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**Morning Questions**

**p. 23, Question 260:**

Change the sixth line of Design Data to read:

Floor live load                      50 lb/ft<sup>2</sup> (non-reducible)

**Solution to Building Problem 260**

**pp. 64–65:** Section (a) should read as follows:

**(a) Tributary width**

Since no member sizes are provided, use span lengths in lieu of clear spans.

$$\text{Trib. width} = \frac{25}{2} \times 2 = 25 \text{ ft}$$

$$\begin{aligned} \text{Roof} \quad w &= 1.2(120 \times 25) + 1.6(0) + 0.5(40 \times 25) && \text{Eq. 16-2} \\ &= 3,600 + 500 \\ &= 4,100 \text{ lb/ft} \end{aligned}$$

$$\begin{aligned} \text{Floor} \quad w &= 1.2(130 \times 25) + 1.6(50 \times 25) + 0.5(0) \\ &= 3,900 + 2,000 \\ &= 5,900 \text{ lb/ft} \end{aligned}$$

$$\begin{aligned} \text{Roof} \quad w &= 1.2(120 \times 25) + 1.6(40 \times 25) + 0 && \text{Eq. 16-3} \\ &= 3,600 + 1,600 \\ &= 5,200 \text{ lb/ft} \end{aligned}$$

$$\begin{aligned} \text{Floor} \quad w &= 1.2(130 \times 25) + 1.6(0) + 0.5(50 \times 25) \\ &= 3,900 + 625 \\ &= 4,525 \text{ lb/ft} \end{aligned}$$

$$\begin{aligned} \text{Roof} \quad w &= 1.2(120 \times 25) + 1.6(0) + 0.5(40 \times 25) && \text{Eq. 16-4} \\ &= 3,600 + 500 \\ &= 4,100 \text{ lb/ft} \end{aligned}$$

$$\begin{aligned} \text{Floor} \quad w &= 1.2(130 \times 25) + 1.6(0) + 0.5(50 \times 25) \\ &= 3,900 + 625 \\ &= 4,525 \text{ lb/ft} \end{aligned}$$

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**Solution to Building Problem 260 (continued)**

<b>Maximum loading:</b>	Roof	5,200 lb/ft	Eq. 16-3
	Floor	5,900 lb/ft	Eq. 16-2

**Maximum positive moments (ACI coefficients – ACI 8.3.3)**

Roof  $M_R = 5.200 \times 25^2 / 11$  (Discontinuous end unrestrained)  
 $= 295$  ft-kips

*or*

$M_R = 5.200 \times 25^2 / 14$  (Discontinuous end integral  
 $= 232$  ft-kips with support)

Floor  $M_F = 5.900 \times 25^2 / 11 = 335$  ft-kips

*or*

$M_F = 5.900 \times 25^2 / 14 = 263$  ft-kips

**(b) Shear and overturning moment at base of wall**

**Case 1: Eq. 16-3  $1.2 D + 1.6 L_r + 0.8 W + f_1 L$  ( $f_1 = 0.5$ )**

Shear  $V = 0.80 (12) + 0.80 (36) = 9.6 + 28.8 = 38.4$  kips

Vertical load

$$DL = 1.2[18.8 + 20.3 + \frac{8}{12} (150)(8)(24)]$$

$$= 22.6 + 24.4 + 23.0$$

$$= 70 \text{ kips}$$

$$L_R = 1.6(6.3)$$

$$= 10.1 \text{ kips}$$

$$L = (0.5)(7.8)$$

$$= 3.9 \text{ kips}$$

$$\text{VERT} = 84.0 \text{ kips}$$

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**Solution to Building Problem 260 (continued)**

OT Moment at left edge

$$M_{\text{WIND}} = (9.6 \times 24) + (28.8 \times 12) = 576 \text{ ft-kips} \quad \text{counterclockwise}$$

$$M_{\text{RES DL}} = 0.50(22.6 + 24.4) + (23 \times 4) = 115.5 \text{ ft-kips} \quad \text{clockwise}$$

$$M_{\text{OT}} = M_{\text{WIND}} - M_{\text{RES}} = 460.5 \text{ ft-kips}$$

$$\text{Eq. 16.3: } V = 38.4 \text{ kips}$$

$$P = 84.0 \text{ kips}$$

$$M_{\text{OT}} = 460.5 \text{ ft-kips}$$

**Case 2: Eq. 16.4    1.2 D + 1.6 W + f<sub>1</sub>L + 0.5 L<sub>R</sub>**

$$\text{Shear } V = 1.6(12) + 1.6(36) = 19.2 + 57.6 = 76.8 \text{ kips}$$

Vertical load

$$\text{DL} = 1.2[18.8 + 20.3 + \frac{8}{12}(150)(8)(24)]$$

$$= 22.6 + 24.4 + 23.0$$

$$= 70 \text{ kips}$$

$$L_{\text{R}} = 0.5(6.3)$$

$$= 3.15 \text{ kips}$$

$$L = 0.5(7.8)$$

$$= 3.9 \text{ kips}$$

$$\text{VERT} = 77.05 \text{ kips}$$

OT Moment at left edge

$$M_{\text{WIND}} = (19.2 \times 24) + (57.6 \times 12) = 1,152 \text{ ft-kips} \quad \text{counterclockwise}$$

$$M_{\text{RES DL}} = 0.50(22.6 + 24.4) + (23 \times 4) = 115.5 \text{ ft-kips} \quad \text{clockwise}$$

$$M_{\text{OT}} = M_{\text{WIND}} - M_{\text{RES}} = 1,036.5 \text{ ft-kips}$$

$$\text{Eq. 16.4: } V = 76.8 \text{ kips}$$

$$P = 77.1 \text{ kips}$$

$$M_{\text{OT}} = 1,036.5 \text{ ft-kips}$$

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**Solution to Building Problem 260 (continued)**

**Case 3: Eq. 16.6    0.9 D + 1.6 W**

Shear:    Same as Case 2:  $V = 76.8$  kips

Vertical load

$$\begin{aligned} DL &= 0.9[18.8 + 20.3 + \frac{8}{12}(150)(8)(24)] \\ &= 16.92 + 18.27 + 17.28 \\ &= 52.5 \text{ kips} \end{aligned}$$

OT Moment

$$\begin{aligned} M_{\text{WIND}} &= 1,152 \text{ ft-kips} \quad \text{counterclockwise (same as Case 2)} \\ M_{\text{RES}} &= 0.50(16.92 + 18.27) + (17.28 \times 4) = 86.7 \text{ ft-kips} \quad \text{clockwise} \\ M_{\text{OT}} &= M_{\text{WIND}} - M_{\text{RES}} = 1,065 \text{ ft-kips} \end{aligned}$$

$$\begin{aligned} \text{Eq. 16.6: } V &= 76.8 \text{ kips} \\ P &= 52.5 \text{ kips} \\ M_{\text{OT}} &= 1,065 \text{ ft-kips} \end{aligned}$$

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**Solution to Building Problem 260 (continued)**

**(c) Check required wall reinforcement**

$$V_{\mu} = 80.0 \text{ kips}$$

$$M_{\mu} = 800 \text{ ft-kips}$$

Moment:  $b = 8 \text{ in.}$        $d \sim 96 - 4 = 92 \text{ in.}$

$$k_{\mu} \sim \frac{800 \times 12}{0.90 \times 2.5 \times 8 \times 92^2} = 0.063 \Rightarrow \delta \sim 0.066$$

$$A_s = 0.066 \times \frac{2.5}{60} \times 8 \times 92 = 2.02 \text{ in}^2$$

$$\sim (2) \#9 \text{ bars}$$

Shear:  $\phi V_c = 2 \times 0.75 \sqrt{2,500} \times 8 \times 92 / 1,000 = 55.2 \text{ kips}$

Assume #4 bars

$$S \sim \frac{0.75 \times 0.20 \times 60 \times 92}{80 - 55.2} = 33.4 \text{ in.} \quad \text{min. horizontal steel controls}$$

Check minimum reinforcement:

$$A_H = 0.0020 \times 8^{17.6} \times 12 = 0.19 \text{ in}^2 \quad (\text{ACI 14.3.3}) \quad \#4 @ 12 \text{ in. o.c.}$$

$$A_V = 0.0012 \times 8 \times 12 = 0.12 \text{ in}^2 \quad (\text{ACI 14.3.2}) \quad \#4 @ 18 \text{ in. o.c. (max)}$$

Connection to foundation (shear friction) (ACI 11.7.4)

$$V_N = A_{vf} f_y \mu \quad \mu = 0.60 \lambda, \lambda = 1.0 \quad (\text{Assume concrete surface not intentionally roughened})$$

$$80 = A_{vf} \times 60 \times 1.0 \quad A_{vf} = 2.22 \text{ in}^2 \quad 7 \#4 \text{ (min)}$$

Development length for #9 bars (ACI Chapter 12)

Tension:

$$l_d = \left[ \frac{3}{40} \times \frac{f_y}{\sqrt{f'_c}} \times \frac{\psi_t \psi_e \psi_s \lambda}{C_b + k_{tr}} \right] d_b \quad \begin{array}{l} d_b = 1.128 \text{ in.} \\ f_y = 60 \\ \sqrt{f'_c} = 50 \end{array} \quad (\text{ACI 12.-1})$$

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**Solution to Building Problem 260 (continued)**

ACI 12.2.4       $\psi_t = \psi_e = \psi_s = 1.0$

(a),(b),(c)  
ACI 12.2.4 (d)       $\lambda = 1.0$

$K_{tr} = 0$

$C_b = 2.0$

$\frac{C_b + K_{tr}}{d_b} = \frac{1.5 + 0}{1.128} = 1.33 < 2.5$

OK

$l_d = \left( \frac{3}{40} \frac{60,000}{50} \frac{1.0 \times 1.0 \times 1.0 \times 1.0}{1.33} \right) 1.128$   
 $= 76.33 \text{ in.} \quad 6 \text{ ft} - 5 \text{ in. for \#9 bars}$

Compression (ACI 12.3.2)

$l_{dc} = 0.02 \frac{60,000}{\sqrt{2,500}} 1.128 = 27 \text{ in.}$

$l_{dc} = 0.0003 \times 60,000 \times 1.128 = 20.25 \text{ in.}$

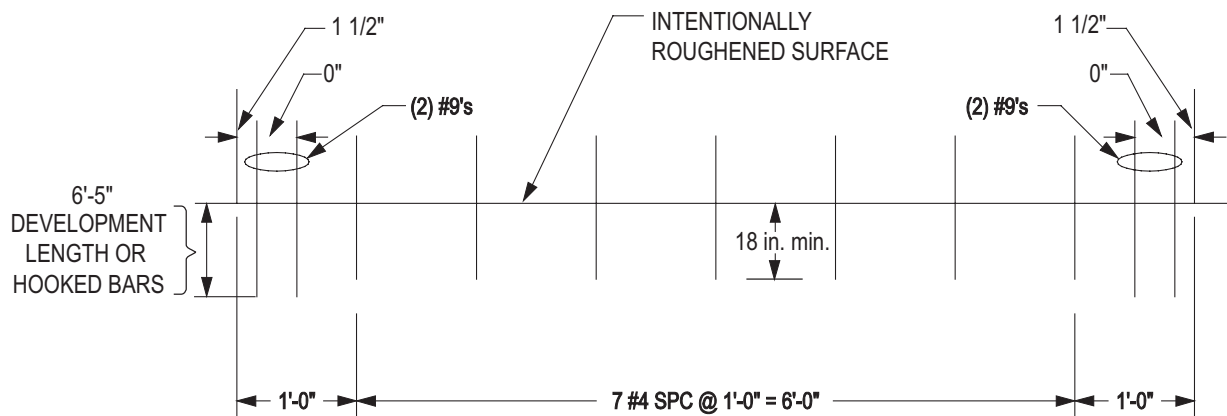
Note under Eq. 12-2

For #4 bars:

$\frac{C_b + K_{tr}}{d_b} = \frac{4}{0.5} = 8$

use 2.5

$l_d = \left( \frac{3}{40} \right) \left( \frac{60,000}{50} \right) \left( \frac{1.0}{2.5} \right) (0.5)$   
 $= 18 \text{ in.}$



NOTE; IF SURFACE IS NOT INTENTIONALLY ROUGHENED,  
(11) #4 DOWELS ARE REQUIRED TO TRANSFER SHEAR

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## **Building Problem 450**

### **SITUATION:**

A one-story maintenance facility is to be constructed consisting of a composite concrete roof deck supported on structural steel beams and columns. Lateral loads are resisted by ordinary steel concentrically braced frames, as shown in **Figure 450A**. Wind loads may be neglected.

### Design Codes:

IBC: *International Building Code*, 2006 edition (without supplements).

AISC: *Steel Construction Manual*, 13th edition.

### Design Data:

Strength of concrete, $f'_c$	4,000 psi
Weight of concrete	150 pcf
Reinforcing steel	ASTM 615, Grade 60
Steel	
Wide flange shapes	A992
Steel plate	ASTM A36
Weld electrode	E70XX
Occupancy Category II	
Site Class D	
$S_s = 1.0$	
$S_1 = 0.2$	
Assume $\rho = 1.0$	
Roof slab DL (includes steel weight)	50 psf
Roof LL (snow)	30 psf
Rooftop equipment	100 psf
All beam-column connections are pinned.	
Allowable soil bearing pressure (D + L)	6,000 psf    1/3 increase for seismic

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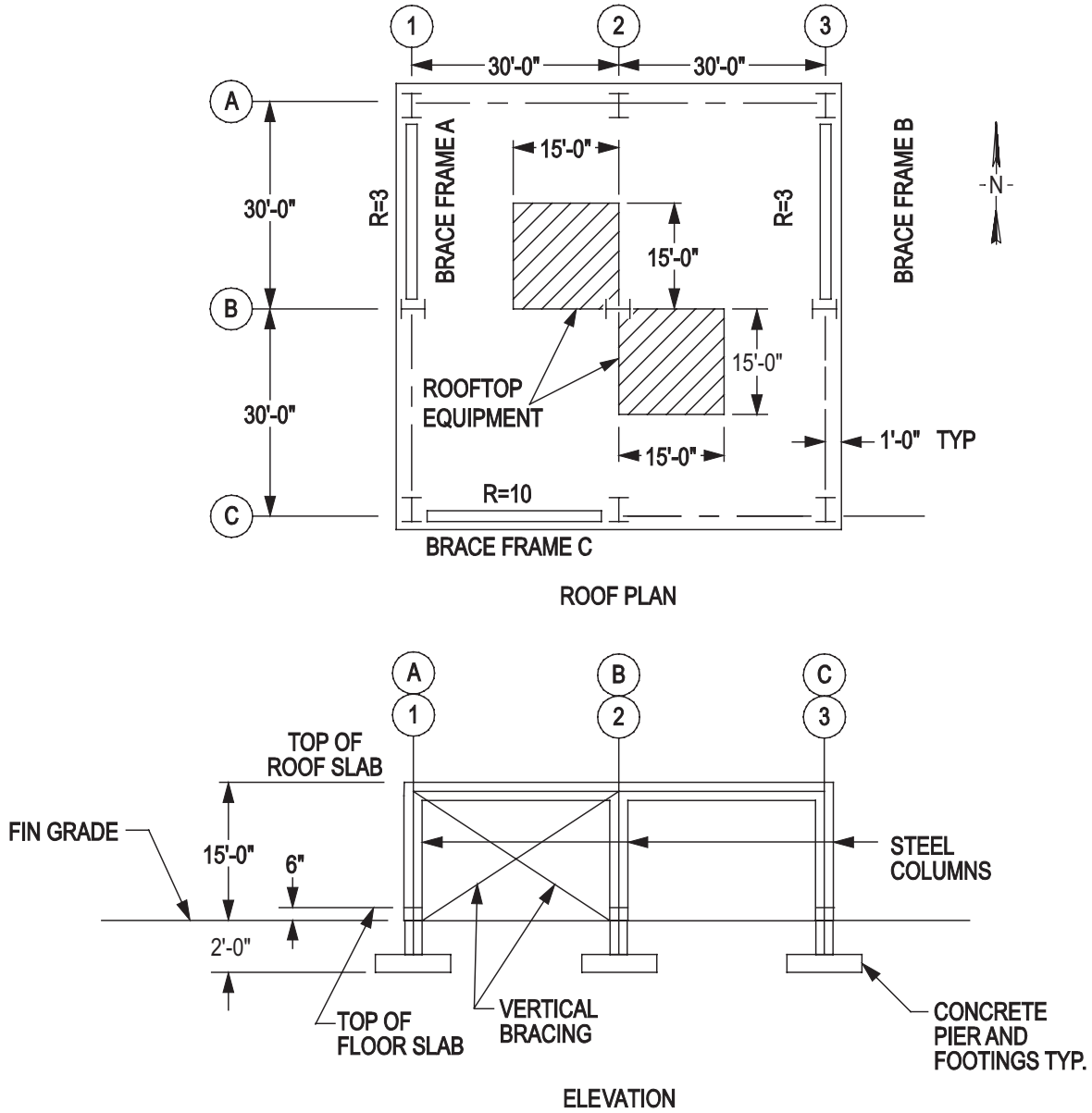
**Building Problem 450 (Continued)**

**REQUIREMENTS:**

- (a) Determine the seismic base shear on the building in both the N-S and E-W directions.
- (b) Determine the distribution of seismic shear to the three braced frames when the earthquake is in the E-W direction.
- (c) Determine the column design load for Column C-2 and a square HSS column size.
- (d) Design a concrete pier and footing for Column C-2. Draw a legible sketch showing all steel reinforcing required. Neglect soil over the footing.

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**Building Problem 450 (Continued)**



**FIGURE 450**

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## Building Solution 450

**(a) Seismic base shear per IBC 2006**

$$F_a = 1.1 \quad \text{IBC Table 1613.5.3(1) with Site Class D and } S_s = 1.0$$

$$\begin{aligned} S_{MS} &= F_a S_s && \text{Eq. 16-37} \\ &= 1.1 \times 1.0 = 1.1 \end{aligned}$$

$$F_V = 2.0 \quad \text{IBC Table 1613.5.3(2) with Site Class D and } S_1 = 0.2$$

$$\begin{aligned} S_{M1} &= F_V S_1 && \text{Eq. 16-38} \\ &= 2.0 \times 0.2 = 0.4 \end{aligned}$$

$$\begin{aligned} S_{DS} &= 2/3 S_{MS} && \text{Eq. 16-39} \\ &= 2/3(1.1) = 0.73 \end{aligned}$$

$\therefore$  Seismic Design Category = D Table 1613.5.6(1)

$$\begin{aligned} S_{D1} &= 2/3 S_{M1} && \text{Eq. 16-40} \\ &= 2/3(0.4) = 0.27 \end{aligned}$$

$\therefore$  Seismic Design Category = D Table 1613.5.6(2)

Note: Per 1616.3 The most severe seismic design category from  $S_{DS}$  and  $S_{D1}$  must be used. In this case, both tables indicate seismic design category D.

### Ordinary Concentrically Braced Steel Frames ( $R = 3.25$ )

Seismic Design Category D

$$V = C_s W \quad \text{ASCE 7-05, Eq. 12.8-1}$$

$$C_s = \frac{S_{DS}}{R/I_E} \quad I_E = 1.0 \quad \text{ASCE 7-05, Eq. 12.8-2}$$

$$C_s = \frac{0.73}{3.25} = 0.225$$

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**Building Solution 450 (Continued)**

$$C_{S \max} = \frac{S_{D1}}{(R/I_E)T} \geq 0.01 \quad \text{ASCE 7-05, Eq. 12.8-3}$$

$$= \frac{0.27}{(3.25)T} \quad T_a = C_T h_N^{3/4} = (0.02)(14.5)^{3/4}$$

$$T_a = 0.02(7.43) = 0.149 \quad S_{D1} = 0.27$$

$$C_u = 1.4 + \left( \frac{0.3 - 0.27}{0.3 - 0.2} \right) (1.5 - 1.4) \quad \text{ASCE 7-05, Table 12.8-1}$$
$$= 1.4 + 0.03 = 1.43$$

$$T_{\max} = C_u T_a = 1.43(0.149) = 0.213$$

$$T = T_a = 0.149$$

$$C_{S \max} = \frac{0.27}{(3.25)(0.149)} = 0.558$$

$$C_{S \min} = 0.01 \quad \text{ASCE 7-05, Eq. 12.8-5}$$

$$C_S = 0.225 \quad C_{S \min} < C_S < C_{S \max}$$

$$\therefore \text{ use } C_S = 0.225$$

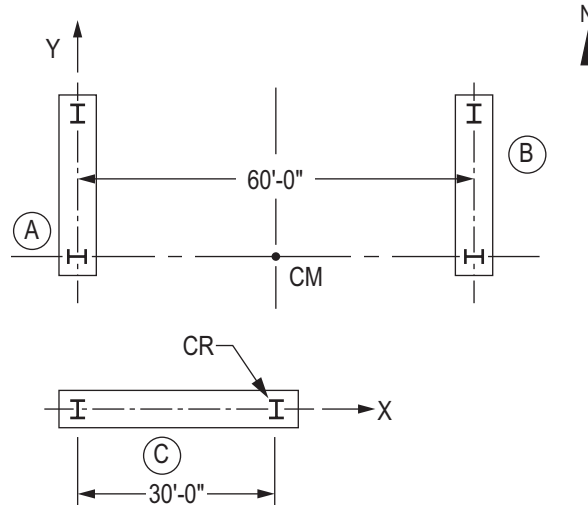
**Building Seismic Weights**

Roof Weight:

Roof Slab	62 ft × 62 ft × 0.05 ksf	=	192.2 kips
Equipment	2 × 15 ft × 15 ft × 0.10	=	<u>45.0 kips</u>
	W	=	237.2 kips

$$\text{Base Shear} = V = C_S W = 0.225 \times 237.2 = 53.4 \text{ kips}$$

**Building Solution 450 (Continued)**



**(b) Seismic shear in the three walls, earthquake in the E-W direction**

When the seismic load acts in E-W direction, Braced Frame C will carry the entire story shear. The next figure shows the position of the center of gravity in relation and the x- and y-axes.

By observation, the position of the center of rigidity in the E-W direction is defined by the following coordinates:

- $x_{CR}$  = distance of the center of rigidity from the y-axis = 30 ft
- $y_{CR}$  = distance of the center of rigidity from the x-axis = 0 ft
- $x_{GR}$  = distance of the center of mass from the y-axis = 30 ft
- $y_{GR}$  = distance of the center of mass from the x-axis = 30 ft

$V = 53.4$  kips

$T = \text{actual torsion} + \text{accidental torsion}$   
 $= 53.4 \text{ kips} (30 \text{ ft}) + 53.4 \text{ kips} (0.05 \times 62)$   
 $= (1,602 + 165.54) \text{ ft-kips}$   
 $= 1,767.54 \text{ ft-kips}$

OK

Shear in Braced Frames A and B due to torsion =  $\frac{1,767.54 \text{ ft-kips}}{60 \text{ ft}} = 29.5$  kips

Shear in Braced Frame C = 53.4 kips

## **Building Solution 450 (Continued)**

**(c) Seismic shear**

For seismic shear = 53.4 kips

$$\text{Axial load from seismic} = \frac{53.4 \text{ kips} \times 14.5 \text{ ft}}{30 \text{ ft}} = 26 \text{ kips} \pm$$

$$\text{DL} = 30 \text{ ft} \times 15 \text{ ft} \times 0.05 \text{ ksf} = 22.5 \text{ kips}$$

$$\text{LL} = 30 \text{ ft} \times 15 \text{ ft} \times 0.03 \text{ ksf} = 13.5 \text{ kips}$$

Equipment load:

OK

$$15 \text{ ft} \times 15 \text{ ft} \times \frac{7.5}{30} \times \frac{22.5}{30} \times 0.01 = 0.42 \text{ kips}$$

$$1.2 \text{ D} + 1.0 \text{ E} + \text{L} = 1.2 (22.5) + 26 + 13.5 + 0.43 = 66.93 \text{ kips}$$

$$0.9 \text{ D} + 1.0 \text{ E} = 0.9 (22.5) + 26 = 46.25 \text{ kips}$$

HSS 4 × 4 × 3/8

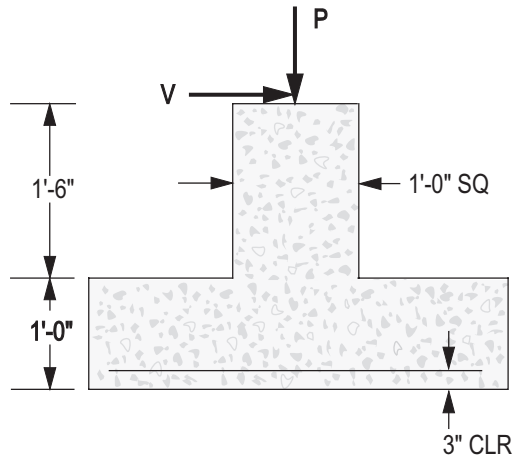
AISC Table 4-4

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**Building Solution 450 (Continued)**

(d) Design a concrete pier and footing for Column C-2

$P_D = 22.5 \text{ kips}$        $V = 53.4/2 = 26.7 \text{ kips}$   
 $P_L = 14 \text{ kips}$   
 $P_E = 26 \text{ kips}$



**Size area of footing**

Assume 5'-0" × 5'-0" × 1'-0" footing

$$\begin{aligned}
 P_r &= (22.5 + 14 + 26) + (1.0^2 \times 1.5)(0.150) + (5.0^2 \times 1.0 \times 0.150) \\
 P_r &= 62.5 + 0.225 + 3.75 \\
 &= 66.475 \text{ kips} = R
 \end{aligned}$$

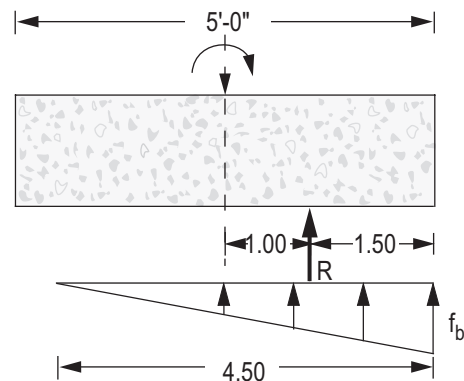
$$M = Vh = 26.7 \times 2.5 = 66.75 \text{ ft-kips}$$

$$e = \frac{M}{P} = \frac{66.75 \text{ ft-kips}}{66.475 \text{ kips}} = 1.00 \text{ ft}$$

$$f_b = \frac{2R}{(4)(4.5)} = 7.38 \text{ ksf}$$

$$1.33 f_{b \text{ all}} = 7.980 \text{ ksf}$$

OK



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**Building Solution 450 (Continued)**

**Design Footing**

**1. Check beam shear:**

$$d = 12 - 3 - 0.75/2 = 8.625 \text{ in.}$$

$$1.2D + 1.0 E + 1.0 L$$

$$P_u = 1.2(25.975) + 1.0(14) + 1.0 (26) = 71.17 \text{ kips}$$

$$M_u = 1.0 (26.7)(2.5) = 66.75 \text{ ft-kips}$$

$$e_u = \frac{M_u}{P_u} = 0.938 \text{ ft} \quad (1.56)$$

$$W_u = \frac{2(71.17)}{4 \times 3(1.56)} = 7.60 \text{ ksf}$$

$$V_u = 5.52(1.28) + (7.60 - 5.52)(1.28/2) = 8.4 \text{ kips/ft or 42 kips}$$

$$V_n = \phi V_c = \phi 2 \times b \times d \times \sqrt{f'_c}$$

$$= 0.75 \times 2 \times 60 \times 8.625 \times \sqrt{4,000}$$

$$= 49.1 \text{ kips} > 42 \text{ kips}$$

OK

**2. Check punching shear:**

$$b_o = (8.625 + 12)(4) = 82.5 \text{ in.}$$

$$V_n = \phi V_c = 0.754 b_o d \sqrt{f'_c}$$

$$= 0.75 (4)(82.5)(8.625)(4,000)^{1/2} = 135 \text{ kips} > 71.2 \text{ kips}$$

OK

**3. Design footing reinforcement:**

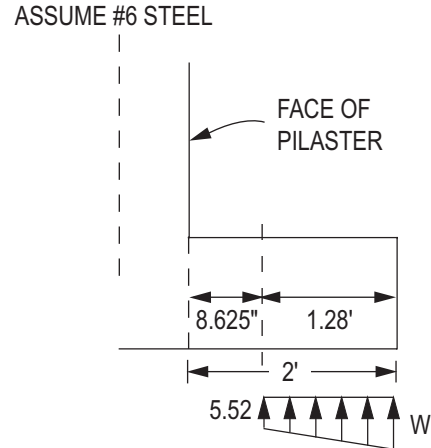
$$M_u = 5.52 \frac{(1.28)^2}{2} + (7.60 - 5.52) \frac{(1.28)}{2} \left(\frac{2}{3}\right) (1.28) = 5.7 \text{ ft-kips/ft}$$

$$\rho_{\min} = 0.0033 \quad \Delta_S \min = 0.0033 \times 60 \times 8.625 = 1.71 \text{ in}^2$$

$$\text{Use \# 6 @ 12" o.c.} \quad 5 \times 0.44 \text{ in}^2 = 2.21 \text{ in}^2 \quad (5 \#6)$$

$$R_u = \frac{M_U (12,000)}{\phi b d^2} = \frac{5.7 \times 12,000}{0.90 \times 8.625 \times 60^2} = 2.44 \quad \rho_{\min} \text{ governs}$$

Use #6 @ 12" o.c. each way



## Building Solution 450 (Continued)

### 4. Design pier – moment:

$$M_u = 26.7 \times 1.5 = 40.05 \text{ ft-kips}$$

$$P_u = 71.17 \text{ kips} \quad \text{Assume 4 vert \#6}$$

$$\phi P_n = 0.80 \phi \left[ 0.85 f'_c (A_g - A_{st}) + f_y A_{st} \right] \quad \phi = 0.65$$
$$= 306 \text{ kips} > 71.17 \text{ kips} \quad \text{OK}$$

$$R_u = \frac{40.05 \times 12,000}{0.90 \times 12 \times 10^2} = 445$$

$$\rho = 0.008$$

$$A_s = \rho b d = 0.96$$

4 #6 OK

### 5. Design pier – shear:

$$V_n = \phi (V_c + V_s)$$

$$V_u = 26.7 \text{ kips} \quad V_u / \phi = 35.6$$

$$V_c = 2\sqrt{f'_c} b d = 2(4,000)^{1/2} (12)(10) = 15.2 \text{ kips}$$

$$V_{s \text{ req}} = \frac{V_u}{\phi} - V_c = 20.4$$

$$V_s = \frac{A_v f_y d}{s}$$

$$s = \frac{(0.4)(60)(10)}{20.4} = 11.7 \text{ in.}$$

Use #4 hoops

$$A_v = 2 \times 0.2 = 0.4 \text{ in}^2$$

Use 8" spacing max

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**Building Solution 450 (Continued)**

