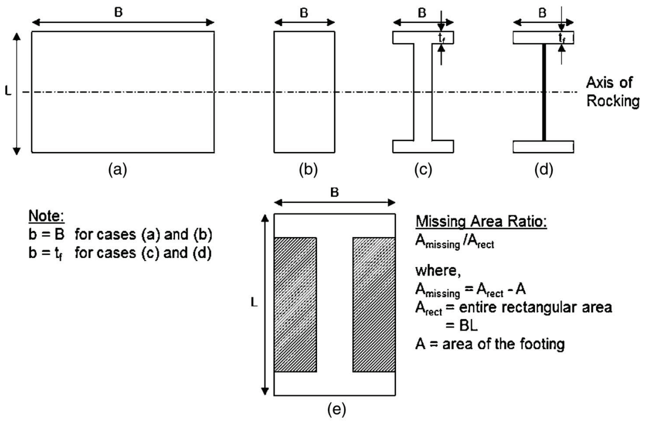


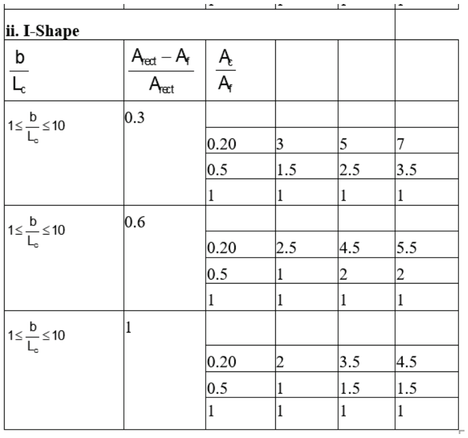
Figure D.3-4 Isolated footing interconnected by a grade beam



Note:
b = B for cases (a) and (b)
b = t_f for cases (c) and (d)

Missing Area Ratio:
$A_{missing}/A_{rect}$

where,
$A_{missing} = A_{rect} - A$
A_{rect} = entire rectangular area
= BL
A = area of the footing



ii. I-Shape					
$\frac{b}{L_c}$	$\frac{A_{rect}-A_f}{A_{rect}}$	$\frac{A_c}{A_f}$			
$1 \le \frac{b}{L_c} \le 10$	0.3				
		0.20	3	5	7
		0.5	1.5	2.5	3.5
		1	1	1	1
$1 \le \frac{b}{L_c} \le 10$	0.6				
		0.20	2.5	4.5	5.5
		0.5	1	2	2
		1	1	1	1
$1 \le \frac{b}{L_c} \le 10$	1				
		0.20	2	3.5	4.5
		0.5	1	1.5	1.5
		1	1	1	1

Figure D.3-5 m-Factors from ASCE/SEI 41-23 Table 8-7

$$L_c = \frac{P_U}{B_f q_{cDA}} = \frac{513.7}{10 * 18} = 2.854\ ft$$

$$A_c = \frac{P_U}{q_{cDA}} = 28.539;\ ft^2$$

$$A_{rect} = 400;\ ft^2$$

$$A_f = 260;\ ft^2$$

$$A_{miss\_ratio} = \frac{A_{rect} - A_f}{A_{rect}} = \frac{140}{400} = 0.35;$$

$$b_{ratio} = \frac{b}{L_c} = \frac{10}{2.854} = 3.504;\ \ (1 \leq \frac{b}{L_c} \leq 10)$$

$$A_{c\_ratio} = \frac{A_c}{A_f} = \frac{28.539}{260} = 0.11 < 0.2;$$

Interpolating the m-factor from Table 8-7

$$m_{CP} = 7 - (7 - 5.5)\frac{(A_{miss\_ratio} - 0.3)}{(0.6 - 0.3)} = 6.75;$$

$m_{CP} = 6.75$

Acceptance Criteria, soil bearing, from ASCE/SEI 41-23 Eq. 8-21 is given as:

$$M_{OT} \leq m\kappa M_{CE} \qquad \text{ASCE/SEI } 41 - 23\ (\ Eq. 8 - 21)$$

Rewriting Eq. 8-21 in terms of an Acceptance Ratio (AR)

$$AR = \frac{M_{OT}}{m\kappa M_{CE}} = \frac{30{,}000}{6.75 * 1 * 9146.3} = 0.486$$

D.3.5.2 SOLUTION ASCE 41-17: FOUNDATION INTERFACE MODELED AS A FLEXIBLE BASE

D.3.5.2.1 Foundation Moment Capacity – When actions from seismic and gravity are additive

$$M_{CE} = \frac{P_{UD}L_f}{2}\left(1 - \frac{q}{q_c}\right) = \frac{513.7 * 40}{2}\left(1 - \frac{1.976}{18}\right) = 9146.3\ kip - ft$$

Acceptance Criteria, soil bearing:

$$m\kappa Q_{CE} > Q_{UD} \qquad ASCE/SEI\ 41 - 17\ (Eq.\ 7 - 36)$$

$$Q_{CE} = M_{CE} \qquad ASCE/SEI\ 41 - 17\ (Sec. 8.4.2.3.2.1)$$

$$Q_{UD} = M_{UD} \qquad ASCE/SEI\ 41 - 17\ (Eq. 7 - 34)$$

Acceptance Ratio (AR)

$$AR = \frac{M_{OT}}{m\kappa M_{CE}} = \frac{30{,}000}{6.75 * 1 * 9146.3} = 0.486$$

D.3.5.2.2 Foundation Moment Capacity and Acceptance Criteria – When actions from seismic forces and gravity loads are counteracting

Axial load demand, when load combiation dead load is counteractive (Eq. 7-2):

$$P_{UD} = P_G - \frac{P_E}{DCR} = 0.9 * (300 + 117) - \frac{0}{2.0} = 375.3\ kips$$

Soil bearing pressure:

$$q = \frac{P_{UD}}{A_f} = \frac{375.3}{260} = 1.443\ ksf$$

Moment Capacity:

$$M_{CE} = \frac{P_{UD} L_f}{2}\left(1 - \frac{q}{q_c}\right) = \frac{375.3 * 40}{2}\left(1 - \frac{1.443}{18}\right) = 6904\ kip - ft$$

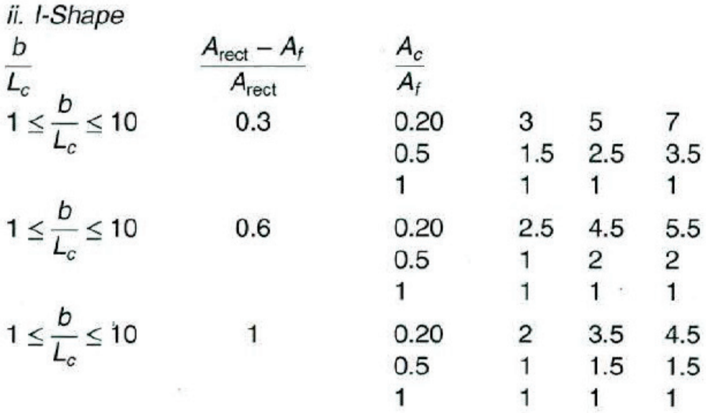
Acceptance Criteria, soil bearing

$$m\kappa Q_{CE} > Q_{UD} \qquad ASCE\ 41-17\ (Eq.\ 7-36)$$

$$Q_{CE} = M_{CE} \qquad ASCE\ 41-17\ (Sec. 8.4.2.3.2.1)$$

$$Q_{UD} = M_{UD} \qquad ASCE\ 41-17\ (Eq. 7-34)$$

Determination *of m-factor (ASCE 41-17, Sec. 8.4.2.3.2.2)*



ii. I-Shape

$\frac{b}{L_c}$	$\frac{A_{rect} - A_f}{A_{rect}}$	$\frac{A_c}{A_f}$			
$1 \leq \frac{b}{L_c} \leq 10$	0.3	0.20	3	5	7
		0.5	1.5	2.5	3.5
		1	1	1	1
$1 \leq \frac{b}{L_c} \leq 10$	0.6	0.20	2.5	4.5	5.5
		0.5	1	2	2
		1	1	1	1
$1 \leq \frac{b}{L_c} \leq 10$	1	0.20	2	3.5	4.5
		0.5	1	1.5	1.5
		1	1	1	1

$$L_c = \frac{P_{UD}}{B_f q_{cDA}} = \frac{375.3}{10 * 18} = 2.085\ ft$$

$$A_c = \frac{P_{UD}}{q_{cDA}} = 20.85;\ ft^2$$

$$A_{rect} = 400;\ ft^2$$

$$A_f = 260;\ ft^2$$

$$A_{miss} = \frac{A_{rect} - A_f}{A_{rect}} = 0.35$$

$$b_{ratio} = \frac{b}{L_c} = 4.796;\ \ (1 \leq \frac{b}{L_c} \leq 10)$$

$$A_{c\_ratio} = \frac{A_c}{A_f} = \frac{20.85}{260} = 0.08 < 0.2;$$

Interpolating the m-factor from Table 8-3

$$m_{CP} = 7 - (7 - 5.5)\frac{(A_{miss} - 0.3)}{(0.6 - 0.3)} = 6.75;$$

m_{CP} = 6.75

Acceptance Ratio (AR)

The code is not specific which m-factor is to be used in this case, therefore the following are options are considered:

Option 1: m_{CP} = 6.75, overturning compression

$$AR = \frac{M_{UD}}{m\kappa M_{CE}} = \frac{30{,}000}{6.75 * 1 * 6904} = 0.644$$

Option 2: m_{CP} = 10.0 when overturning results in axial uplift force demand

$$AR = \frac{M_{UD}}{m\kappa M_{CE}} = \frac{30{,}000}{10 * 1 * 6904} = 0.435$$

D.3.6 Summary

For both ASCE/SEI 41-17 and ASCE/SEI 41-23, the governing Acceptance Ratio (AR) is higher when the foundations are interconnected than is they were treated as isolated footings for both the fixed base and flexible base solutions. Similar to the results from Design Example 1, the results using ASCE/SEI 41-17 and more conservative than the results using ASCE/SEI 41-23 if the compression m-values are use when seismic loads and gravity are counteraction. ASCE/SEI 41-17 and 41-23 give identical results if only the load combination where seismic and gravity are additive is used.

Case 1: Acceptance Ratio: Isolated Footings, Foundation Interface Modeled as a Fixed Base

Load Combination	ASCE 41-23 (Proposed)	ASCE/SEI 41-17
Compression	0.165	N/A
Uplift	0.712	0.712
Governing LC	0.712	0.712

Case 2: Acceptance Ratio: Isolated Footings, Foundation Interface Modeled as a Flexible Base

Load Combination	ASCE 41-23 (Proposed)	ASCE/SEI 41-17
Compression	0.137	N/A
Uplift	0.57	0.57
Governing LC	0.57	0.57

Case 3: Acceptance Ratio: Combined Footings, Foundation Interface Modeled as a Fixed Base

Load Combination	ASCE 41-23 (Proposed)	ASCE/SEI 41-17
Compression	0.82	0.82
Uplift	N/A	Option 1 = 1.086
		Option 2 = 0.543
Governing LC	**0.82**	**0.82 or 1.086**

Case 4: Acceptance Ratio: Combined Footings, Foundation Interface Modeled as a Flexible Base

Load Combination	ASCE/SEI 41-23 (Proposed)	ASCE/SEI 41-17
Compression	0.486	0.486
Uplift	N/A	Option 1 = 0.644
		Option 2 = 0.435
Governing LC	0.486	0.486 or 0.644

D.4 Design Example – 3: Stair Tower on a Mat Foundation

D.4.1 Problem Statement:

A reinforced concrete shear wall stair tower of a five-story building with 12-foot floor to floor heights is supported on a 3´ thick mat foundation as shown in Figure D.4-1, with top of footing embedded 1' below the ground surface. The shear walls also act as bearing walls resisting gravity loads from the floor slabs in addition to the self-weight of the wall. The gravity dead and live loads are transferred from the walls to the foundation at the top of the Mat Foundation. The applied dead and live loads assumed as (D + 0.25 L) at the top of the 12" thick walls are 1.5 klf and 0.5 klf for the 8" thick walls.

The walls also resist overturning seismic demand from a fixed base analysis at the collapse prevention performance level of 52,800 kip-ft for overturning about the X- axis and 42,240 kip-ft for overturning about the Y- axis acting concurrently in the two orthogonal directions X- and Y-, for orthogonal load combinations of 100% and 30%. Pseudo seismic axial fluctuation on the wall can be ignored.

Determine the soil overturning acceptance using each of the following per Section 8.4.4.1.2 of ASCE/SEI 41-23:

1. Footing considered as an isolated footing.
2. Procedure 2 of Section 8.4.4.1.2.3 using spring stiffness values from Section 8.4.4.1.2.1 item 2
3. Procedure 1 of Section 8.4.4.1.2.3 using spring stiffness values from Section 8.4.4.1.2.1 item 3

CODE: ASCE 41-23

SPECIFICATIONS:

Allowable Soil pressure (D + L): q_{allow} = 3 ksf

Existing Concrete Strength: f'_c = 3,000 psi.

Existing Foundation Steel Strength f_y = 60,000 psi.

Unit weight of concrete 150 pcf

Standard penetration blow count N_{60} = 15

Atmospheric pressure p = 2.12 ksf

Site Class D

S_{XS} = 1.25 g

ν = 0.25

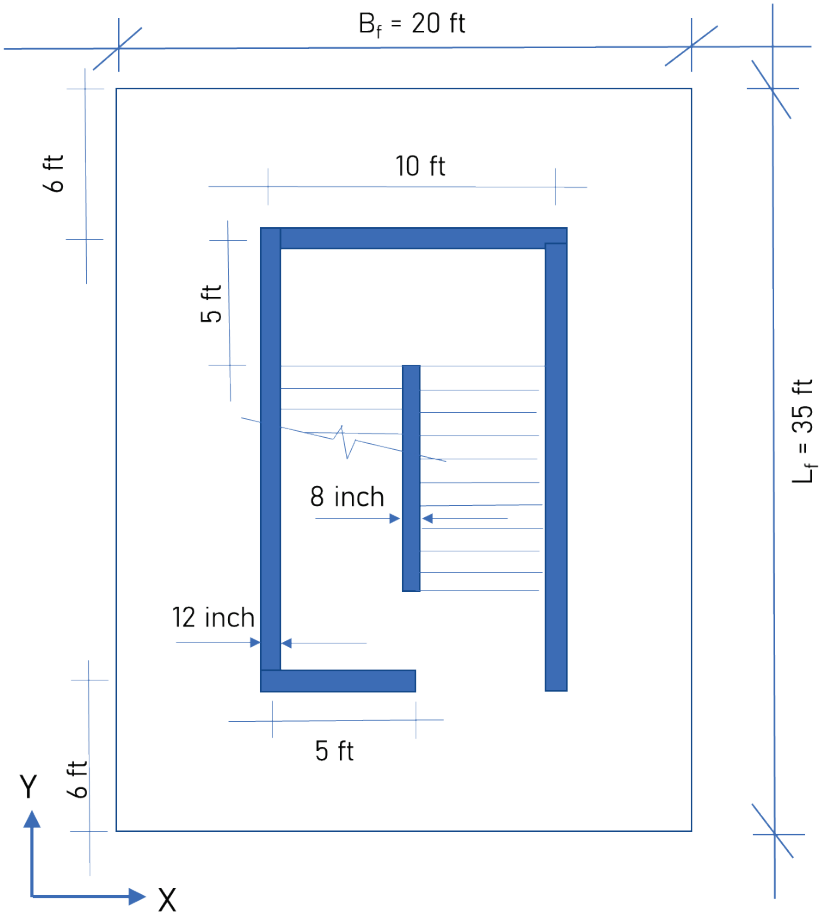


Figure D.4-3: Plan view of stair tower

D.4.2 Solution (Proposed Provisions ASCE 41-23):

GIVEN:

Footing dimensions:

B_f = 20.0 ft

L_f = 35 ft

Loading: Based on a pseudo seismic force demands including orthogonal load combinations of 100% and 30%.

$M_{OT,x}$ = 52,800 kip-ft

$M_{OT,y}$ = 42,240 kip-ft

Maximum Axial Demand Capacity Ratio:

DCR_{max} = 1.0 given

Knowledge Factor:

κ = 1.0

Ductility factor at collapse prevention performance level

$m_{C.P.}$ = 4.0

Soil Strength for short term seismic loads:

$q_{cDA} = 2(q_c) = 2(3)(q_{allow}) = 6(3) = 18\text{ksf}$ ASCE/SEI 41 – 23 (*Eq.* 8-9)

D.4.2.1 FOUNDATION CONSIDERED AS AN ISOLATED FOOTING

For the footing to be checked as an isolated footing, the footing is assumed as rigid, the axial load and moment demands on the footing are required to be applied at the centroid of the section. This requires the following steps:

- Determination of the axial load on the footing and the center of mass.
- Moment caused by gravity axial load eccentricity must be added to the seismic overturning moment of the footing without modification by the ductility of m-factor.
- When uniaxial overturning moments in both directions exceed $0.2mM_{CE}$ of the corresponding uniaxial moment capacity, bidirectional effects must be considered.

D.4.2.1.1 Determination of Mass Eccentricities at the Top of the Footing

For wall numbers shown in Figure D.4.-2, and the loads per floor tabulated in Table D.4-1, the center of mass and eccentricities are calculated as:

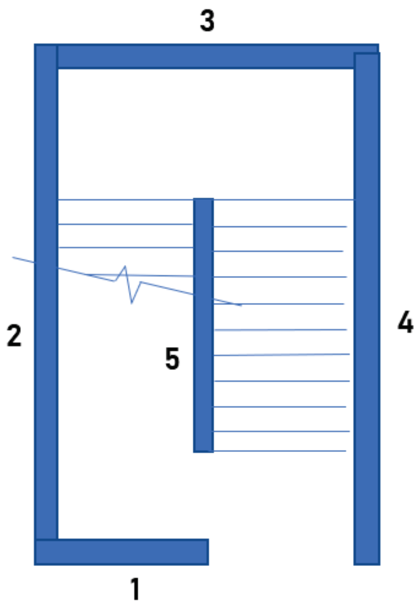


Figure D4-2: Wall numbers

Table D.4-1 Determination of the center of mass of the axial loads on the foundation

Wall Number	Unit weight per floor (klf)	Self weight (kips)	Length (ft)	Weight at top of Footing (klf)	Weight (kips	$X_{c.g.}$	Yc.g.	W*$X_{c.g.}$	W*Yc.g.
1	1.5	1.8	5	16.5	82.5	7.5	6	618.75	495
2	1.5	1.8	23	16.5	379.5	5	17.5	1897.5	6641.25
3	1.5	1.8	10	16.5	165	10	29	1650	4785
4	1.5	1.8	23	16.5	379.5	15	17.5	5692.5	6641.25
5	0.5	1.2	13	8.5	110.5	10	17.5	1105	1933.75
			74		**1117**			**10963.75**	**20496.25**

$X_{c.m.}$ = 10963.75/1117 = 9.82

$Y_{c.m.}$ = 20496.25/1117 = 18.25

e_x = 9.82 – 20/2 = -0.18 ft

$e_{y.}$ = 18.25 – 35/2 = 0.85 ft

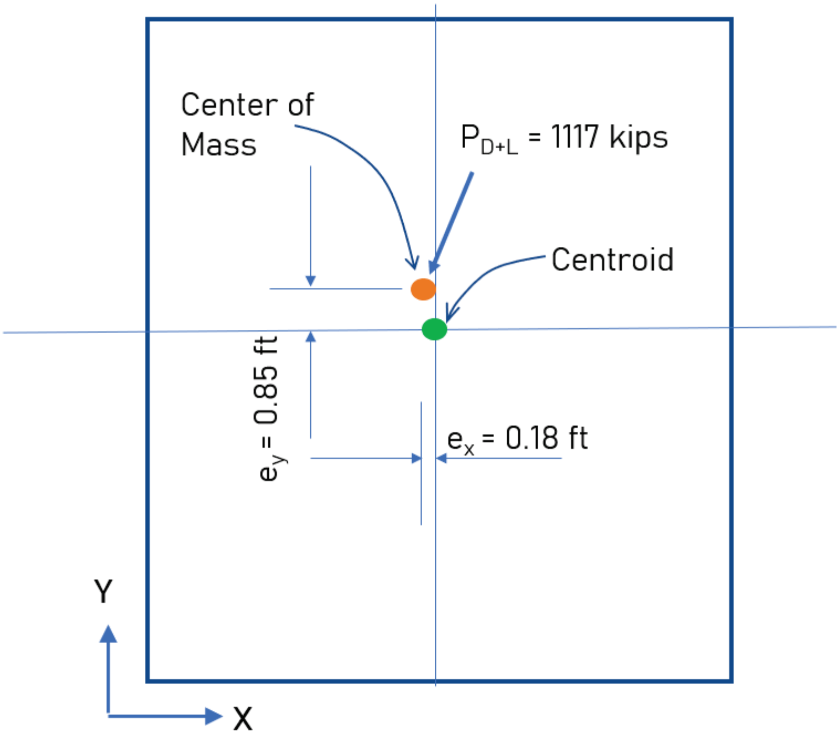


Figure D.4-3: Axial load at top of footing

D.4.2.1.2 Calculation of Inherent moment due to applied axial load

Axial load at the top of footing:

The center of mass of the gravity axial load at the top of the footing from Table D.4-1, is shown in Figure D.4-3. From ASCE/SEI 41-23 (Eq. 8-12), the axial load at the top of the footing $P_{U\_top\_of\_footing}$ is:

$$P_{U\_top\_of\_footing} = P_G + P_E/DCR_{max} \qquad \text{ASCE/SEI 41} - 23 \quad \text{(Eq. 8-9)}$$

$$= 1.1(1117) + 0 = 1228.7 \text{ kips}$$

Inherent Moment on footing due to eccentricities of the applied axial loads:

$$M_{x\_inherent} = -P_U \text{ x } e_{y.} = (1228.7)(0.85) = -1043.63 \text{ kip-ft}$$

$$M_{y\_inherent} = P_U \text{ x } e_{x.} = (1228.7)(-0.18) = -226.8 \text{ kip-ft}$$

Note: e_x and e_y were calculated without the 1.1 factor. It is assumed as negligible and is ignored.

D.4.2.1.3 Load demand at soil structure interface

Axial load at the soil structure interface

Weight of footing = 20 x 35 x 3 x 0.15 = 315 kips

P_{D+L} = 1117 + 315 = 1432 kips

$P_{UF} = 1.1P_G + P_E/DCR_{max}$ = 1.1(1432) + 0 = 1575.2 kips

D.4.2.1.4 Applied pseudo force moment

$M_{OT,x}$ = 52,800 kip-ft

$M_{OT,y}$ = 42,240 kip-ft

D.4.2.1.5 Check if Biaxial effects Need to be considered §8.4.4.1.1.3.1

q = 1575.2/(20)(35) = 2.25 ksf

$M_{CE,x\_uniaxial}$ = (1575.2)(35)/2(1-2.25/18) = 24,120 kip-ft

$M_{CE,y\_uniaxial}$ = (1575.2)(20)/2(1-2.25/18) = 13,783 kip-ft

$M_{OT,x}/m$ = 52,800/4 = 13,200 > 0.2(24,120) = 4,824 kip-ft

$M_{OT,y}/m$ = 42,240/4 = 10,560 > 0.2(13,783) = 2,756 kip-ft

Applied moments > 0.2 the m factor amplified moments in each direction; therefore, bi-directional effects need to be considered.

For rectangular footings, acceptance is based on either Eq. (8-20) or Eq. (8-21).

D.4.2.1.6 Acceptance based on ASCE/SEI 41-23 Eq. 8-20.

$$\left(\frac{M_{OT,x}}{m\kappa M_{CE,x}}\right)^2 + \left(\frac{M_{OT,y}}{m\kappa M_{CE,y}}\right)^2 \le 1.0 \qquad \text{ASCE/SEI 41} - 23\ (\text{Eq.}\ 8 - 20)$$

The applied overturning moment needs to be adjusted by the inherent moment of the Mat foundation, therefore

$$M_{OT,x} = M_{OT,x\_applied} + M_{x\_inherent}$$

And

$$M_{OT,y} = M_{OT,y\_applied} + M_{y\_inherent}$$

$$\left(\frac{52{,}800-(4)(1044)}{(4)(1)(24120)}\right)^2+\left(\frac{42{,}240-(4)(226.9)}{(4)(1)(13{,}783)}\right)^2=0.816\le 1.0$$

D.4.2.1.7 Acceptance based on ASCE/SEI 41-23 (Eq. 8-21).

$$M_{OT}\ \le m\kappa M_{CE} \qquad \text{ASCE/SEI 41}-23\ (Eq.8-21)$$

Where:

$$M_{OT}=\sqrt{(M_{OT.x})^2+\left(M_{OT,y}\right)^2} \qquad \text{ASCE/SEI 41}-23\ (Eq.8-19)$$

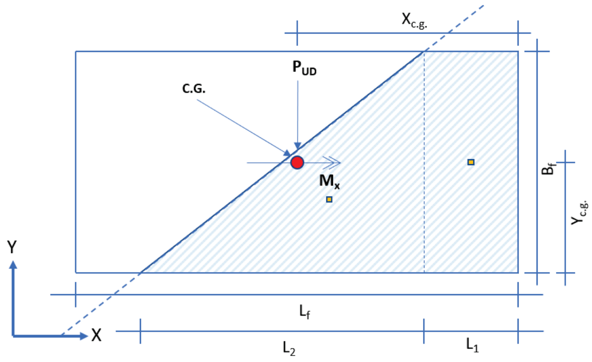
and

$$M_{CE}=\sqrt{\left(\frac{M_{OT.x}}{m}\right)^2+\left(M_{CE,y}\right)^2} \qquad \text{ASCE/SEI 41}-23\ (C8-5)$$

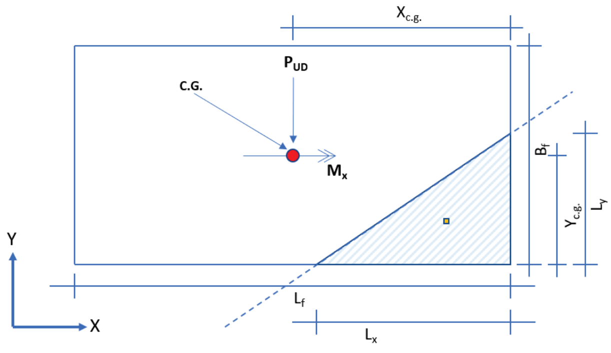
This requires the moment capacity to be determined considering bi-directional overturning moment action.

D.4.2.1.8 Foundation Moment Capacity for given axial load and weak axis moment

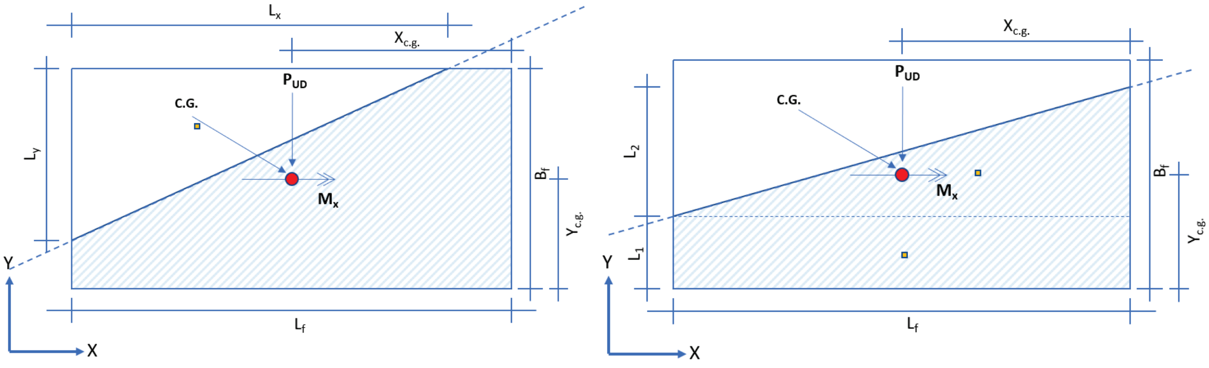
For an applied axial load P_U and an M_x or M_y moment on an isolated footing, the ultimate moment capacity in the orthogonal direction is determined by solving the equations of equilibrium of the applied load and the resisting soil pressure block under the footing. There are four distinct cases (Figure D.4-4) where a feasible solution is obtained for major axis moment for a given axial load and minor axis moment of the footing depending on where the resultant zero-pressure line intersects the footing edges.



Case 1



Case 2



Case 3 **Case 4**

FigureD.4-4 Soil pressure distribution cases where the Zero-Pressure line intersects footing edges

Case 1 – Zero pressure line intersects two opposite edges

If the zero-pressure line of the soil pressure block intersects two opposite edges of the footing as sown in Figure D.4-5, the ultimate moment in the orthogonal direction is given by the following expressions:

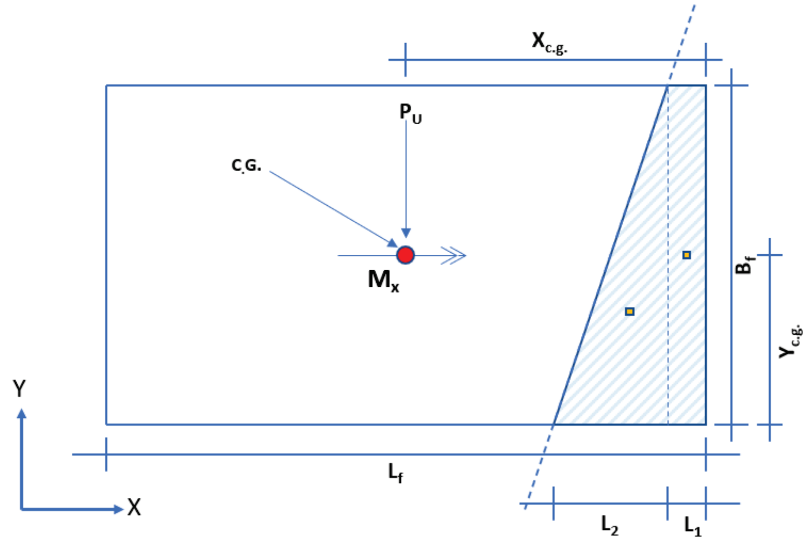


Figure D.4-5 Zero-Pressure Line intersects two opposite edges of the footing

$$M_{y,CE} = \frac{1}{2} q_c B_f \left\{ L_1 \left(X_{c.g.} - \frac{L_1}{2} \right) + \frac{1}{2} L_2 \left(X_{c.g.} - L_1 - \frac{L_2}{3} \right) \right\}$$

$$L_2 = \frac{6}{q_c B_f} \left\{ P_U - \frac{2\left(P_U Y_{c.g.} - M_x\right)}{B_f} \right\}$$

$$L_1 = \frac{P_U}{q_c B_f} - \frac{L_2}{2}$$

Interchanging the x and y coordinates to match the example, these equations can be written as:

$$M_{x,CE} = \frac{1}{2} q_c B_f \left\{ L_1 \left(Y_{c.g.} - \frac{L_1}{2} \right) + \frac{1}{2} L_2 \left(Y_{c.g.} - L_1 - \frac{L_2}{3} \right) \right\}$$

$$L_2 = \frac{6}{q_c B_f} \left\{ P_U - \frac{2\left(P_U X_{c.g.} - M_y\right)}{B_f} \right\}$$

$$L_1 = \frac{P_U}{q_c B_f} - \frac{L_2}{2}$$

The above equations are derived for the presumed actual demands on the footing accounting for overturning stability, therefore the pseudo force demands are divided by the *m*-factor for the desired performance level can be written as:

$M_y = M_{OT,y}/m_{C.P.} + M_{y\_inherent}$

$M_y = 42{,}240/4 - 226.8 = 10{,}333.2$ kip-ft

Substituting in the above equations,

$$L_2 = \frac{6}{18 \times 20} \left\{ 1575.2 - \frac{2(1575.2 \times 10 - (10333.2))}{20} \right\}$$

$L_2 = 17.22$ ft

$$L_1 = \frac{1575.2}{18 \times 20} - \frac{17.22}{2}$$

$L_1 = -4.23$ ft

Solution is infeasible.

Case 2 – Zero pressure line intersects two adjacent edges

If the zero-pressure line of the soil pressure block intersects two adjacent edges of the footing as sown in Figure D.4-6, the ultimate moment is the orthogonal direction is given by the following expressions:

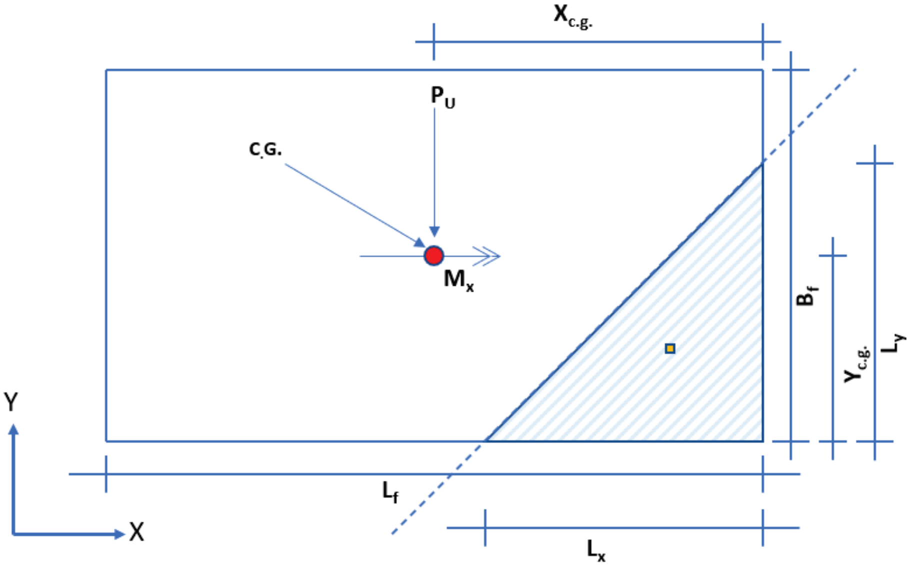


Figure D.4-6: Zero Pressure Line intersects two adjacent edges of the footing

$$M_{y,CE} = \frac{1}{2} q_{cDA} L_x L_y \left(X_{c.g.} - \frac{L_x}{3}\right)$$

$$L_y = 3\left(Y_{c.g.} - \frac{M_x}{P_U}\right)$$

$$L_x = 2\left(\frac{P_U}{q_c L_y}\right)$$

Since the major axis of overturning is about the X-axis, these equations can be written as:

$$M_{x,CE} = \frac{1}{2} q_c L_x L_y \left(Y_{c.g.} - \frac{L_y}{3}\right)$$

$$L_x = 3\left(X_{c.g.} - \frac{M_y}{P_U}\right)$$

$$L_y = 2\left(\frac{P_U}{q_c L_x}\right)$$

Where:

$M_y = M_{OT,y}/m_{C.P.} + M_{y\_inherent}$

M_y = 42,240/4 -226.8 = 10,333.2 kip-ft

$$L_x = 3\left(10.0 - \frac{10{,}333.2}{1575.2}\right) ft$$

L_x = 10.3 ft < 20 ft ok

$$L_y = 2\left(\frac{1575.2}{18 \times 10.3}\right)$$

= 17 ft < 35 feet Ok

$$M_{x,CE} = \frac{1}{2} q_c L_x L_y \left(Y_{c.g.} - \frac{L_y}{3}\right)$$

$$M_{x,CE} = \frac{1}{2} 18 \times 10.3 \times 17 \left(17.5 - \frac{17}{3}\right)$$

$M_{x,CE}$ = 18,661.5 kip-ft

$$M_{CE} = \sqrt{\left(\frac{M_{OT.x}}{m}\right)^2 + \left(M_{CE,y}\right)^2} \qquad \text{ASCE/SEI } 41 - 23\ (C8 - 5)$$

For our case the x and y are interchanged, therefore:

$$M_{CE} = \sqrt{\left(\frac{M_{OT.y}}{m}\right)^2 + \left(M_{CE,x}\right)^2}$$

$$M_{OT} = \sqrt{(52{,}800 - (4)(1044))^2 + (42{,}240 - (4)(226.9))^2} = 63{,}817\ ft - kip$$

and

$$M_{CE} = \sqrt{(10{,}333)^2 + (18{,}661)^2}$$

$$M_{CE} = 21{,}331\ ft - kips$$

D.4.2.1.9 Foundation acceptance criteria

Acceptance Ratio **(AR) =** 63,817/(4 x 21,331) **= 0.75**

Foundation is acceptable for soil bearing but must consider all load combinations for direction of applied overturning moment.

D.4.3.2 SOLUTION USING PROCEDURE 2 OF SECTION 8.4.4.1.2.3

D.4.3.2.1 Effective width of the footing for soil stiffness calculation, Section 8.4.4.1.2.1, item 2:

From Table D.4-1, the axial compression load demand on the footing can be represented as wall line loads at the top of the footing as shown in Figure D.4-7.

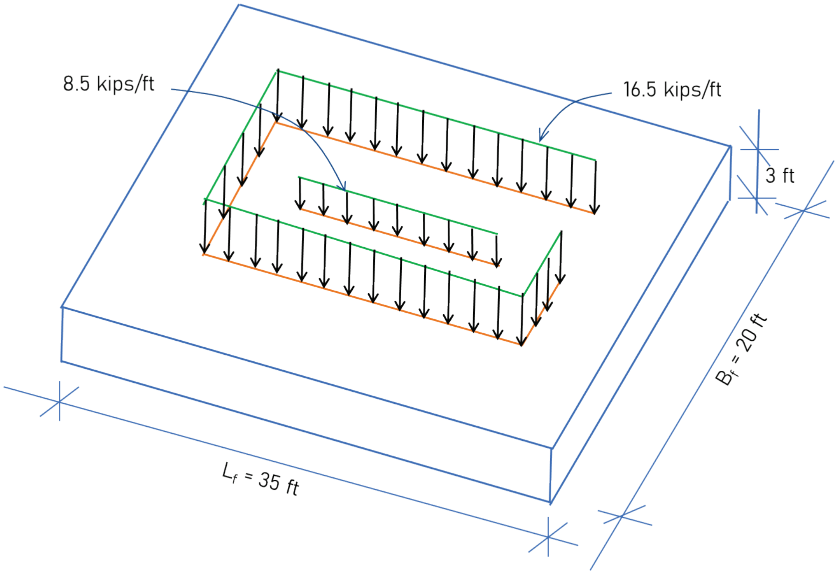


Figure D.4-7: Gravity load distribution at top of footing

Determination of effective width for the continuous 12' wall

Weight per unit length 16.5 kips/ft

Assume a width on all sides of the centerline of the wall of X'/2 feet as shown in Figure D.4-8.

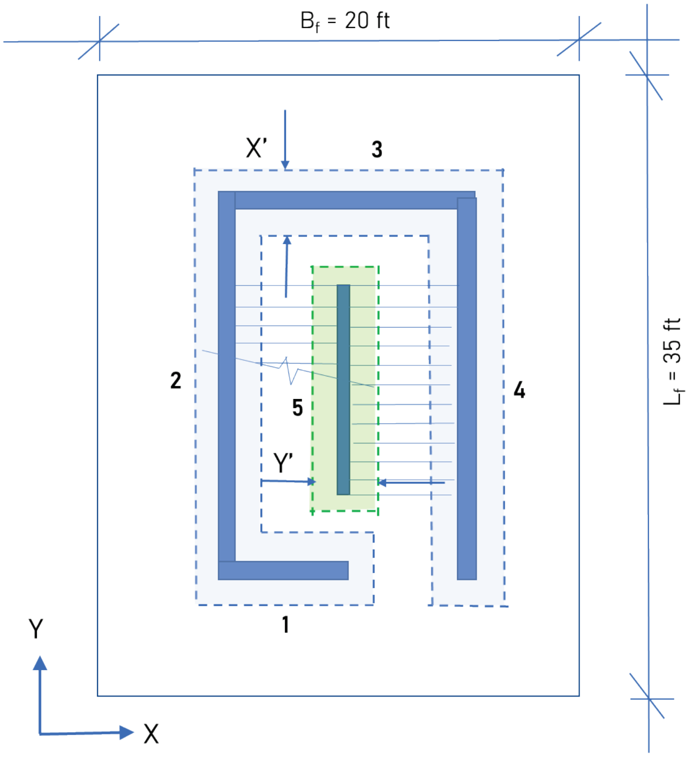


Figure D.4-8: Effective footing width required to support 1.5 times gravity load

Area required to support 1.5 times the gravity load is determined from:

A_{reqd} = 1.5(Axial Load)/q_{allow}

Or,

$X'(23+10+5+23+2X'/2) = 1.5 \times 16.5 \times (23 \times 2 + 10 + 5)/3.0$

Simplifying:

$X'^2 + 61X' + 503.25 = 0$

$X' = (-61 + \mathrm{sqrt}(61^2 + 4 \times 503.25)\}/2 = 7.36$ ft

Determination of the effective width of for the **middle** *wall*

Assume a width on all sides of the centerline of the wall of Y'/2 feet.

Therefore, area required to support 1.5 the applied load for an allowable bearing value q_{allow} of 3 ksf is determined as:

$(13 + Y') \times Y' = 1.5 \times (13 \times 8.5)/3$

$Y'^2 + 13\, Y' - 55.25 = 0$

$Y' = \{-13 + \mathrm{sqrt}(13^2 + 4 \times 55.25)\}/(2)$

$Y' = 2.9$ ft

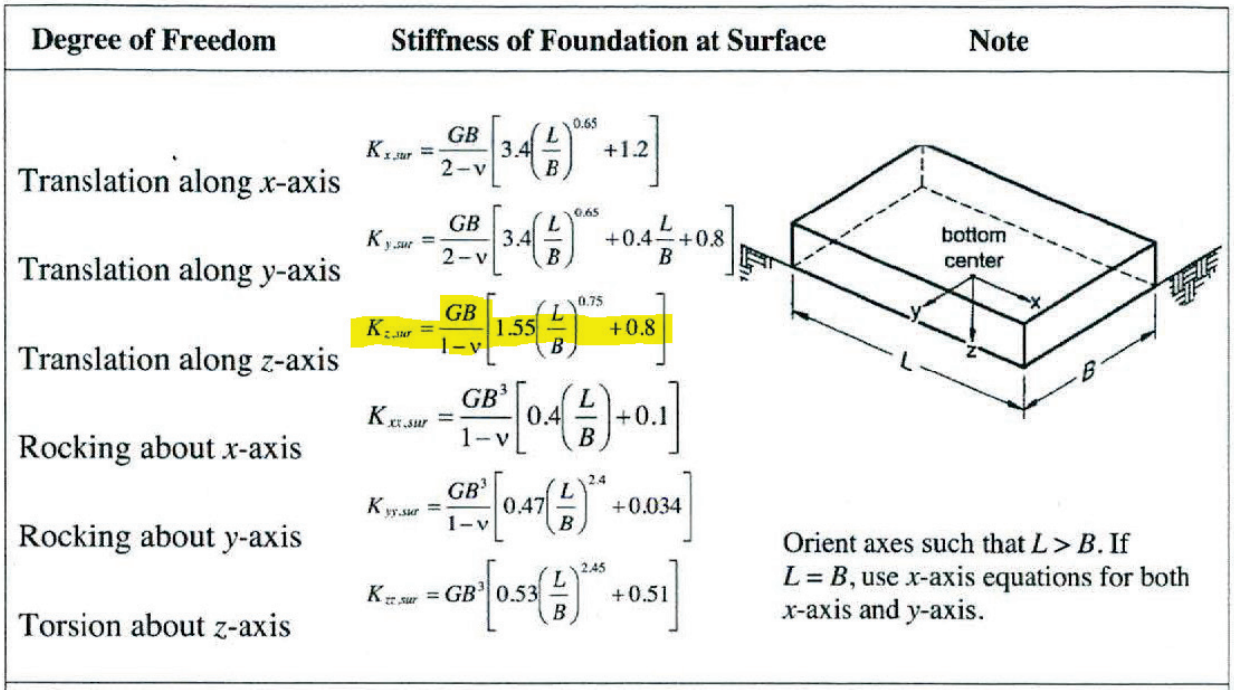
7.36/2 + 2.9/2 = 5.13 ft > 5 ft the distance between the centerline of the 12" and 8" wall, see Figure D.4-8.

Wall areas overlap, use an effective width of 10 + 7.36 = 17.36 ft

D.4.3.2.2 Determination of Soil Spring Stiffness

$L_f/B_f = 35/17.36 = 2.0 < 3.0$

Use spring stiffness from Figure 8-2 of ASCE 41-23



Degree of Freedom	Stiffness of Foundation at Surface	Note
Translation along x-axis	$K_{x,sur}=\frac{GB}{2-\nu}\left[3.4\left(\frac{L}{B}\right)^{0.65}+1.2\right]$	
Translation along y-axis	$K_{y,sur}=\frac{GB}{2-\nu}\left[3.4\left(\frac{L}{B}\right)^{0.65}+0.4\frac{L}{B}+0.8\right]$	
Translation along z-axis	$K_{z,sur}=\frac{GB}{1-\nu}\left[1.55\left(\frac{L}{B}\right)^{0.75}+0.8\right]$	
Rocking about x-axis	$K_{xx,sur}=\frac{GB^3}{1-\nu}\left[0.4\left(\frac{L}{B}\right)+0.1\right]$	
Rocking about y-axis	$K_{yy,sur}=\frac{GB^3}{1-\nu}\left[0.47\left(\frac{L}{B}\right)^{2.4}+0.034\right]$	
Torsion about z-axis	$K_{zz,sur}=GB^3\left[0.53\left(\frac{L}{B}\right)^{2.45}+0.51\right]$	Orient axes such that $L > B$. If $L = B$, use x-axis equations for both x-axis and y-axis.

Figure D.4.9 Figure 8.2 in ASCE/SEI 41-23, elastic soil stiffness at soil foundation interface.

$$K_{z\_sur} = \frac{GB}{(1-\nu)}\left[1.55\left(\frac{L}{B}\right)^{0.75} + 0.8\right]$$

Where B = 17.36 and L = 35. Adjustment for embedment is permitted.

Calculation of small strain soil Shear Modulus, G

$G_0 = 120p_a(N_{60})^{0.77}$ ksf ASCE/SEI 41-23 (*Eq.* 8-1)

$G_0 = (120)(2.12)\ (15)^{0.77}$ ksf

$G_0 = 2047$ ksf

$S_{XS} = 1.25$

$S_{XS}/2.5 = 0.5$

For Site class D from Table 8-2

$$\frac{G}{G_0} = 0.5 - (0.5-0.1)\frac{(0.5-0.4)}{(0.8-0.4)} = \ 0.4$$

$$G = 0.4(2047) = 819\ ksf$$

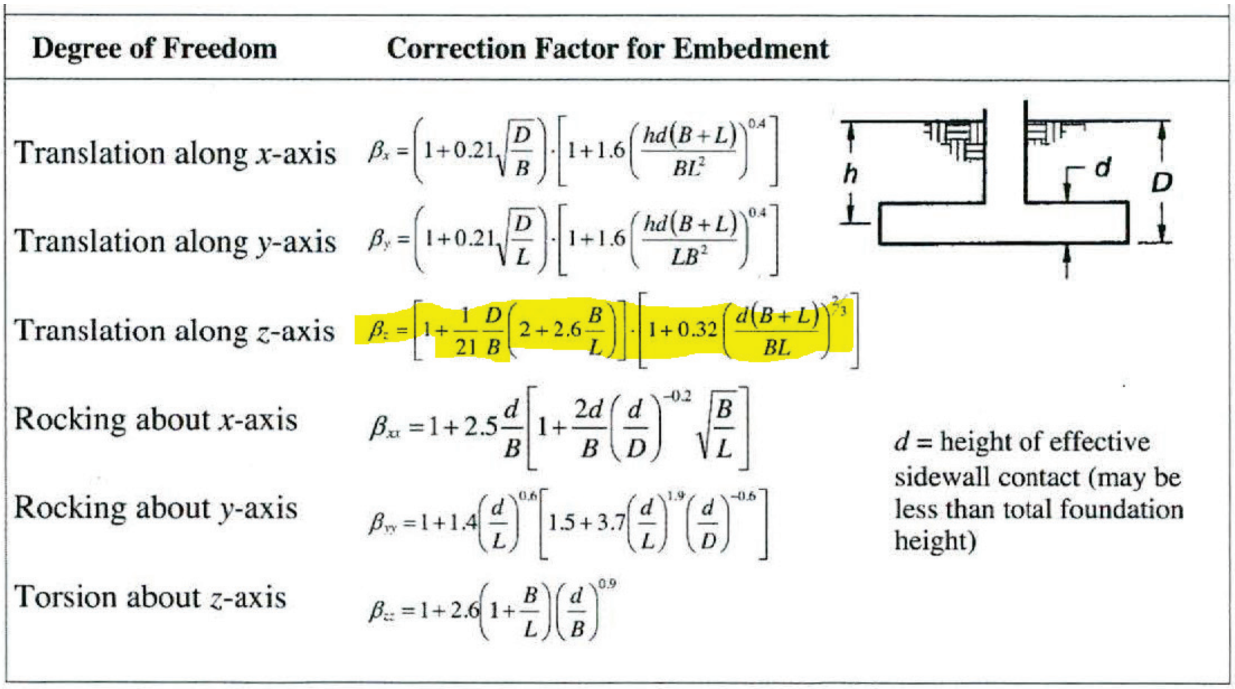
$$K_{z\_sur} = \frac{819 \times 17.36}{(1-0.25)}\left[1.55\left(\frac{35}{17.36}\right)^{0.75} + 0.8\right]$$

$$K_{z\_sur} = 64{,}864 \text{ k/ft}^3$$

or

$$k_{z\_sur} = 0.062 \text{ k/in}^3$$

Adjustment factor for embedment depth



Degree of Freedom	Correction Factor for Embedment
Translation along *x*-axis	$\beta_x = \left(1+0.21\sqrt{\frac{D}{B}}\right)\cdot\left[1+1.6\left(\frac{hd(B+L)}{BL^2}\right)^{0.4}\right]$
Translation along *y*-axis	$\beta_y = \left(1+0.21\sqrt{\frac{D}{L}}\right)\cdot\left[1+1.6\left(\frac{hd(B+L)}{LB^2}\right)^{0.4}\right]$
Translation along *z*-axis	$\beta_z = \left[1+\frac{1}{21}\frac{D}{B}\left(2+2.6\frac{B}{L}\right)\right]\cdot\left[1+0.32\left(\frac{d(B+L)}{BL}\right)^{2/3}\right]$
Rocking about *x*-axis	$\beta_{xx} = 1+2.5\frac{d}{B}\left[1+\frac{2d}{B}\left(\frac{d}{D}\right)^{-0.2}\sqrt{\frac{B}{L}}\right]$
Rocking about *y*-axis	$\beta_{yy} = 1+1.4\left(\frac{d}{L}\right)^{0.6}\left[1.5+3.7\left(\frac{d}{L}\right)^{1.9}\left(\frac{d}{D}\right)^{-0.6}\right]$
Torsion about *z*-axis	$\beta_{zz} = 1+2.6\left(1+\frac{B}{L}\right)\left(\frac{d}{B}\right)^{0.9}$

d = height of effective sidewall contact (may be less than total foundation height)

Figure D.4-10 Figure 8.2 in ASCE/SEI 41-23, Soil stiffness correction for embedment.

$$\beta_z = \left[1+\frac{1}{21}\frac{D}{B}\left(2+2.6\frac{B}{L}\right)\right]\left[1+0.32\left(\frac{d(B+L)}{BL}\right)^{\left(\frac{2}{3}\right)}\right]$$

$$\beta_z = \left[1+\frac{1}{21}\frac{(4)}{(17.36)}\left(2+2.6\frac{(17.36)}{35}\right)\right]\left[1+0.32\left(\frac{3(17.36+35)}{(17.36)(35)}\right)^{\left(\frac{2}{3}\right)}\right]$$

$$\beta_z = 1.17$$

$$k_z = (1.17)(0.062) = 0.072\ k/in^3$$

Alternatively:

$$k_{sv} = \frac{1.3G}{B_f(1-\nu)} \qquad \text{ASCE/SEI } 41-23\ (Eq. 8-22)$$

$$k_{sv} = \frac{1.3(819)}{17.36(1-0.25)} = 81.7\ k/ft^3$$

or

$$k_{sv} = 0.047 = k/in^3$$

Use $k_z = 0.072\ k/in^3$

D.4.3.2.3 Solution using Finite Element Modeling (ETABS):

Applied overturning loads to the footing:

For Procedure 2, soil does not resist tension, it is permitted to reduce the Pseudo seismic forces by the ductility factor m.

$M_{OT,x} / m_{CP}$ = 52,800/4 = 13,200 kip-ft

$M_{OT,y} / m_{CP}$ = 42,240/4 = 10,560 kip-ft

Where:

m_{CP} = 4.0, at the collpase prevention level from ASCE/SEI 41-23, Table 8-5.

The adjusted applied loads to the model resulting in the same overturning moment at the top of the footing are shown in Figure D.4-11 using the applied loads from Table D.4-2.

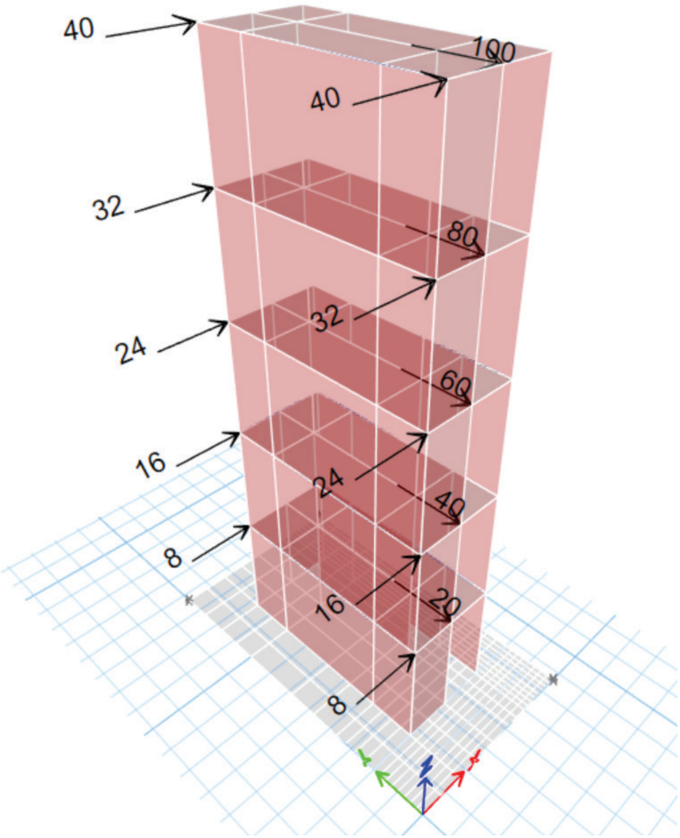


Figure D.4-11 Adjusted overturning demands on the footing

Table D.4-2 Applied loads to structure

	Force X (kips)	Height (ft)	M_y (kip-ft)
Story5	80	60	4800
Story4	64	48	3072
Story3	48	36	1728
Story2	32	24	768
Story1	16	12	192
Sum			**10560**

	Force Y (kips)	Height (ft)	M_x (kip-ft)
Story5	100	60	6000
Story4	80	48	3840
Story3	60	36	2160
Story2	40	24	960
Story1	20	12	240
Sum			**13200**

Maximum Soil Bearing Pressure:

The maximum soil pressure from the computer analysis (Figure D.4-12) assuming soil does not resist tension is 16.8ksf < 18 ksf, footing is **OK** for soil bearing.

Acceptance Ratio = 16.8/18 = **0.933**

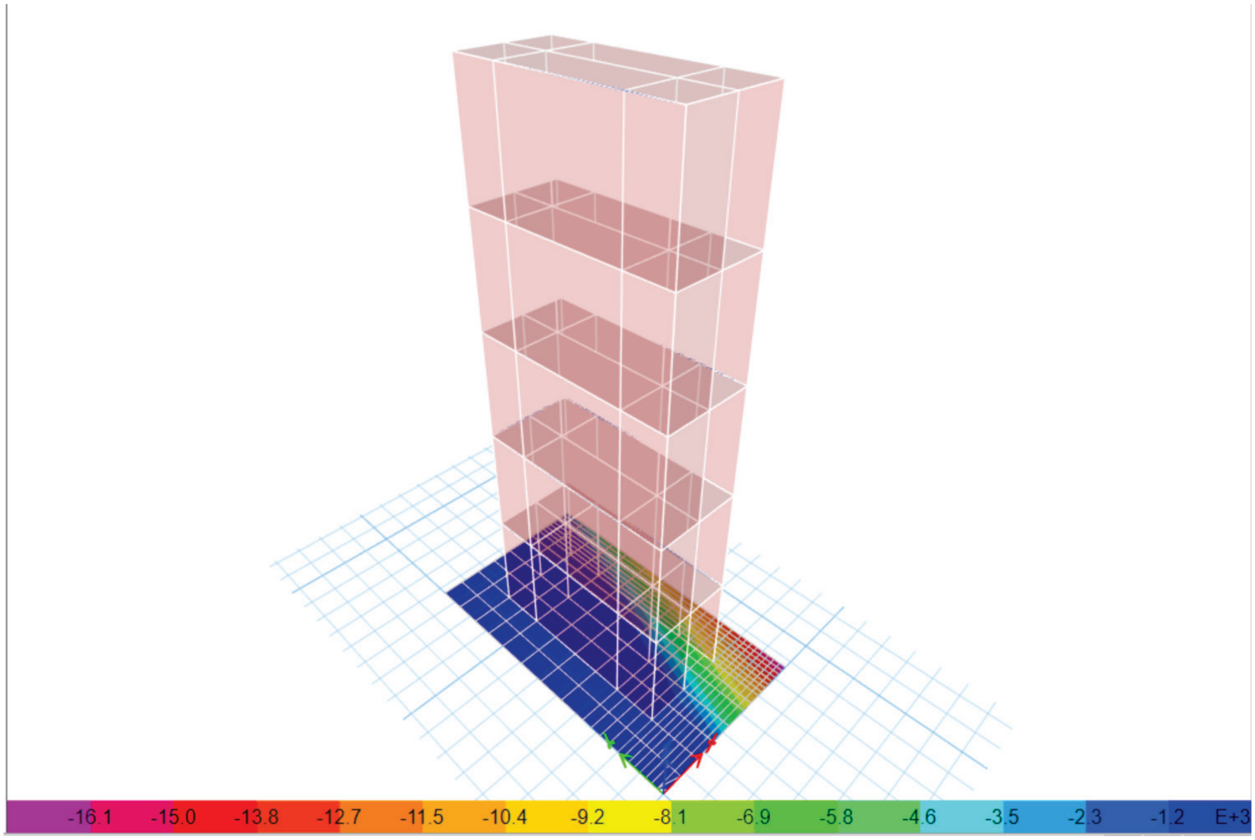


Figure D.4-12 Soil pressure distribution for adjusted loads

Note: For equivalence between the theoretical solution for moment capacity assuming a uniform soil pressure block, and the finite element solution, and adjustment factor is required. When the zero-pressure line of the soil pressure block at the ultimate moment capacity intersects two adjacent edges of the footing, the maximum soil bearing pressure is in the range of 1.5 – 1.69 times the maximum permitted soil bearing pressure for the same applied loads.

D.4.3.2.4 Verify Finite Element results with theoretical ultimate moment capacity:

To verify the finite element results with the theoretical results, the applied load to the foundation should be the moments corresponding to the footing ultimate capacity. The ultimate moment capacity in each direction is determined with appropriate adjustments to account for the center of mass offset of the applied axial load on the footing with footing centroid. The applied overturning moments should satisfy the following equations:

$M_{x,CE} = M_{x,OT} + M_{x\_inherent}$, and

$M_{y,CE} = M_{y,OT} + M_{y\_inherent}$

For an applied moment

$M_y = M_{y,CE} = 10560 - 260 = 10333$ kip-ft

$$L_x := 3\left(X_{c.g.} - \frac{M_y}{P_U}\right) = 10.32$$

$$L_y := 2\left(\frac{P_U}{q_{cDA} \cdot L_x}\right) = 16.959$$

$$M_{xCE} := \frac{1}{2}\, q_{cDA} \cdot L_x \cdot L_y \cdot \left(Y_{c.g.} - \frac{L_y}{3}\right) = 18661.5$$

Moment capacity in orthogonal direction, $M_{x,CE}$

$M_{x,CE}$ = 18661

Therefore $M_{x,OT} = M_{x,CE} - M_{x\_inherent}$

$M_{x,OT}$ = 18661 – (1043.63) = 19,705 kip-ft

Maximum Soil Pressure when applied Moments is at Moment capacity of the footing

Applied overturning moments:

$M_{x,OT}$ = 10560 kip-ft

$M_{y,OT}$ = 19,705 kip-ft

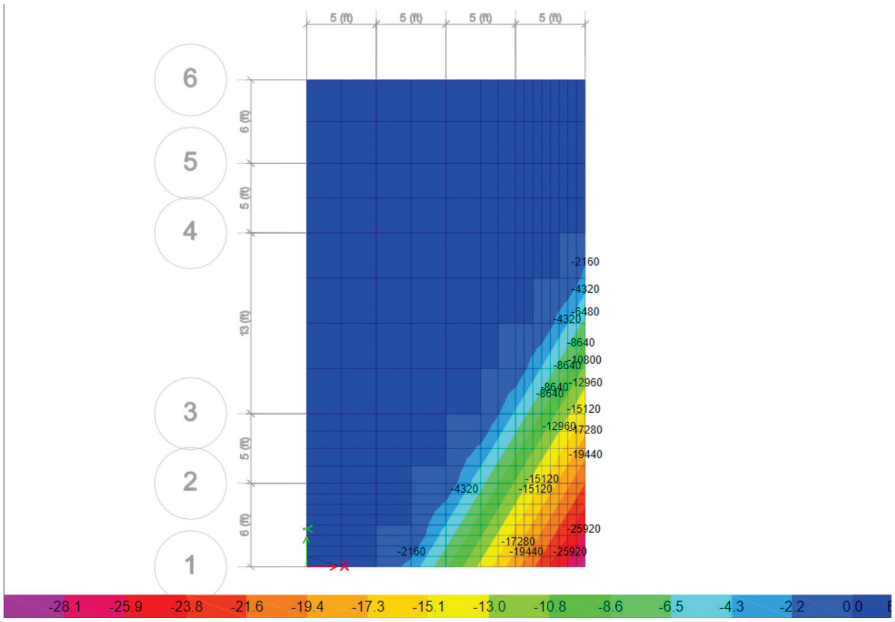
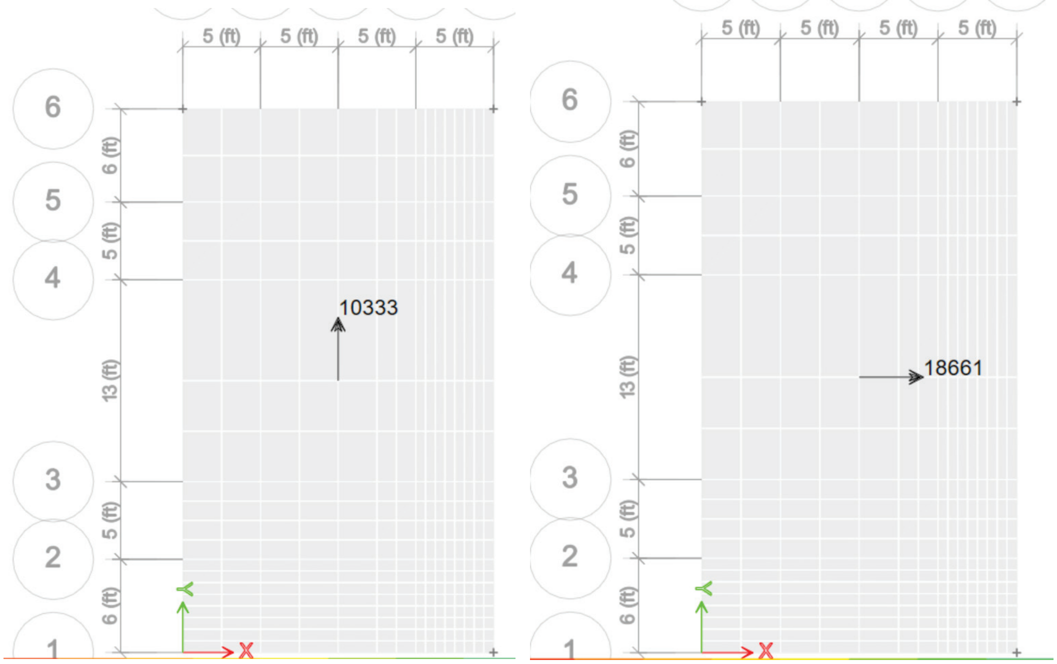


Figure D.4-13 Maximum soil pressure when applied moment is at the moment capacity of the footing

Maximum soil bearing pressure q_{max} = 29.2 ksf.

q_{max}/q_{cDA} = 29.2/18 = 1.62 < 1.69 ok

An additional study was done where the loads were applied directly to the footing, the rigidity of the foundation slab was increased. Loads lat_y and lat_x were applied in the Y- and X- directions as nonlinear static load case. Load case lat_x was applied after load case lat_y was applied which in turn was applied used dead load as the initial conditions.



Load Case: Lat_y

Load Case: Lat_x

Figure D.4-14 Applied moments on the footing

The maximum soil bearing pressure at the corner was 29.655 ksf.

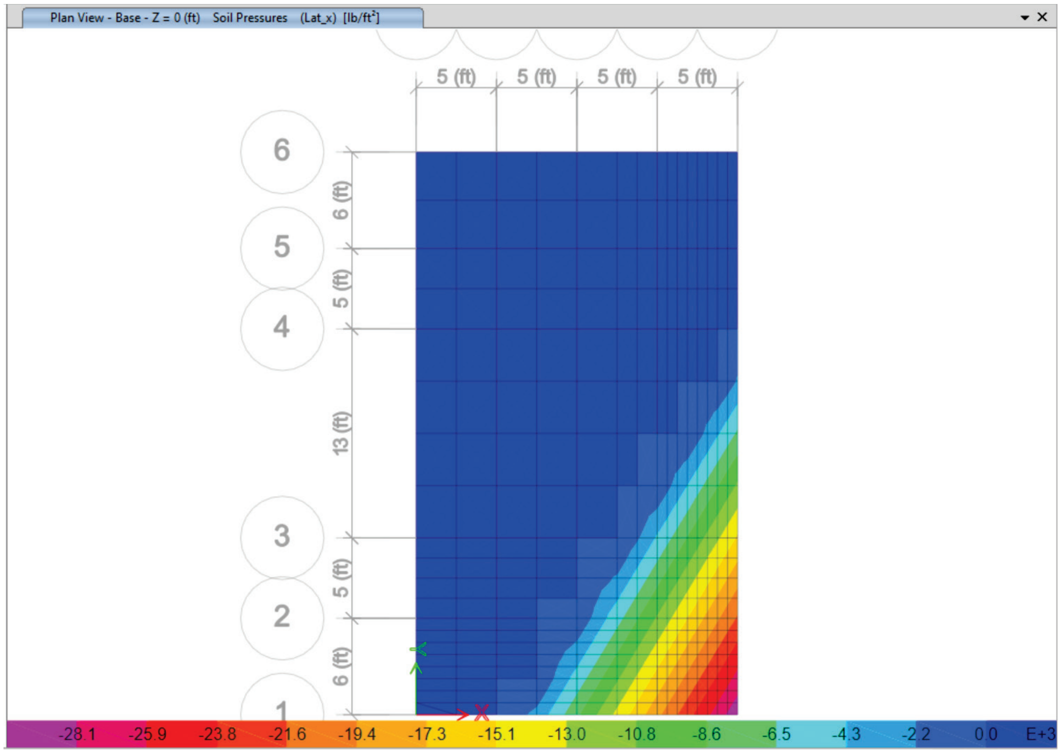


Figure D.4-15 Maximum soil pressure when applied moment is at the moment capacity of the footing for loads applied only on the footing.

Maximum soil bearing pressure q_{max} = 29.655 ksf.

q_{max}/q_{cDA} = 29.655/18 = 1.65 < 1.69 ok

D.4.3.3 SOLUTION USING PROCEDURE 1 OF SECTION 8.4.4.1.2.3

D.4.3.3.1 Soil stiffness calculation, Section 8.4.4.1.2.1 item 3:

Effective width of footing for soil stiffness calculation

The width $B_f{}'$ used in the stiffness calculations is 4 times the footing thickness.

Therefore $B_f{}'$ = 4(3) = 12 feet

$L_f/B_f{}'$ = 35/12 = 2.91 < 3.0

Use spring stiffness from Figure 8-2 of ASCE 41-23

$$K_{z\_sur} = \frac{GB}{(1-\nu)}\left[1.55\left(\frac{L}{B}\right)^{0.75} + 0.8\right] \qquad \text{ASCE/SEI } 41-23\ (\text{Figure } 8-2)$$

Where B = 12 and L = 35. Adjustment for embedment is permitted.

$$K_{z\_sur} = \frac{819 \times 12}{(1-0.25)}\left[1.55\left(\frac{35}{12}\right)^{0.75} + 0.8\right]$$

$$K_{z\_sur} = 55{,}800\ k/ft^3$$

or

$$k_{z\_sur} = 0.077\ k/in^3$$

Adjustment factor for embedment depth

$$\beta_z = \left[1+\frac{1}{21}\frac{D}{B}\left(2+2.6\frac{B}{L}\right)\right]\left[1+0.32\left(\frac{d(B+L)}{BL}\right)^{\left(\frac{2}{3}\right)}\right] \qquad \text{ASCE/SEI 41} - 23\ (\text{Figure } 8-2)$$

$$\beta_z = \left[1+\frac{1}{21}\frac{(4)}{(17.36)}\left(2+2.6\frac{(17.36)}{35}\right)\right]\left[1+0.32\left(\frac{3(17.36+35)}{(17.36)(35)}\right)^{\left(\frac{2}{3}\right)}\right]$$

$$\beta_z = 1.21$$

$$k_z = (1.21)(0.077) = 0.093\ k/in^3$$

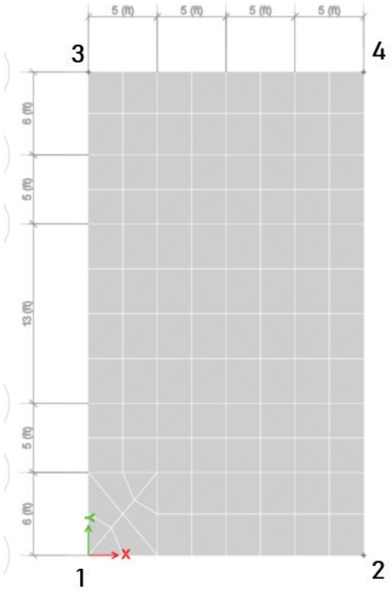
Note, for use in Procedure 1, this stiffness is required to be multiplied by 0.5 for elastic analysis.

Use k = 0.0465 k/in^3

D.4.3.3.2 Foundation Acceptance for soil bearing:

Acceptance is based on the footing rotation being less than 0.75 times the rotation values given in Table 8-8.

Footing deflections and soil pressure distribution under the footing are shown in Figure D.4-16 and Figure D.4-17 respectively.



Point	Defection (in)
1	0.31
2	-4.6
3	4.16
4	-1.18

Figure D.4-16 Deflections at the four corners of the footing

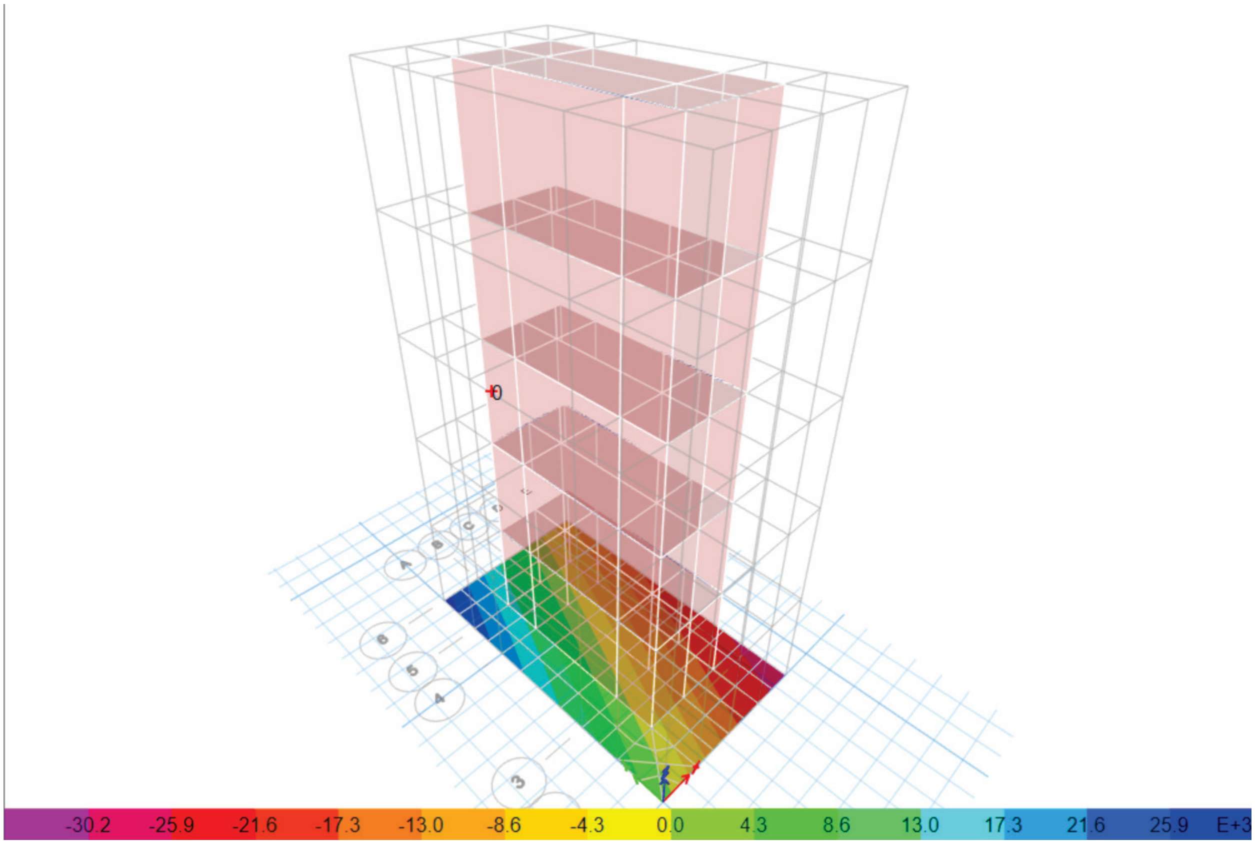
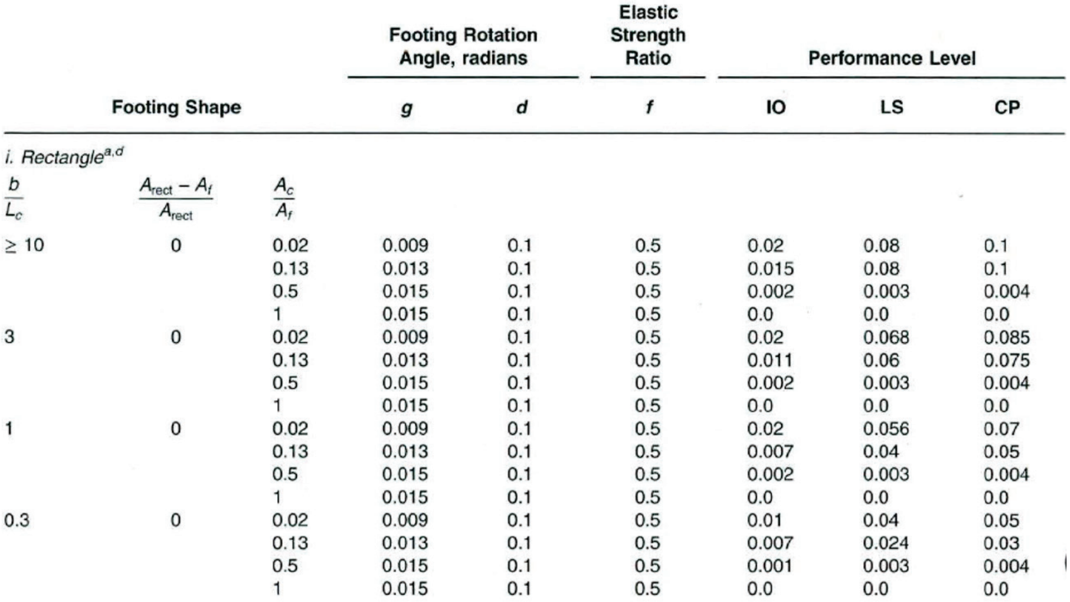


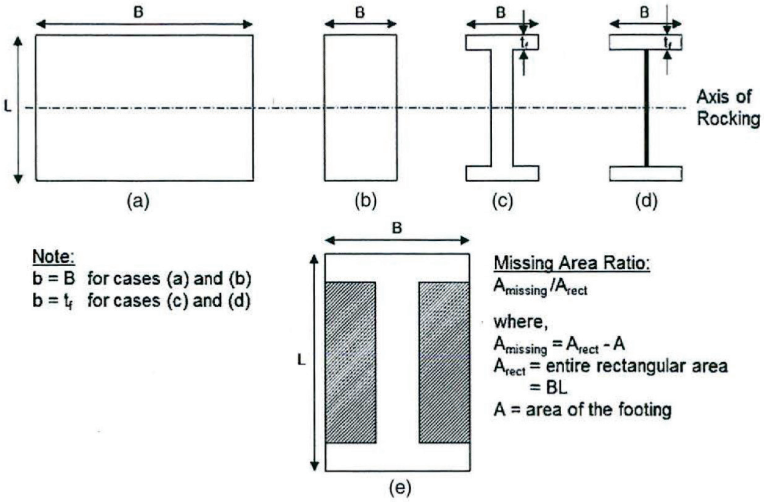
Figure D4-17 Soil pressure distribution under the footing.

q_{max} = 30.7 ksf, q_{min} = 27.9 ksf. Note: loads applied are unreduced pseudo seismic loads

Determination of allowable rotation (Procedure 1)



Footing Shape			Footing Rotation Angle, radians		Elastic Strength Ratio	Performance Level		
			g	*d*	*f*	IO	LS	CP
i. Rectangle[a,d]								
$\frac{b}{L_c}$	$\frac{A_{rect} - A_f}{A_{rect}}$	$\frac{A_c}{A_f}$						
≥ 10	0	0.02	0.009	0.1	0.5	0.02	0.08	0.1
		0.13	0.013	0.1	0.5	0.015	0.08	0.1
		0.5	0.015	0.1	0.5	0.002	0.003	0.004
		1	0.015	0.1	0.5	0.0	0.0	0.0
3	0	0.02	0.009	0.1	0.5	0.02	0.068	0.085
		0.13	0.013	0.1	0.5	0.011	0.06	0.075
		0.5	0.015	0.1	0.5	0.002	0.003	0.004
		1	0.015	0.1	0.5	0.0	0.0	0.0
1	0	0.02	0.009	0.1	0.5	0.02	0.056	0.07
		0.13	0.013	0.1	0.5	0.007	0.04	0.05
		0.5	0.015	0.1	0.5	0.002	0.003	0.004
		1	0.015	0.1	0.5	0.0	0.0	0.0
0.3	0	0.02	0.009	0.1	0.5	0.01	0.04	0.05
		0.13	0.013	0.1	0.5	0.007	0.024	0.03
		0.5	0.015	0.1	0.5	0.001	0.003	0.004
		1	0.015	0.1	0.5	0.0	0.0	0.0



Note:
b = B for cases (a) and (b)
b = t_f for cases (c) and (d)

Missing Area Ratio:
$A_{missing}/A_{rect}$

where,
$A_{missing} = A_{rect} - A$
A_{rect} = entire rectangular area
= BL
A = area of the footing

b = *B*$_f$ = 20 ft

$$L_c = \frac{P_U}{B_f q_{cDA}} = \frac{1575.2}{20 * 18} = 4.37\ ft$$

$$A_c = \frac{P_U}{q_{cDA}} = 87.5;\ ft^2$$

$$A_{rect} = A_f = 700;\ ft^2$$

$$A_{miss} = \frac{A_{rect} - A_f}{A_{rect}} = 0;\ ft^2$$

$$b_{ratio} = \frac{b}{L_c} = \frac{20}{4.37} = 4.57;$$

$$A_{c\_ratio} = \frac{A_c}{A_f} = \frac{87.5}{700} = 0.125;$$

Interpolating the θ_{CP} from Table 8-8

$$\theta_{CP} = 0.085 + (0.1 - 0.085)\frac{(b_{ratio} - 3.0)}{(10.0 - 3.0)} = 0.088;$$

$$\theta_{CP} = 0.088$$

Permitted rotation for elastic analysis using Procedure 1

$$\theta_{CP} = (0.75)0.088 = 0.066$$

Footing rotation demands are calculated based on the deflections at the four corners of the footing and the distance between them, and are given in Tables D.4-3 and D.4-4 below:

Table D.4-3 Distance between the corner points of the footing

Point	X coord (ft)	Y coord (ft)	Distance from Point (ft)			
			1	2	3	4
1	0	0	-	20	35	40.3
2	20	0		-	40.3	35
3	0	35			-	20
4	20	35				-

Table D.4-4 Footing rotation demands compared with allowable

Point	Defection (in)	Rotation from Point (radians)				Maximum Rotation	Allowable Rotation	Acceptance Ratio
		1	2	3	4			
1	0.31	-	0.020	0.009	0.008	0.020	0.066	0.31
2	-4.6		-	0.018	0.021	0.021	0.066	0.32
3	4.16			-	0.022	0.022	0.066	0.34
4	-1.18				-	-	-	-

Using this procedure, the maximum acceptance ratio, AR = 0.34.

Footing is acceptable for soil bearing using Procedure 1, Max. AR = 0.34.

D.4.3 Summary

A comparison of AR for soil bearing for the case study example between the various methods is shown below:

	Isolated Footing	**Procedure 1**	**Procedure 2**
Acceptance Ratio	Eq. (8-20): 0.82	0.34	0.93
	Eq. (8-21): 0.75		

Note: Procedure 1 is dependent on the stiffness used, so this result will change depending on the soil stiffness values used.