

ERRATA for
PE Structural Engineering Practice Exam
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Revisions are shown in red.

Vertical Forces Depth—Buildings, Scenario 1, Question 3, p. 51

Assumptions:

Live load reduction

Roof and floor systems are 24 in. deep

Basement system is 12 in. deep

Vertical Forces Depth—Buildings, Scenario 2, Question 5, p. 67

Assumptions:

Use an angle between the axis of the glue-laminated beam and the tension rod of 30°.

Tension in the rod is from dead load.

Steel rod, collar, and nut are adequate.

Vertical Forces Depth—Buildings, Scenario 2, Question 10, p. 71

Select the five structural roof framing members that must be checked for structural adequacy for the added load shown in the figure.

Vertical Forces Depth—Bridges, Scenario 1, Question 2, p. 77

$F_y = 50$ ksi

Vertical Forces Depth—Bridges, Scenario 1, Question 3, p. 78

$F_{yrb} = F_{yrt} = 60$ ksi for reinforcement

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Vertical Forces Depth—Bridges, Scenario 1, Question 4, p. 79

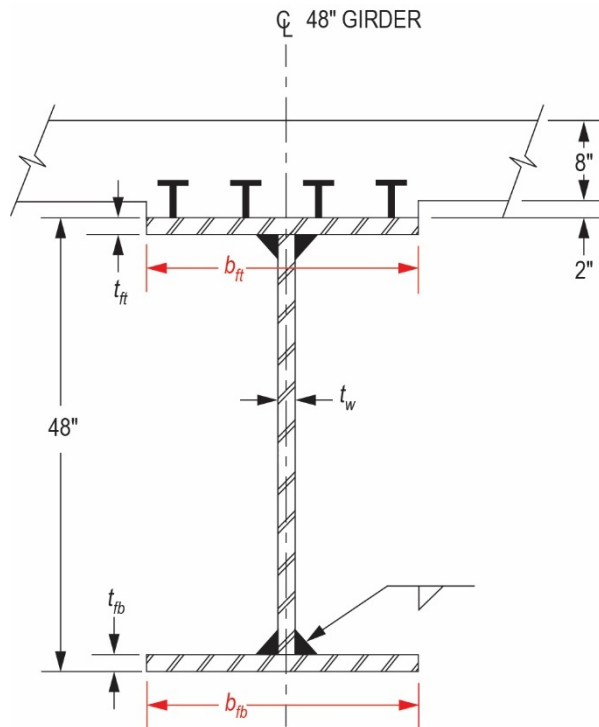
Design Data:

$$t_{ft} = t_{fb} = 1 \frac{1}{2} \text{ in.}$$

$$b_{ft} = b_{fb} = 18 \text{ in.}$$

$$n = 8.5$$

$$t_w = 5/8 \text{ in.}$$



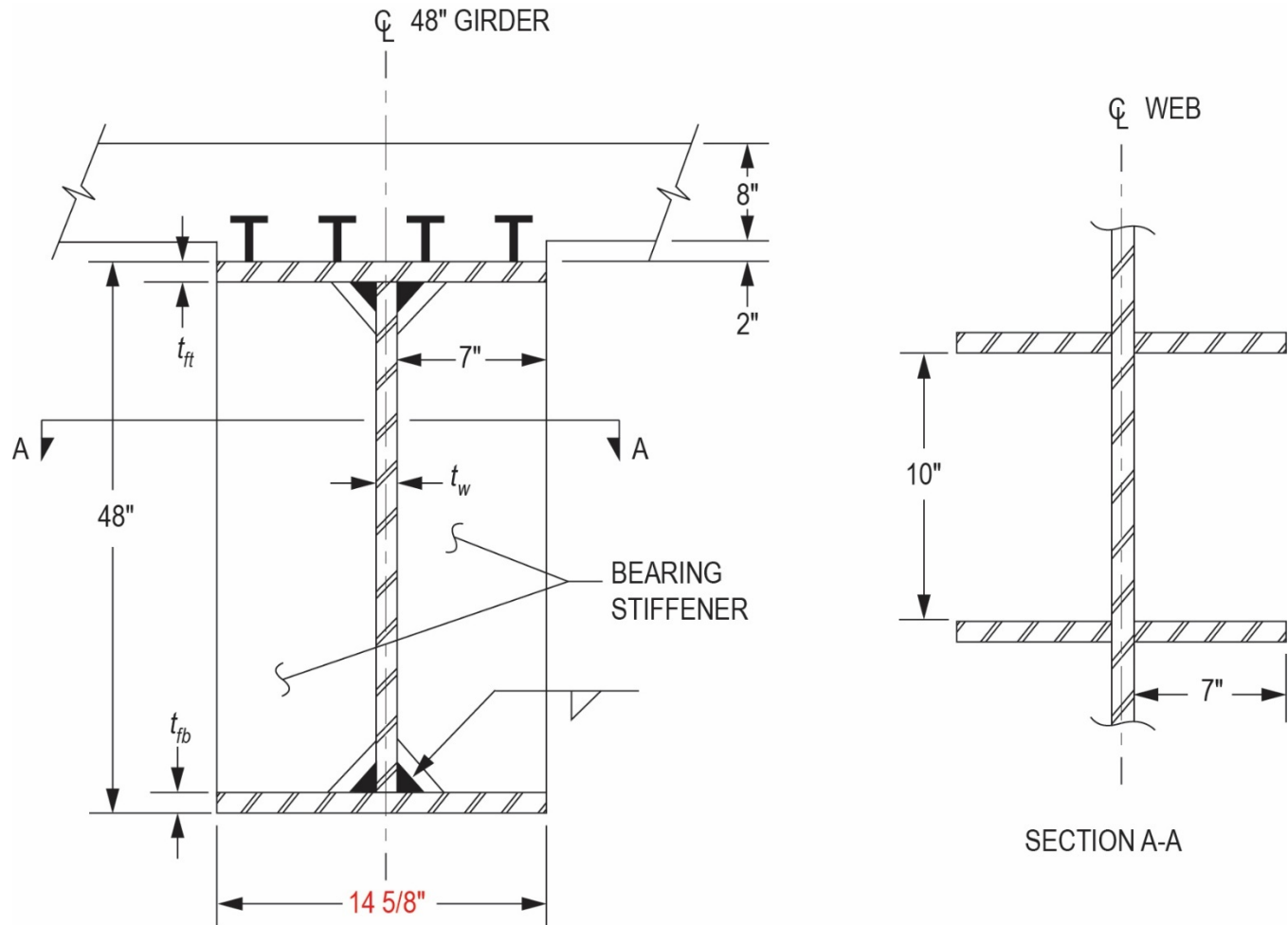
Vertical Forces Depth—Bridges, Scenario 1, Question 6, p. 81

For the **Fatigue I** limit state, the pitch (in.) required for four 3/4-in.-diameter shear studs is

_____.

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Vertical Forces Depth—Bridges, Scenario 1, Question 7, p. 82

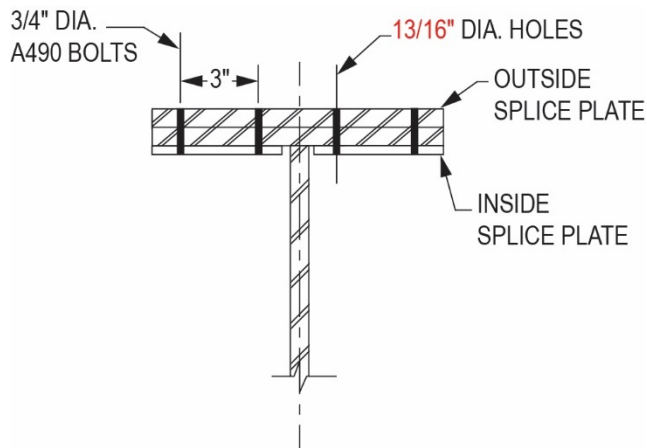


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Vertical Forces Depth—Bridges, Scenario 1, Question 8, p. 83

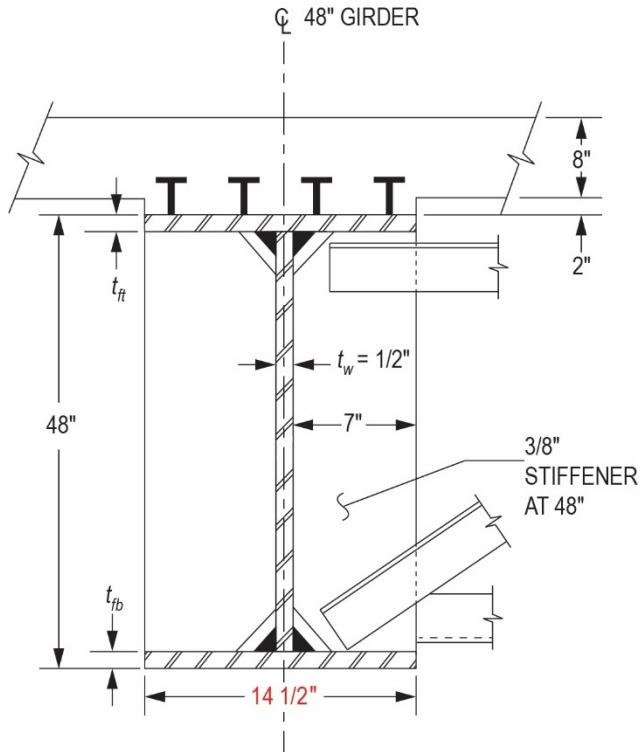
Variable	Value
A_g (in ²)	0.8
A_e (in ²)	0.95
A_n (in ²)	19.125
F_{yf} (ksi)	20.94
F_u (ksi)	24
P_{fy} (ksi)	50
ϕ_u	65
ϕ_y	1,047

Vertical Forces Depth—Bridges, Scenario 1, Question 11, p. 85



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Vertical Forces Depth—Bridges, Scenario 1, Question 12, p. 86



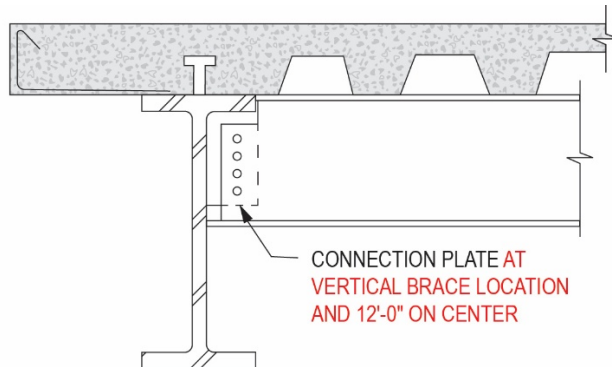
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Lateral Forces Depth—Buildings, Scenario 1, 12, p. 205

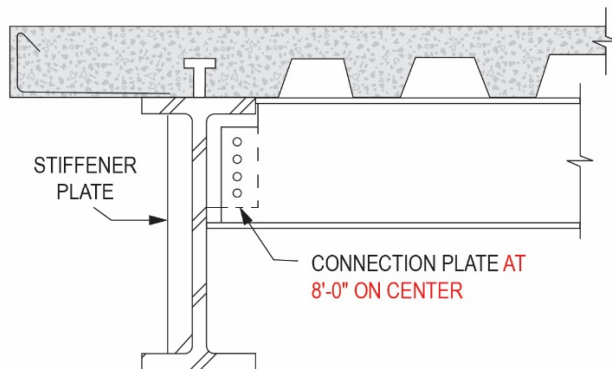
Which of the following designs satisfy the stability bracing requirements at a W24×55 braced frame beam?

Select the **two** that apply.

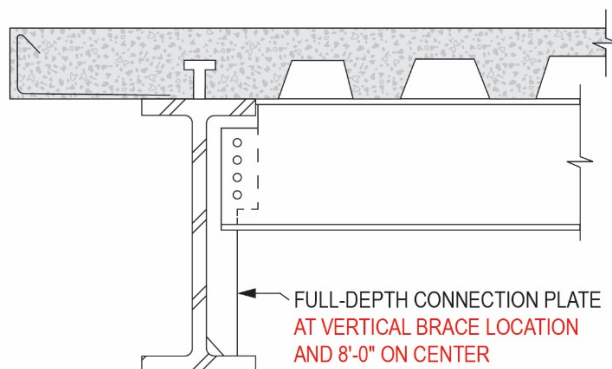
Option A



Option B



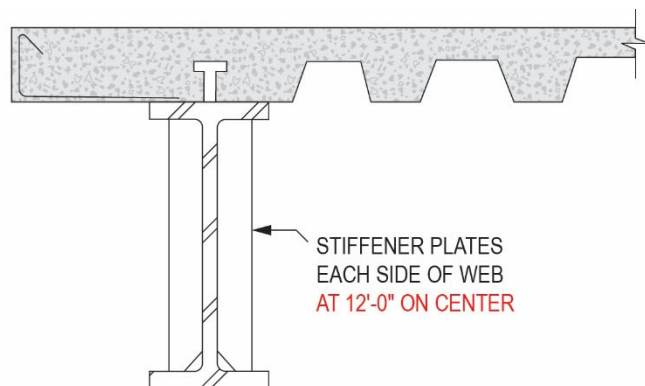
Option C



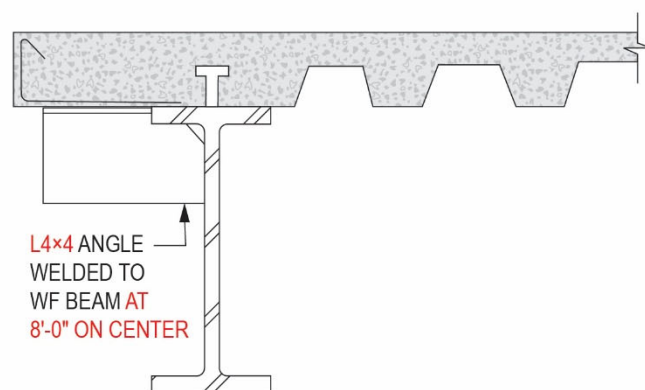
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Lateral Forces Depth—Buildings, Scenario 1, 12, p. 205 (Continued)

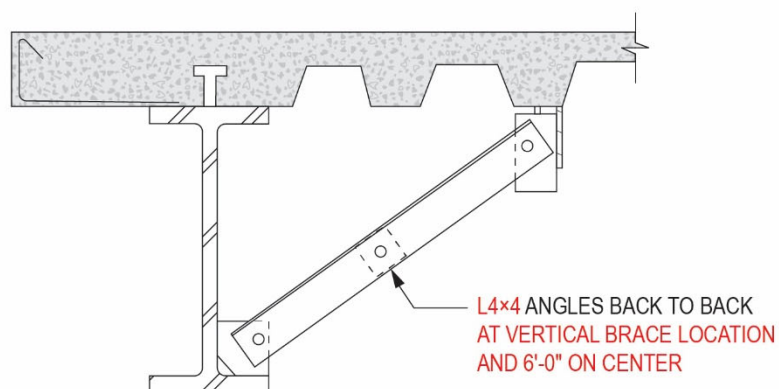
Option D



Option E



Option F



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Lateral Forces Depth—Buildings, Scenario 2, Question 2, p. 210

Based on a wind velocity pressure, q_p , of 50 psf and vertical reinforcement at 16 in. o.c., the design wind pressure (psf), p , for components and cladding elements at the top of the windward parapet at Grid Line 5 is _____.

Enter your response in the blank.

Lateral Forces Depth—Buildings, Scenario 2, Question 3, p. 210

Based on a wind velocity pressure of 50 psf and vertical reinforcement at 16 in. o.c., the building's leeward parapet design wind pressure (psf), p , for components and cladding elements at Grid Line 5 is _____.

Enter your response in the blank.

Lateral Forces Depth—Buildings, Scenario 2, Question 8, p. 212

Assumptions:

Same size nails used throughout

For a distributed diaphragm wind load W of 2,345 plf (strength/LRFD level) and based on 2× joists and blocking at all panel edges, what are the minimum required horizontal plywood nail size and spacing, respectively, along Grid Lines 7 and 8 as shown?

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Lateral Forces Depth—Buildings, Scenario 2, Question 11, p. 215

The contractor is requesting a substitution for a light gauge column cap due to a change in the beam-to-column connector. Determine the **minimum** connection components necessary to resist 1,000-lb (ASD) wind uplift at the beam-to-post and post-to-concrete connections shown.

Match the correct component to each location on the diagram. Some components will be used more than once.

NOTE: JOISTS NOT SHOWN FOR CLARITY

Diagram illustrating a structural connection for wind uplift resistance. The diagram shows a beam-to-post connection and a post-to-concrete connection. The diagram includes a central vertical post, a horizontal beam, and a concrete foundation. Various connection points are indicated with dashed boxes and arrows. The word "AND" appears twice, indicating multiple components may be used at a single location.

18-GAUGE ANGLE WITH NAILS (3) 16d	STEEL PLATE WITH THROUGH BOLTS	STEEL PLATE COLUMN CAP W/ THROUGH BOLTS	HORIZONTAL STRAP WITH NAILS
1/4" STEEL PLATE POST BASE WITH (2) 3/4" THROUGH BOLTS	18-GAUGE CAP WITH (3) 16d EACH SIDE OF BEAM	12-GAUGE POST BASE WITH (3) 16d NAILS FRONT AND BACK OF POST	

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Vertical Forces Breadth—Solution 11, p. 93

$$P_r = \phi P_n \geq P_u$$

$$P_n = 0.85 f_c' A_1 m$$

AASHTO 5.6.5-1

AASHTO 5.6.5-2

Vertical Forces Breadth—Solution 26, p. 105

$$k = \sqrt{(np^2) + 2np} - np = 0.207$$

Vertical Forces Depth—Buildings, Scenario 1, Solution 1, p. 110

Reference: ACI 318, Section 7.3.1.1

Slab support condition is both end continuous.

$$\ell_n = 30'-0" - 18" = 28'-6" = 342"$$

$$\text{Minimum slab thickness, } h = \frac{\ell}{28} = \frac{342"}{28} = 12.21"$$

THE CORRECT ANSWER IS: 12 to 13

Alternate solution:

ACI does allow for center-to-center spacing to be used (ℓ).

Slab support condition is both end continuous.

$$\ell_n = 30'-0" = 360"$$

$$\text{Minimum slab thickness, } h = \frac{\ell}{28} = \frac{360"}{28} = 12.86"$$

THE CORRECT ANSWER IS: 12 to 13

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Vertical Forces Depth—Buildings, Scenario 1, Solution 3, p. 111

$L_r = 20$ psf for roof

The value used to calculate trib area for the LL applied to the column at the basement level is calculated as follows:

$$L = L_o \left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}} \right)$$

$$K_{LL} = 4$$

ASCE 7 Sec.4.7-1

$$A_T = 3 \times 900 \text{ ft}^2 = 2,700 \text{ ft}^2 \text{ tributary area}$$

Per ASCE 7, C4.7.1:

"For multiple floors, areas for members supporting more than one floor are summed."

$$L = 100 \left(0.25 + \frac{15}{\sqrt{4 \times 2,700}} \right) = 39.4 \text{ psf}$$

$< 0.4L_o$ therefore use 40 psf for determination of column LL at the basement level

Live loads:

Roof live load = $20 \text{ psf} \times 30 \text{ ft} \times 30 \text{ ft} = 18 \text{ kips}$

$$L = 40 \text{ psf} \times 3 \times 30 \text{ ft} \times 30 \text{ ft} = 108 \text{ kips}$$

ASD (Allowable Stress Design)

ASCE 7 Sec 2.4.1

Load combo

1) $D = 474 \text{ kips}$

2) $D + L = 474 \text{ kips} + 108 \text{ kips} = 582 \text{ kips}$ (governs)

3) $D + L_r = 474 \text{ kips} + 18 \text{ kips} = 492 \text{ kips}$

4) $D + 0.75L + 0.75L_r = 474 \text{ kips} + 0.75(108 \text{ kips}) + 0.75(18 \text{ kips}) = 568.5 \text{ kips}$

Acceptable range: $\geq 575 \text{ kips}$ and $\leq 590 \text{ kips}$

LRFD (Strength Design)

ASCE 7 Sec 2.3.1

Load combo

1) $1.4D = 1.4(474 \text{ kips}) = 663.6 \text{ kips}$

2) $1.2D + 1.6L + 0.5L_r$
 $= 1.2(474 \text{ kips}) + 1.6(108 \text{ kips}) + 0.5(18 \text{ kips}) = 750.6 \text{ kips}$ (governs)

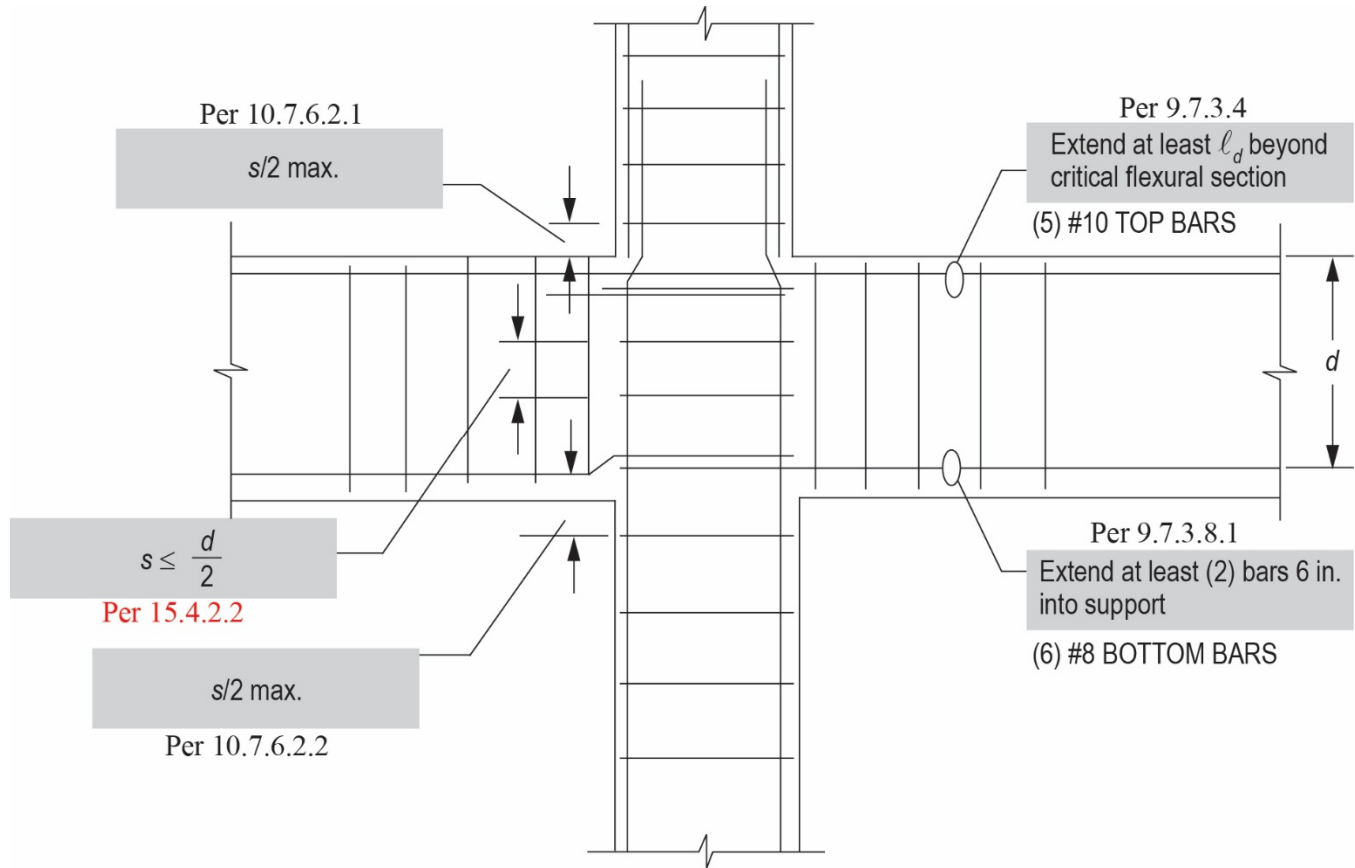
3) $1.2D + 1.6L_r + L$
 $= 1.2(474 \text{ kips}) + 1.6(18 \text{ kips}) + 108 \text{ kips} = 705.6 \text{ kips}$

Acceptable range: $\geq 750 \text{ kips}$ and $\leq 770 \text{ kips}$

THE CORRECT ANSWER IS: ASD 575 to 590
LRFD 750 to 770

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Vertical Forces Depth—Buildings, Scenario 1, Solution 10, p. 117



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Vertical Forces Depth—Buildings, Scenario 1, Solution 11, p. 117

Beam support condition is both end continuous.

ACI 318-14 Sec. 9.3.1.1

$$\ell_n = 30'-0" - 24" = 28'-0" = 336"$$

$$\text{Minimum beam depth, } h = \frac{\ell}{21} = \frac{336"}{21} = 16"$$

THE CORRECT ANSWER IS: 16 to 17.2

Alternate solution:

ACI does allow for center-to-center spacing to be used.

Beam support condition is both end continuous.

$$\ell_n = 30'-0" = 360"$$

$$\text{Minimum beam depth, } h = \frac{\ell}{21} = \frac{360"}{21} = 17.14"$$

THE CORRECT ANSWER IS: 16 to 17.2

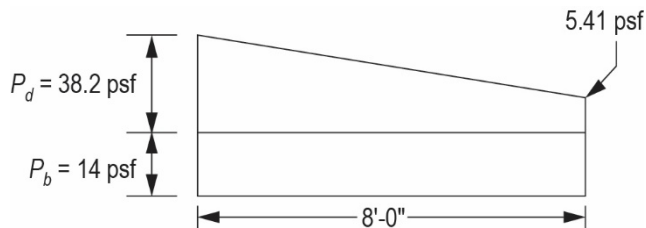
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Vertical Forces Depth—Buildings, Scenario 2, Solution 4, p. 125

$$p_d = (2.33 \text{ ft})(16.6 \text{ pcf}) = 38.2 \text{ psf}$$

$$W = 4h_d = 4(2.33 \text{ ft}) = 9.32 \text{ ft}$$

$$P_b = 14 \text{ psf}$$



$$\left[S = (14 + 5.41) \left(\frac{8 \text{ ft}}{2} \right) \left(\frac{12 \text{ ft}}{2} \right) \right] + \frac{(38.2 - 5.41)}{2} \left(\frac{2}{3} \right) (8 \text{ ft}) \left(\frac{12 \text{ ft}}{2} \right)$$
$$= 1.52 \text{ kips} \leftarrow \text{Governs snow}$$

$$D + L_r = 2.09 + 0.48 = 2.57 \text{ kips}$$

$$D + S = 2.09 + 0.990 = 3.08 \text{ kips} \leftarrow \text{Governs}$$

LRFD (Strength Design)

$$1.2D + 1.6L_r = 3.28 \text{ kips}$$

$$1.2D + 1.6S = 4.09 \text{ kips}$$

THE CORRECT ANSWER IS: ASD 3.0 to 3.2 kips
LRFD 4.0 to 4.2 kips

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Vertical Forces Depth—Buildings, Scenario 2, Solution 7, p. 129

$$\left. \begin{aligned} D &= 16 \text{ psf} (26 \text{ ft}) \left(\frac{16}{12} \right) = 555 \text{ lb (wall)} \\ L_R &= 20 \text{ psf} \left(\frac{8 \text{ ft}}{2} \right) \left(\frac{16}{12} \right) = 107 \text{ lb (roof)} \\ D_R &= 15 \text{ psf} \left(\frac{8 \text{ ft}}{2} \right) \left(\frac{16}{12} \right) = 80 \text{ lb (roof)} \end{aligned} \right\} \Sigma = 742 \text{ lb}$$
$$M = (P_R)(e) = (L_R + D_R)(e)$$
$$M = (P)(e) = (107 + 80)(4.25 \text{ in.}) = 795 \text{ in.-lb}$$

$$\left(\frac{f_c}{F'_c} \right)^2 + \frac{f_b}{F'_b \left(1 - \frac{f_c}{F'_{cE}} \right)} \leq 1.0 \quad (3.9.3) \quad f_b = \frac{M}{S} = \frac{795 \text{ in.-lb}}{7.56 \text{ in}^3} = 105 \text{ psi}$$
$$\left(\frac{90}{181} \right)^2 + \frac{105}{1,006 \left(1 - \frac{90}{182} \right)} = 0.45 \quad f_c = \frac{742}{(1.5)(5.5)} = 90 \text{ psi}$$

LRFD (Strength Design)

$$1.2D + 1.6L_r = 1.2(555 + 107) + 1.6(80) = 922 \text{ lb}$$

$$M = P_e = [1.2(107) + 1.6(80)]4.25 \text{ in.} = 1,090 \text{ in.-lb}$$

THE CORRECT ANSWER IS: ASD 0.42 to 0.48
LRFD 0.38 to 0.42

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Vertical Forces Depth—Bridges, Scenario 1, Solution 3, p. 138

By plastic forces

AASHTO Table D.6.1-2

$$P_{rt} = A_{rt} f_y = 6 (0.44)(60 \text{ ksi}) = 158.4 \text{ kips}$$

6-ft girder spacing, $b_{eff} = 72 \text{ in.}$

6 bars in b_{eff}

$$P_{rb} = A_{rb} f_y = 6 (0.31)(60 \text{ ksi}) = 111.6 \text{ kips}$$

$$T_{flange} = F_y b_{ft} t_{ft} = 50 (16)(1.25) = 1,000 \text{ kips}$$

$$\text{Web} = F_y D t_w = 50 (45.5)(0.5) = 1,137.5 \text{ kips}$$

$$B_{flange} = F_y b_{fb} t_{fb} = 50 (16)(1.25) = 1,000 \text{ kips}$$

$$P_c + P_w \geq P_t + P_{rb} + P_{rt}$$

$$1,000 \text{ kips} + 1,137.5 \text{ kips} \geq 1,000 \text{ kips} + 111.6 \text{ kips} + 158.4 \text{ kips}$$

$$2,137.5 \text{ kips} \geq 1,270 \text{ kips}$$

∴ the plastic neutral axis (PNA) is in the web

Alternate solution:

If $\bar{y}$ is calculated for Case II it will exceed t_f and thus be in the web.

$$\bar{y} = \frac{t_f}{2} \left[\frac{P_w + P_c - P_{rt} - P_{rb}}{P_t} + 1 \right]$$

$$\bar{y} = \frac{1.25}{2} \left[\frac{1,137.5 \text{ kips} + 1,000 \text{ kips} - 158.4 \text{ kips} - 111.6 \text{ kips}}{1,000 \text{ kips}} + 1 \right] = 1.7922$$

1.7922 in. > 1.25 in.; therefore PNA is below the top flange and in the web.

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Vertical Forces Depth—Bridges, Scenario 1, Solution 4, p. 139

Reference: AASHTO 6.10.1.1b

Component	Area	Y	Ay	$A(y - \bar{y})^2$	$I_o = \left(\frac{bd^3}{12} \right)$ $b = b_E/3n$
Slab	$(72)(8)/25.5 = 22.6 \text{ in}^2$	54 in.	$1,220.4 \text{ in}^3$	$22.6(54 - 30.72)^2 = 12,248.26 \text{ in}^4$	120.47 in^4
Fillet	$(18)(2)/25.5 = 1.41 \text{ in}^2$	49 in.	69.09 in^3	$1.41(49 - 30.72)^2 = 471.16 \text{ in}^4$	0.47 in^4
Top Flange	$(18)(1.5) = 27 \text{ in}^2$	47.25 in.	$1,275.75 \text{ in}^3$	$27(47.25 - 30.72)^2 = 7,377.50 \text{ in}^4$	5.06 in^4
Web	$(0.625)(45) = 28.125 \text{ in}^2$	24 in.	675 in^3	$28.125(24.0 - 30.72)^2 = 1,270.08 \text{ in}^4$	$4,746.09 \text{ in}^4$
Bottom Flange	$(18)(1.5) = 27 \text{ in}^2$	0.75 in.	20.25 in^3	$27(0.75 - 30.72)^2 = 24,251.42 \text{ in}^4$	5.06 in^4
$\Sigma 106.14 \text{ in}^2$			$3,260.49 \text{ in}^3$	$45,618.42 \text{ in}^4$	$4,877.15 \text{ in}^4$

$$3n = 3 \times 8.5 = 25.5$$

$$\bar{y} = \frac{3,260.49}{106.14} = 30.72 \text{ in.}$$

$$I_{LT} = 45,618.42 \text{ in}^4 + 4,877.15 \text{ in}^4 = 50,495.57 \text{ in}^4$$

$$S_{LT}^b = \frac{50,495.57 \text{ in}^4}{30.72 \text{ in.}} = 1,643.74 \text{ in}^3$$

$$f_{bLT} = 1,200 / 1,651.61 \text{ in}^3 \left(\frac{12 \text{ in.}}{\text{ft}} \right) = 8.76 \text{ ksi}$$

THE CORRECT ANSWER IS: 8.25 to 9.25

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Vertical Forces Depth—Bridges, Scenario 1, Solution 5, p. 140

Weld E = 1/4 in.

3/4-in. stiffener to 5/8-in. web

3/4-in. stiffener controls

Use 1/4-in. weld

Vertical Forces Depth—Bridges, Scenario 1, Solution 6, p. 141

Find Z_r

$$Z_r = 5.5 d^2 \quad 6.10.10.2-1$$

$$5.5(0.75)^2 = 3.09 \text{ kips}$$

F_{fat} = zero (straight spans and no skews)

$$V_{fat} = \frac{V_f Q}{I} = \frac{(58 \text{ kips})(1,005.63 \text{ in}^3)}{62,200 \text{ in}^4} = \mathbf{0.938} \quad 6.10.10.1.2-3$$

$$V_{fat} = V_{sr} \quad 6.10.10.1.2-2$$

$$p \leq \frac{nZ_r}{V_{sr}} = \frac{4(\mathbf{3.09})}{0.938} = 13.17 \text{ in.} \quad 6.10.10.1.2-1$$

Pitch is less than 48 and greater than $6(0.75) = 4.5$ OK

Use **13.0** in. for pitch and four studs per row.

THE CORRECT ANSWER IS: 12.8 to 13.4

Vertical Forces Depth—Bridges, Scenario 1, Solution 7, p. 142

$$(4)(\mathbf{0.625} \text{ in.})(7 - 2.5) = 11.25 \text{ in}^2 < 12.86 \text{ in}^2 \quad (\text{use } 3/4 \text{ in.})$$

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Vertical Forces Depth—Bridges, Scenario 1, Solution 8, p. 143

$$A_g = (16 \text{ in.})(1.5 \text{ in.}) = 24 \text{ in}^2$$

$$A_e = \frac{0.8 (65)}{0.95 (50)} (19.125) = 20.94 \text{ in}^2$$

AASHTO 6.13.6.1.3 b-2

$$A_n = \left[16 - 4 \left(\frac{13}{16} \right) \right] (1.5) = 19.125 \text{ in}^2$$

↙ Hole for 3/4-in.-diameter bolt

Table 6.13.3.4-1

$$F_{yf} = 50 \text{ ksi}$$

Table 6.4.1-1

$$F_u = 65 \text{ ksi}$$

Table 6.4.1-1

$$P_{fy} = A_e F_{yf} = 1,047 \text{ kips}$$

$$\phi_u = 0.8$$

6.5.4.2

$$\phi_y = 0.95$$

6.5.4.2

Vertical Forces Depth—Bridges, Scenario 1, Solution 9, p. 143

Reference: AASHTO 6.13.6.1.3c

Option H was removed

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Vertical Forces Depth—Bridges, Scenario 1, Solution 12, p. 145

$$I_{t1} = b t_w^3 J \quad \text{AASHTO 6.10.11.1.3}$$

$$J = \frac{2.5}{(d_o/D)^2} - 2.0 \geq 0.5 \quad d_o = 48 \text{ in.} \quad D = 46 \text{ in.} \quad 6.10.11.1.3-1$$

$$J = \frac{2.5}{(1.04)^2} - 2.0 \geq 0.5$$

$$\mathbf{0.311} \geq 0.5 \quad \text{use } 0.5$$

$$I_{t1} = 46 \text{ in.} (0.5)^3 (0.5) = 2.875$$

$$I_{t2} = \frac{D^4 \rho_t^{1.3}}{40} \left(\frac{F_{yw}}{E} \right)^{1.5} \quad 6.10.11.1.3-4$$

$$\rho_t = \frac{50}{25.8} = 1.94 > 1.0$$

$$\rho_t = 1.94$$

$$F_{crs} = \frac{0.31 E}{\left(\frac{b_t}{t_p} \right)^2} \leq F_{ys} = \frac{0.31 (29,000)}{\left(\frac{7}{0.375} \right)^2} = 25.8 \text{ ksi} \quad 6.10.11.1.3-6$$

$$I_{t2} = \frac{(46)^4 1.94^{1.3}}{40} \left(\frac{50}{29,000} \right)^{1.5} = 18.97$$

$$I_{t \text{ (pair)}} = \frac{2}{12} (0.375) (7)^3 + \mathbf{2(.375)(7) \left(\frac{7}{2} + \frac{0.5}{2} \right)^2} = \mathbf{95.27 \text{ in}^4}$$

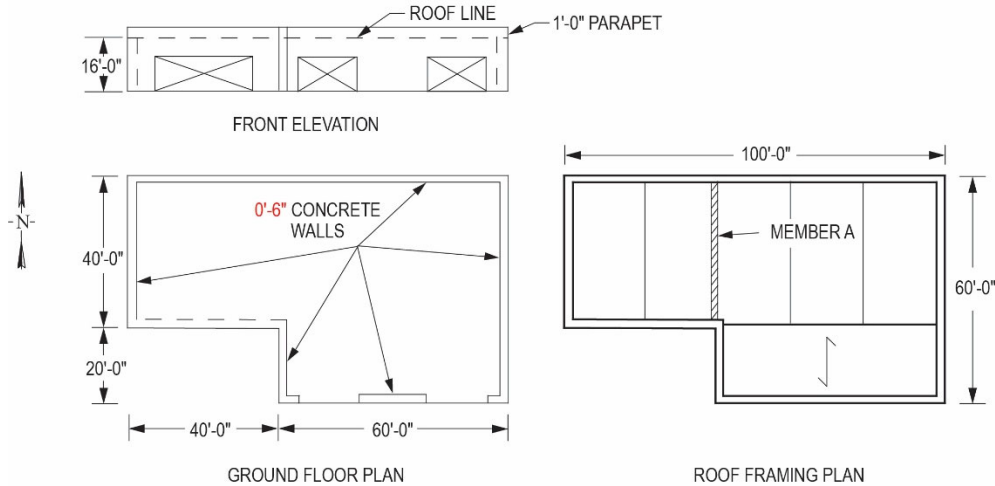
$$I_t \geq I_{t1} \quad \therefore \text{OK}$$

$$I_t \geq I_{t2} \quad \therefore \text{OK}$$

THE CORRECT ANSWER IS: 94.0 to 96.0

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Lateral Forces Breadth—Question 7, p. 160



Lateral Forces Breadth—Question 16, p. 173

The figure shows a **special concentrically** braced frame connection at the beam/brace location.

Lateral Forces Breadth—Question 19, p. 174

Design Data:

Table 1 shows computer output for all frame members.

Special steel concentric braced frame

$F_y = 50$ ksi for HSS

$F_y = 50$ ksi for WF

$E = 29,000$ ksi

ASTM A500 Gr. C

Lateral Forces Breadth—Question 22, p. 177

☐ C. **48**

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Lateral Forces Depth—Buildings, Scenario 1, Figure C, p. 193

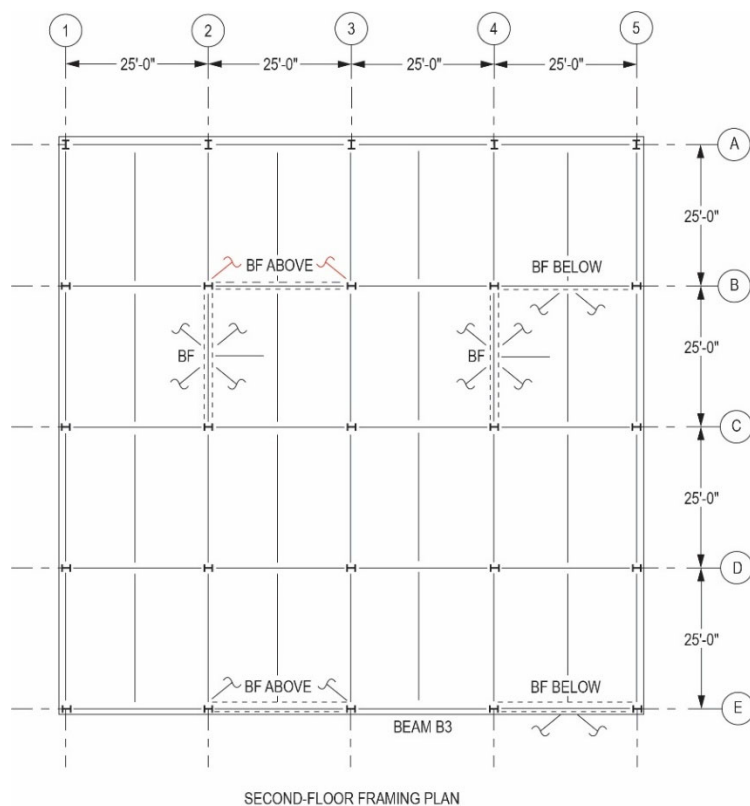


FIGURE C

Lateral Forces Depth—Buildings, Scenario 1, Question 2, p. 198

Based on the following information, the lightest W36 beam for Beam B1 in Figure E is _____.

Lateral Forces Depth—Buildings, Scenario 1, Question 3, p. 198

Based on the story forces provided in Table 1 and effective brace length $KL = 18$ ft, the lightest round hollow steel section using a round HSS 7.500 brace member between the sixth floor and roof is:

Lateral Forces Depth—Buildings, Scenario 1, Question 4, p. 199

Which of the elements identified in the figure are required to be designed for overstrength?

Lateral Forces Depth—Buildings, Scenario 1, Question 5, p. 200

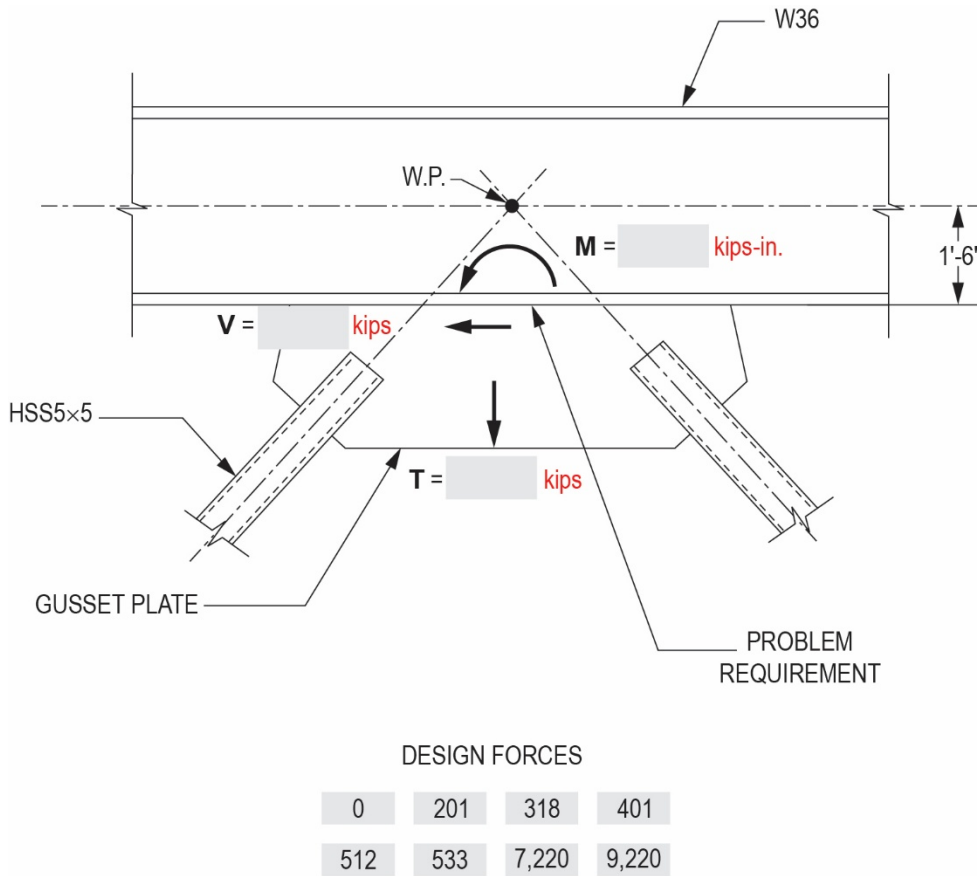
Based on story forces provided in Table 1, the governing axial force for Beam B3 in Figure F is:

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Lateral Forces Depth—Buildings, Scenario 1, Question 6, p. 200

For this question, consider bolt failure states only. Based on the figure, the number of bolts required to transfer the axial force to the special concentric brace frame is:

Lateral Forces Depth—Buildings, Scenario 1, Question 7, p. 201



Lateral Forces Depth—Buildings, Scenario 1, Question 10, p. 204

- ☐ A. Attachment of steel deck using steel headed stud anchors

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Lateral Forces Depth—Buildings, Scenario 1, Solution 12, p. 253

$r_y = 1.34 \text{ in.}$

Spacing $< L_b$

One set of braces required at the point of intersection of the V- or inverted V-braced frames

$$L_b = \frac{0.17r_y E}{R_y F_y}$$

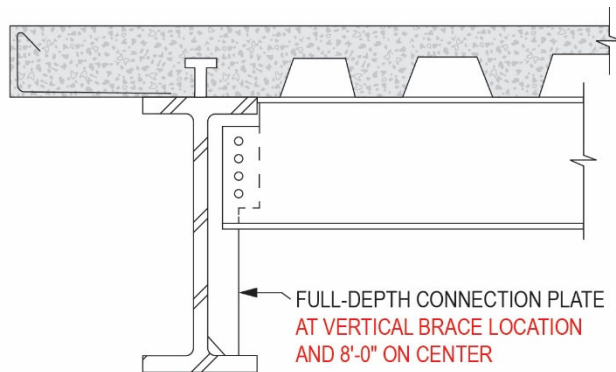
$L_b = 120 \text{ in.} = 10 \text{ ft}$

AISC SDM Sec. D1.2a

AISC SDM Sec. F2.4b

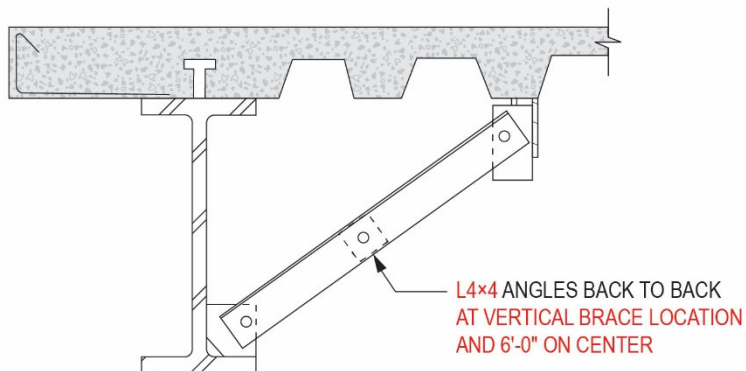
AISC SDM Eq. D1-2

Option C



AISC SDM, AISC 341
Sec. F2-4b(b); Sec. D1-2a

Option F



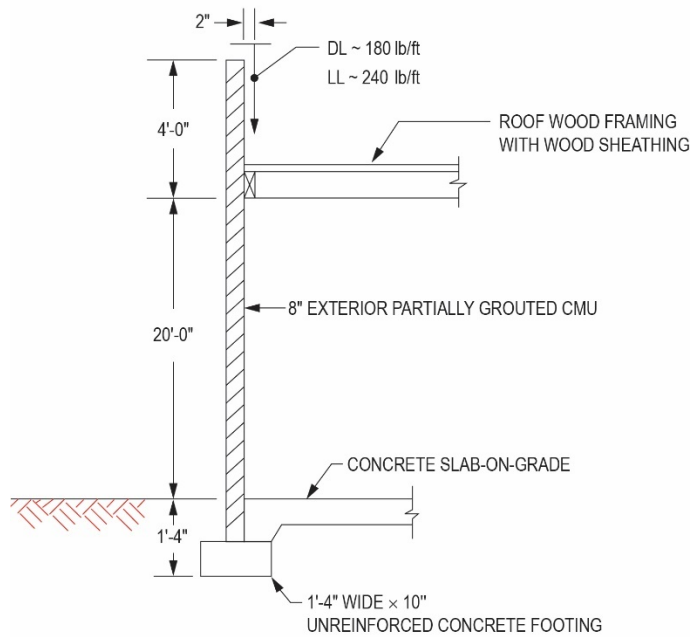
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Lateral Forces Depth—Buildings, Scenario 2, p. 206

Material Specifications:

Steel reinforcement ASTM A615, $f_y = 60$ ksi

Lateral Forces Depth—Buildings, Scenario 2, Figure A, p. 207



WALL SECTION

FIGURE A

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Lateral Forces Depth—Buildings, Scenario 2, Figure B, p. 208

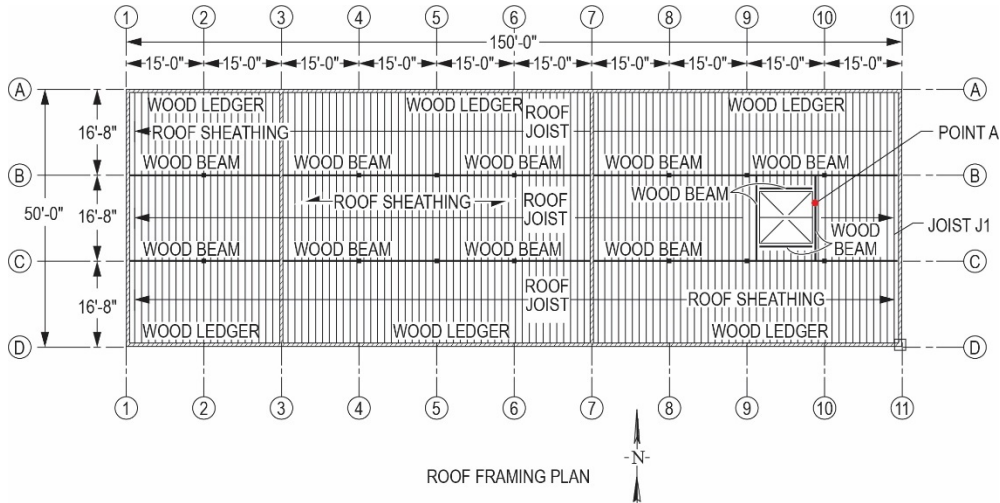


FIGURE B

Lateral Forces Depth—Buildings, Scenario 2, Figure C, p. 209

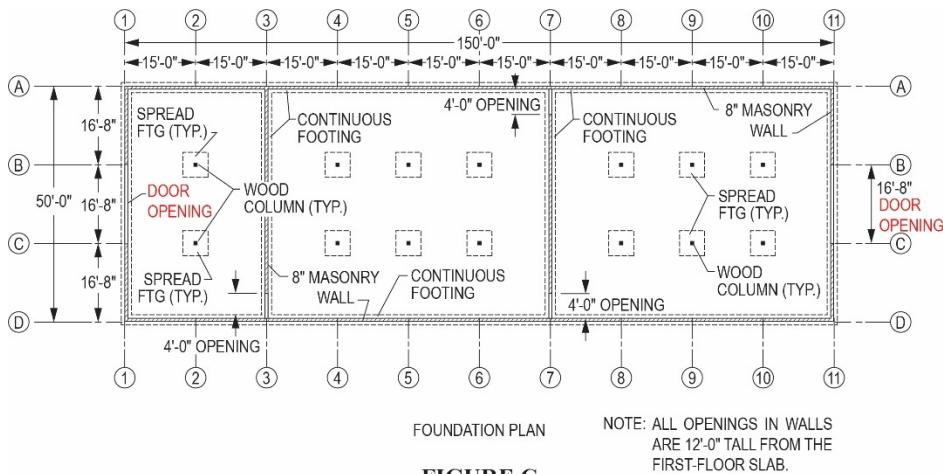


FIGURE C

Lateral Forces Depth—Buildings, Scenario 2, Question 1, p. 210

Design Data:

Base wind speed = 142 mph

Exposure C

$K_{zt} = 1.0$

$K_e = 1.0$

The wind velocity pressure q_p (psf) at the top of the building's parapet is _____.

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Lateral Forces Depth—Buildings, Scenario 2, Question 2, p. 210

Based on a wind velocity pressure of 50 psf and vertical reinforcement at 16 in. o.c., the wind design pressure p (psf) at the top of the windward parapet at Grid Line 5 is _____.

Lateral Forces Depth—Buildings, Scenario 2, Question 3, p. 210

Based on a wind velocity pressure of 50 psf and vertical reinforcement at 16 in. o.c., the building's leeward parapet design wind pressure p (psf) at Grid Line 5 is _____.

Lateral Forces Depth—Buildings, Scenario 2, Question 4, p. 211

Assumptions:

$q_z = 35$ psf at 15 ft above floor slab

$q_h = 40$ psf at 20 ft above floor slab

Total leeward parapet design wind pressure = 85 psf

Anchorage from the wall to the roof has an effective wind area of 50 ft².

Enclosed building

The wind load R_B (plf on roof) from the I windward wall at Grid Line 5 that is resisted by the roof diaphragm is _____.

Enter your response in the blank. Positive and negative values signify forces acting toward and away from the wall surface, respectively.

Lateral Forces Depth—Buildings, Scenario 2, Question 5, p. 211

Based on the wind pressures in the table, the maximum moment M_{max} (lb-ft) due to wind pressures on the wall shown in Figure A is _____.

Lateral Forces Depth—Buildings, Scenario 2, Question 6, p. 211

Loading criteria:

$M = 1,080$ ft-lb per foot (ASD)

$M_u = 1,080$ ft-lb per foot (LRFD)

Assumptions:

Ignore axial loading

Based on Figure A, the maximum spacing of #5 vertical bar reinforcement for bars placed at the center of the 8-in. CMU wall is:

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Lateral Forces Depth—Buildings, Scenario 2, Question 7, p. 212

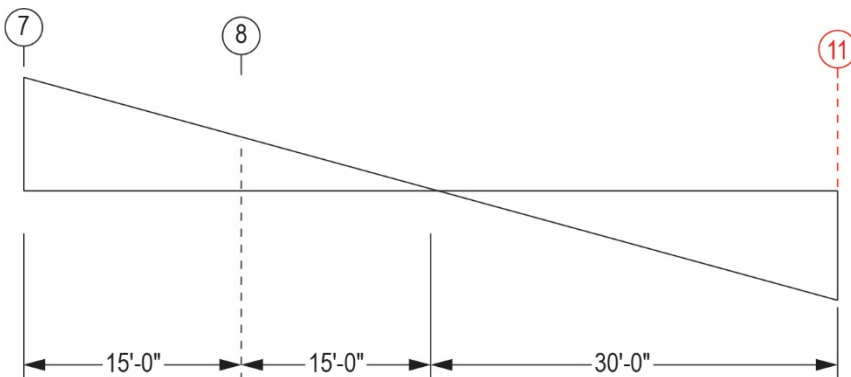
Assumptions:

- $q_h = 46$ psf
- Enclosed building
- Bridging/blocking fully braces bottom of joists.
- Joists are spaced at 16 in. o.c.

The **minimum** 2× wood joist size required for bending stresses **due to wind pressure on** Joist J1 on the roof framing plan in **Figure B** is _____.

Lateral Forces Depth—Buildings, Scenario 2, Question 8, p. 212

For a distributed diaphragm wind load W of 2,300 plf (strength/LRFD level) and based on 2× joists and blocking at all panel edges, what are the required horizontal plywood nail size and spacing, respectively, along Grids **7** and **8**?



Lateral Forces Depth—Bridges, Scenario 1, Question 3, p. 221

Assume half the mass of the columns contributes to the weight of the structure. The weight (kips) of the structure used to compute the period of the structure is _____.

Lateral Forces Depth—Bridges, Scenario 1, Question 10, p. 224

Consider a resultant elastic transverse EQ load only applied at the center of the bent beam. Which of the following statements are accurate when computing forces at the bottom of a Pier 2 column?

Lateral Forces Breadth—Solution 19, p. 238

$R_y = 1.3$ HSS ASTM A500 Grade C

AISC SDM Table **A3.2**

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Lateral Forces Breadth—Solution 22, p. 240

$$\therefore \lambda_{rc} = \frac{1.27 \text{ in.}}{2.5 + 0.5} = 0.42$$

$$K_{tr} = 40 A_{tr} / (sn)$$

$$A_{tr} = 0.2 \text{ in}^2; s = 4 \text{ in.}; n = 4$$

$$K_{tr} = 40(0.2) / 4(4) = 0.5$$

$$\begin{aligned} \therefore l_d &= 91.4 \text{ in.} \left(\frac{1.0 \times 1.0 \times 0.42 \times 1.0}{1.0} \right) \\ &= 38.39 \text{ in.} \times 1.25 = 47.985 \text{ in.} \sim 48 \text{ in.} \end{aligned}$$

AASHTO 5.11.4.3

Lateral Forces Depth—Buildings, Scenario 1, Solution 3, p. 247

Round HSS 7.500 × 0.375 is correct.

$$P_u = \frac{(120/2)}{\cos(46.1^\circ)} = 86.5 \text{ kips (LRFD)}; P = \frac{0.7(120/2)}{\cos(46.1^\circ)} = 60.6 \text{ kips (ASD)}$$

$$\phi P_n = 199 \text{ kips (LRFD)};$$

$$P_{all} = 132 \text{ kips (ASD)}$$

$$D/t = < 25.7$$

Selecting a round HSS 7.500 × 0.250 or HSS 7.500 × 0.312 is not acceptable.

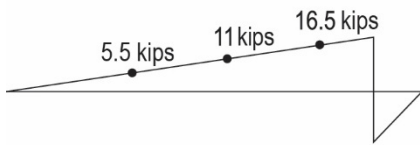
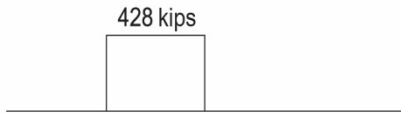
Lateral Forces Depth—Buildings, Scenario 1, Solution 4, p. 248

Type 4 in-plane discontinuity in vertical lateral force-resisting element irregularity

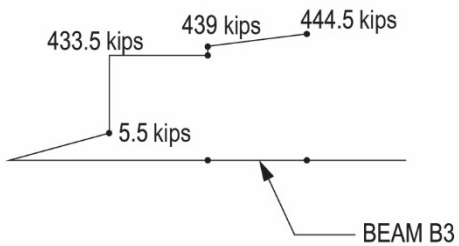
See Section 12.3.3.3

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Lateral Forces Depth—Buildings, Scenario 1, Solution 5, p. 248



$$\text{Unit Shear} = \frac{22 \text{ kips}}{100 \text{ ft}} = 220 \text{ plf}$$



$$\text{LRFD} \quad \begin{aligned} \text{min} &= 439 \text{ kips} \\ \text{max} &= 444.5 \text{ kips} \end{aligned}$$

$$\text{ASD} \quad \begin{aligned} \text{min} &= 0.7 (439) = 307.3 \text{ kips} \\ \text{max} &= 0.7 (444.5) = 311.2 \text{ kips} \end{aligned}$$

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Lateral Forces Depth—Buildings, Scenario 1, Solution 6, p. 249

A section of the solution was removed.

Reference: AISC Steel Construction Manual

1 1/8 in. ϕ , A490X bolts given, $A_{bolt} = 0.994 \text{ in}^2$

Using AISC Table 7-1, Group B, $V_{ASD} = 41.7 \text{ kips}$, $= V_{LRFD} = 62.6 \text{ kips}$

AISC Table 7-17: $F_U = 150 \text{ ksi}$, $F_V = 84 \text{ ksi}$

AISC Table J3.2: $R_n = F_v A_{bolt}$, $R_n/\Omega = 41.7 \text{ kips}$, $\phi R_n = 62.6 \text{ kips}$

ASD (Allowable Design)

ASD = $557 \text{ kips (given)} / 41.7 \text{ kips/bolt} = 13.4 \approx 14 \text{ bolts}$

LRFD (Strength Design)

LRFD = $796 \text{ kips (given)} / 62.6 \text{ kips/bolt} = 12.7 \approx 13 \text{ bolts}$

THE CORRECT ANSWER IS: **ASD 14**
 LRFD 13

Lateral Forces Depth—Buildings, Scenario 2, Solution 1, p. 254

142 mph, Exposure **C**

RC II, parapet 20–24 ft

$$q_p = 0.00256(K_z)(K_{zt})(K_d)(K_e)(V)^2$$

at z of 20 ft, $K_z = 0.90$; at z of **24** ft, $K_z = 0.932$

$K_{zt} = 1.0$, $K_d = 0.85$, $K_e = 1.0$

$$q_p = 0.00256(0.932)(1.0)(0.85)(1.0)(142)^2 = 40.9 \text{ psf}$$

Range: **40.5 to 41.5**

THE CORRECT ANSWER IS: 40.5 to 41.5

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Lateral Forces Depth—Buildings, Scenario 2, Solution 2, p. 254

Effective wind area = $4(4)/3 = 5.33 < 10 \text{ ft}^2$; use 10 ft^2

Wall Zone 4 positive coefficient = +1.0

Roof Zone 2 uplift pressure coefficient = -2.3

$GC_{pi} = 0$ for solid parapet

$q_p = 50$ psf, given

$$p = q_p[(GC_p) - (GC_{pi})] = 50[1.0(0.9) - 0] - 50(-2.3 - 0) = 50(3.2)$$

$p = 160$ psf

Footnote 5, Figure 30.3-1

Figure 30.3-1

Figure 30.3-2A

THE CORRECT ANSWER IS: 155 to 165

Lateral Forces Depth—Buildings, Scenario 2, Solution 3, p. 254

$q_2 = 50$ psf

$GC_{pi} = 0.00$ (solid parapet)

Effective wind area = $4.0 \text{ ft} \times 4.0 \text{ ft}/3 = 5.33 \text{ ft}^2$

$GC_p =$ Figure 30.3-1

Reference Footnote 5, Figure 30.3-1

$$= -1.0 \times 0.9 = +0.9 = GC_p \text{ for } p_{\text{roof}}$$

$$-1.1 \times 0.9 = -0.99 = GC_p \text{ for } p_{\text{wall}}$$

$$p_{\text{roof}} = 50 \text{ psf } (0.9 - 0) = 45 \text{ psf}$$

$$p_{\text{wall}} = 50 \text{ psf } (0.99 - 0) = 49.5 \text{ psf}$$

$$p_{\text{total}} = 94.5 \text{ psf}$$

THE CORRECT ANSWER IS: 90.0 to 99.0

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Lateral Forces Depth—Buildings, Scenario 2, Solution 4, p. 255

$$GC_{pi} = 0.18 \pm$$

Find GC_p :

Figure 30.3-1: (4) = Range of **0.85** to **0.95**

Footnote 5, reduce GC_p by 10%

$$0.85 \times 0.9 = 0.76 - 0.18 = 0.94$$

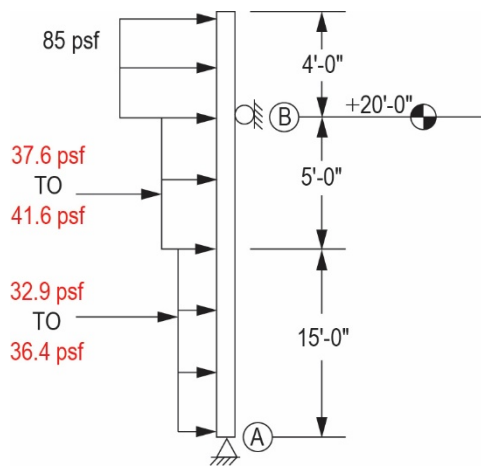
$$0.95 \times 0.9 = 0.86 - 0.18 = 1.04$$

Design wind pressure P at wall below roof:

$$0-15 \text{ ft} = 35 \text{ psf} \left(\frac{0.94}{1.04} \right) = \begin{matrix} 32.9 \text{ psf} \\ \text{to} \\ 36.4 \text{ psf} \end{matrix}$$

$$15-20 \text{ ft} = 40 \text{ psf} \left(\frac{0.94}{1.04} \right) = \begin{matrix} 37.6 \text{ psf} \\ \text{to} \\ 41.6 \text{ psf} \end{matrix}$$

Parapet wind pressure given as 85 psf



$$\Sigma M_A = 0:$$

$$R_B(20 \text{ ft}) = (85 \text{ psf})(4.0 \text{ ft})(22 \text{ ft})$$

+

$$\left(\frac{37.6}{41.6} \right) (5.0 \text{ ft})(17.5 \text{ ft})$$

+

$$\left(\frac{32.9}{36.4} \right) (15 \text{ ft})(7.5 \text{ ft})$$

$$20R_B = 7,480 + \left(\frac{3,290}{3,640} \right) + \left(\frac{3,701}{4,095} \right)$$

$$20R_B = \left(\frac{14,471}{15,215} \right)$$

$$R_B = 723 \text{ to } 761 \text{ plf}$$

THE CORRECT ANSWER IS: 715 to 775

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Lateral Forces Depth—Buildings, Scenario 2, Solution 6, p. 256
ASD (Allowable Design)

From *NCEES PE Structural Reference Handbook*:

$$2/jk = [12 (3.81)^2(900)]/[(1,080)(12)] = 12.1$$

$$n\rho = 0.019$$

$$npj = [16.11(1,080)(12)]/[12(3.81)^2(32,000)] = 0.037$$

$$n\rho = 0.041$$

$$0.041 > 0.019$$

$$\text{At 16 in. o.c.: } A_s = 0.041(16)(3.81)/16.11 = 0.15 \text{ in}^2$$

$$24 \text{ in. o.c.: } A_s = 0.041(24)(3.81)/16.11 = 0.23 \text{ in}^2$$

$$32 \text{ in. o.c.: } A_s = 0.041(32)(3.81)/16.11 = 0.30 \text{ in}^2 < 0.31 \text{ in}^2$$

LRFD (Strength Design)

From *NCEES PE Structural Reference Handbook*:

$$R = M_u/\phi bd^2$$

$$R = (1,800 \times 12)/[0.9(12)(3.81)^2] = 137.8$$

$$\text{At } R = 137.8, \rho = 0.0024$$

$$\text{At 16 in. o.c.: } A_s = 0.0024(16)(3.81) = 0.15 \text{ in}^2$$

$$24 \text{ in. o.c.: } A_s = 0.0024(24)(3.81) = 0.22 \text{ in}^2$$

$$32 \text{ in. o.c.: } A_s = 0.0024(32)(3.81) = 0.29 \text{ in}^2 < 0.31 \text{ in}^2$$

$$40 \text{ in. o.c.: } A_s = 0.0024(40)(3.81) = 0.37 \text{ in}^2$$

THE CORRECT ANSWER IS: ASD: 31 to 33
LRFD: 31 to 33

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Lateral Forces Depth—Buildings, Scenario 2, Solution 7, p. 257

Effective wind area need not be less than $(16.67)(16.67/3) = 93 \text{ ft}^2$

Therefore, use effective wind area = 93 ft^2

ASCE 7 Figure 30.3-2A, Zone 2: $G C_p$ can be taken with range of -1.7 to -1.9

Design wind uplift pressure, LRFD/Strength: $P = 46 \text{ psf} \left(\begin{matrix} -1.7 & -0.18 \\ -1.9 & -0.18 \end{matrix} \right) = \begin{matrix} 86.5 \text{ psf} \\ \text{to} \\ 95.7 \text{ psf} \end{matrix}$

Distributed wind load on joist: $W = \left(\begin{matrix} 86.5 \text{ psf} \\ \text{to} \\ 95.7 \text{ psf} \end{matrix} \right) (1.33 \text{ ft}) = \begin{matrix} 115.0 \text{ plf} \\ \text{to} \\ 127.3 \text{ plf} \end{matrix}$

Moment in joist: $\frac{Wl^2}{8} = \frac{W(16.67 \text{ ft})^2}{8} = \begin{matrix} 3,995 \text{ lb-ft} & 47,936 \text{ lb-in.} \\ \text{to} & \text{to} \\ 4,422 \text{ lb-ft} & 53,063 \text{ lb-in.} \end{matrix}$

ASD (Allowable Design)

$$S_{req} = \frac{(0.6)(47,936)}{1,656 \text{ psi}} = 17.4 \text{ in}^3$$
$$= \frac{(0.6)(53,063)}{1,656 \text{ psi}} = 19.2 \text{ in}^3$$

$2 \times 8 S_x = 13.14 < 17.4 \text{ in}^3 \therefore \text{NG}$

$2 \times 10 S_x = 21.39 > 19.2 \text{ in}^3 \therefore \text{OK}; \text{ use } 2 \times 10$

LRFD (Strength Design)

$$S_{req} = M/F_b'$$

$$S_{req} = \frac{47,936 \text{ lb-in.}}{3,575 \text{ psi}} = 13.4 \text{ in}^2$$

to

$$\frac{53,063 \text{ lb-in.}}{3,575 \text{ psi}} = 14.8 \text{ in}^2$$

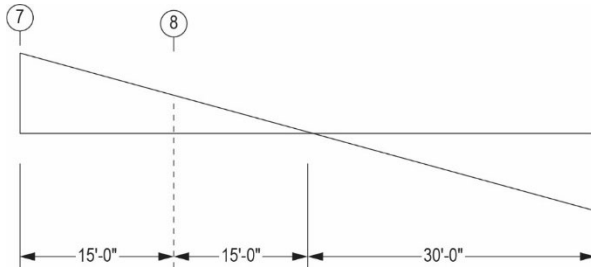
$2 \times 8 S_x = 13.14 < 13.4 \text{ in}^2 \therefore \text{NG}$

$2 \times 10 S_x = 21.39 > 14.8 \text{ in}^2 \therefore \text{OK}; \text{ use } 2 \times 10$

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Lateral Forces Depth—Buildings, Scenario 2, Solution 8, p. 259

$W = 2,345$ plf, strength/LRFD level



LRFD (Strength Design)

Shear at Grid Line 7: $V_7 = 2,345$ plf (30 ft) = 70,350 lb
 $v_7 = 70,350$ lb/50 ft = 1,407 plf

Shear at Grid Line 8: $v_8 = 1/2 v_7 = 703.5$ plf

2015 NDS, Table 4.2A: $\phi_D = 0.8$

Structural I sheathing	8d nails
10d nails	2× members
2× members	Blocked diaphragm
Blocked diaphragm	$2/3$ nailing capacity = $0.8 \times 1,680 = 1,344$ plf
	∴ 8d does not work for both; use 10d

Grid Line 7: $2\frac{1}{2}/4$ nailing = 1,432 plf > 1,407 plf ∴ OK; use $2\frac{1}{2}/4$ nailing

Grid Line 8: 6/6 nailing = 716 plf > 703.5 plf ∴ OK; use 6/6 nailing

ASD (Allowable Design)

$v_7 = 0.6(1,407 \text{ plf}) = 844$ plf

8d nails

$v_8 = 0.6(703.5 \text{ plf}) = 422$ plf

$2/3$ nailing capacity = $\frac{1,680}{2} = 840$ plf

∴ 8d does not work for both; use 10d

2015 NDS, Table 4.2A ASD; capacity $\Omega = 2.0$

10d nails

6/6 nailing capacity = $895/2 = 447.5$ plf

4/6 nailing capacity = $1,190/2 = 595$ plf

$2\frac{1}{2}/4$ nailing capacity = $1,790/2 = 895$ plf

$2/3$ nailing capacity = $2,045/2 = 1,023$ plf

Grid Lines A and 7: $2\frac{1}{2}/4$ nailing = 895 plf > 844 plf ∴ OK; use $2\frac{1}{2}/4$ nailing

Grid Line 8: 6/6 nailing = 447.5 plf > 422 plf ∴ OK; use 6/6 nailing

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Errata posted 09/01/2026

Lateral Forces Depth—Buildings, Scenario 2, Solution 11, p. 262

First, review the post-to-concrete connection options.

$$Z = 149 \text{ lb} \times 6 = 894 \text{ lb} < 2,000 \text{ lb}$$

NDS Table 12P

This is **not** a valid solution.

1/4" plate post base:

By inspection of the $Z_{||}$ values, (2) 3/4"-dia. through bolts will work with any side member thickness for 1,000-lb uplift.

NDS Table 12G

This is a valid solution.

Second, review the beam-to-post connection options.

The horizontal strap and 18-gauge cap would not transfer uplift forces to the column and are not compatible with the 1/4" weld, which would be required to interface with the other options. Therefore, these may only be used in conjunction with the 18-gauge angle with nails.

Check the lateral capacity of the nails into the column using NDS Table 12P:

$$Z = 137 \times 6 = 822 \text{ lb} < 1,000 \text{ lb}$$

This is **not** a valid solution. Therefore, the 18-gauge cap and horizontal strap may also be eliminated.

The only valid solution involves the assembly of steel plates with through bolts and welds to complete the uplift connection.

