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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

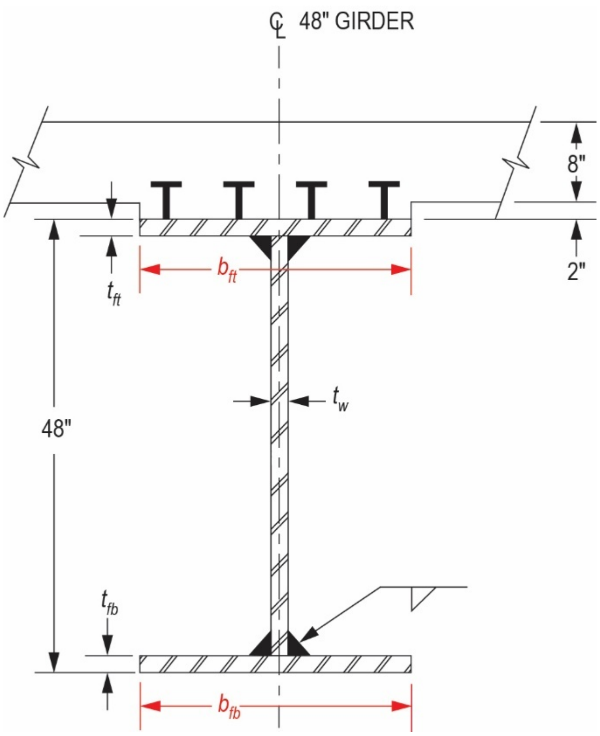
Design Data:

$t_{ft} = t_{fb}$ = 1 1/2 in.

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

n = 8.5

t_w = 5/8 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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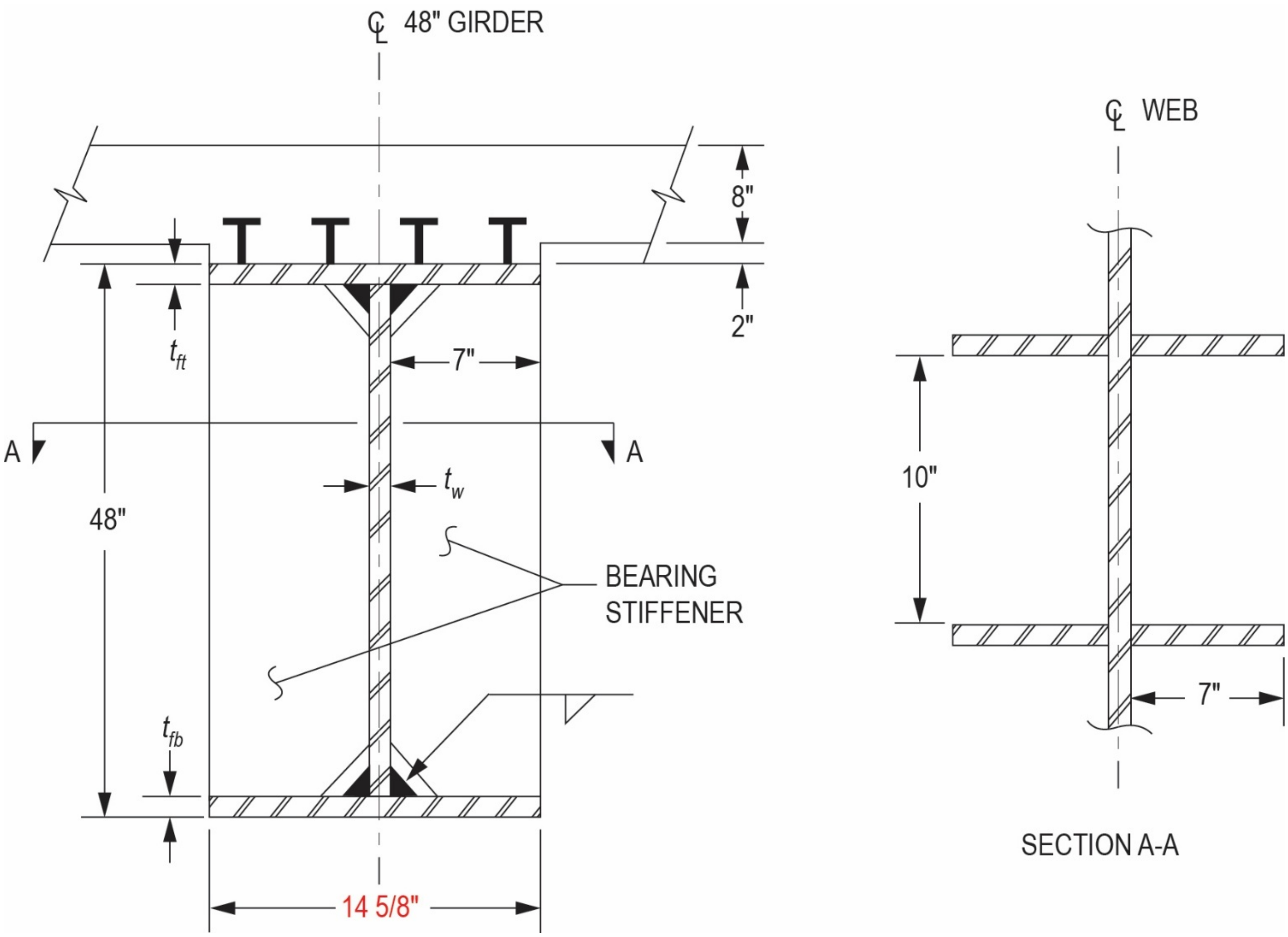
Vertical Forces Depth—Bridges, Scenario 1, Question 7, p. 82



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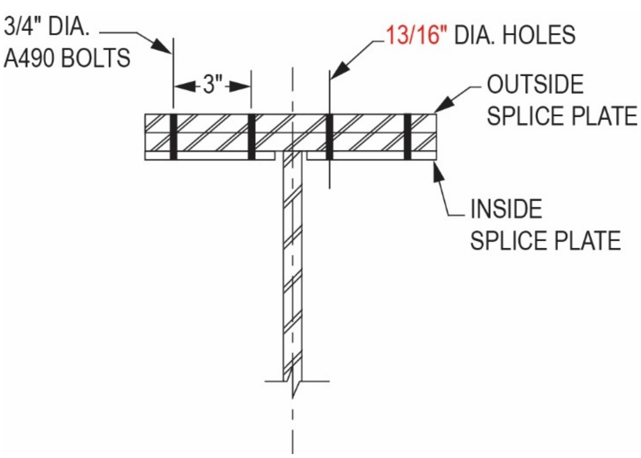
Vertical Forces Depth—Bridges, Scenario 1, Question 8, p. 83

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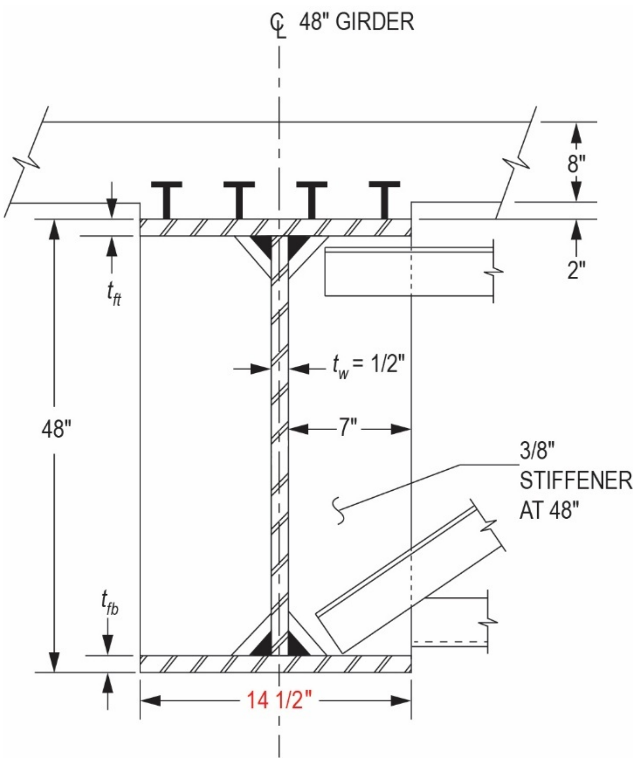
Vertical Forces Depth—Bridges, Scenario 1, Question 12, p. 86



Vertical Forces Breadth—Solution 11, p. 93

$$P_r = \phi P_n \geq P_u \qquad \text{AASHTO 5.6.5-1}$$

$$P_n = 0.85\, f_c' A_1 m \qquad \text{AASHTO 5.6.5-2}$$

Vertical Forces Breadth—Solution 26, p. 105

$$k = \sqrt{\left(n\rho^2\right) + 2n\rho} - n\rho \;= 0.207$$

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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.

$$\ell_n = 30'\text{-}0'' - 18'' = 28'\text{-}6'' = 342''$$

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

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.

$$\ell_n = 30'\text{-}0'' = 360''$$

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

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

$A_T = 3 \times 900\ \text{ft}^2 = 2,700\ \text{ft}^2$ 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 psf × 30 ft × 30 ft = 18 kips
L = 40 psf × 3 × 30 ft × 30 ft = 108 kips

ASD (Allowable Stress Design) ASCE 7 Sec 2.4.1

Load combo

1) D = 474 kips
2) $D + L$ = 474 kips + 108 kips = 582 kips (governs)
3) $D + L_r$ = 474 kips + 18 kips = 492 kips
4) $D + 0.75L + 0.75L_r$ = 474 kips + 0.75(108 kips) + 0.75(18 kips) = 568.5 kips

Acceptable range: ≥ 575 kips and ≤ 590 kips

LRFD (Strength Design) ASCE 7 Sec 2.3.1

Load combo

1) $1.4D$ = 1.4(474 kips) = 663.6 kips
2) $1.2D + 1.6L + 0.5L_r$
   = 1.2(474 kips) = 1.6(108 kips) = 0.5(18 kips) = 750.6 kips (governs)
3) $1.2D + 1.6\ L_r + L$
   = 1.2(474 kips) + 1.6(18 kips) + 108 kips = 705.6 kips

Acceptable range: ≥ 750 kips and ≤ 770 kips

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

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Vertical Forces Depth—Buildings, Scenario 1, Solution 9, p. 116

(a) Vertical load at footing center = 400 kips + 130 kips = 530 kips

(b) Soil weight on foundation toe (width = 30'-0")

2,000 psf (30'-0") (1'-11") = 115 kips

Soil weight applied at $e = \frac{2'\text{-}2''}{2} + \frac{1'\text{-}11''}{2}$

$= \frac{26''}{2} + \frac{23''}{2} = 24.5'' = 2.042'$

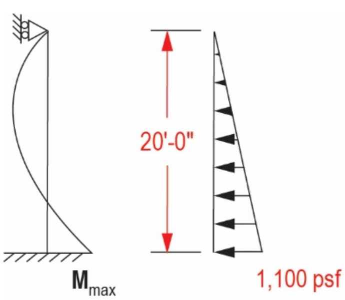
Total vertical load = 530 kips + 115 kips = 645 kips

Weight of footing = 150 pcf × 3'-0" × 6'-0" × 30'-0" = 81.0 kips

Total vertical load with footing weight:

P = 645 kips + 81.0 kips = 726 kips

Moment due to lateral load at footing base:



$$\mathbf{M} \text{ at base} = \frac{1{,}100 \text{ psf } (20 \text{ ft})^2 (30)}{15\ (1{,}000)} = 800 \text{ ft-kips}$$

Total moment at footing centerline:

$= 800 \text{ ft-kips} - 115 \text{ kips} \times 2.042'$

$= 645.17 \text{ ft-kips}$

$$e = \frac{\mathbf{M}}{\mathbf{P}} = \frac{645.17 \text{ ft-kips}}{726 \text{ kips}} = 0.89'$$

$$\frac{B}{6} = \frac{6'\text{-}0''}{6} = 1'\text{-}0'' \qquad e < \frac{B}{6}$$

$$q_{max} = \frac{P}{BL}\left(1 + \frac{6e}{B}\right) = \frac{726 \text{ kips}}{6'(30')}\left(1 + \frac{6(0.89')}{6'}\right)$$

$$q_{max} = 7.62 \text{ ksf}$$

THE CORRECT ANSWER IS: 7.5 to 7.8

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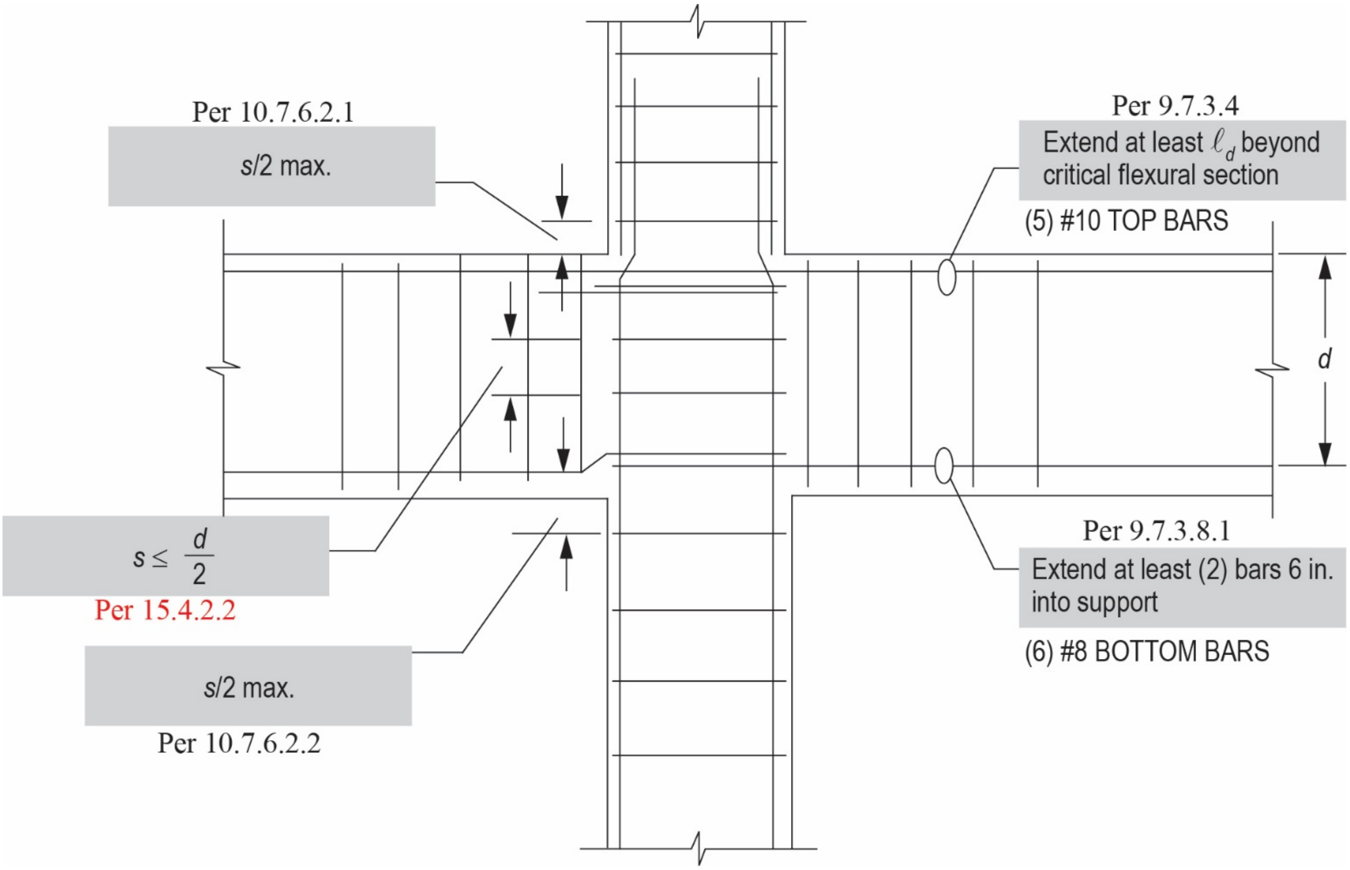
Vertical Forces Depth—Buildings, Scenario 1, Solution 10, p. 117



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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

$\ell_n = 30'\text{-}0'' - 24'' = 28'\text{-}0'' = 336''$

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.

$\ell_n = 30'\text{-}0'' = 360''$

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

THE CORRECT ANSWER IS: 16 to 17.2

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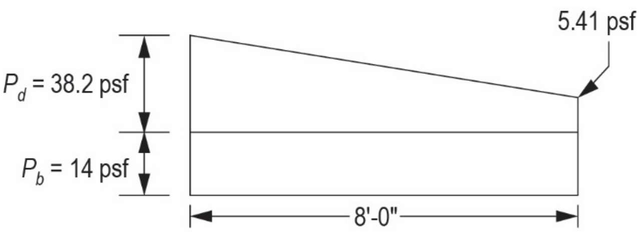
Vertical Forces Depth—Buildings, Scenario 2, Solution 4, p. 125

$p_d = (2.33\text{ ft})(16.6\text{ pcf}) = 38.2\text{ psf}$
$W = 4h_d = 4(2.33\text{ ft}) = 9.32\text{ ft}$
$P_b = 14\text{ psf}$



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

$D + L_r = 2.09 + 0.48 = 2.57\text{ kips}$
$D + S = 2.09 + 0.990 = 3.08\text{ kips} \leftarrow \text{Governs}$

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

$$D = 16 \text{ psf } (26 \text{ ft})\left(\frac{16}{12}\right) = 555 \text{ lb (wall)}$$

$$L_R = 20 \text{ psf}\left(\frac{8 \text{ ft}}{2}\right)\left(\frac{16}{12}\right) = 107 \text{ lb (roof)}$$

$$D_R = 15 \text{ psf}\left(\frac{8 \text{ ft}}{2}\right)\left(\frac{16}{12}\right) = 80 \text{ lb (roof)}$$

$$\Sigma = 742 \text{ lb}$$

$M = (P_R)(e) = (L_R + D_R)(e)$

$M = (P)(e) = (107 + 80)(4.25 \text{ in.}) = 795 \text{ in.-lb}$

$$\left(\frac{f_c}{F_c'}\right)^2 + \frac{f_b}{F_b'\left(1-\frac{f_c}{F_{cE}'}\right)} \le 1.0 \qquad (3.9.3)$$

$$f_b = \frac{M}{S} = \frac{795 \text{ in.-lb}}{7.56 \text{ in}^3} = 105 \text{ psi}$$

$$\left(\frac{90}{181}\right)^2 + \frac{105}{1{,}006\left(1-\frac{90}{182}\right)} = 0.45$$

$$f_c = \frac{742}{(1.5)(5.5)} = 90 \text{ psi}$$

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, $b_{eff} = 72$ in.

6 bars in b_{eff}

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

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

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

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

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

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

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

$\therefore$ 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 in^2	54 in.	1,220.4 in^3	22.6(54 – 30.72)^2 = 12,248.26 in^4	120.47 in^4
Fillet	(18)(2)/25.5 = 1.41 in^2	49 in.	69.09 in^3	1.41(49 – 30.72)^2 = 471.16 in^4	0.47 in^4
Top Flange	(18)(1.5) = 27 in^2	47.25 in.	1,275.75 in^3	27(47.25 – 30.72)^2 = 7,377.50 in^4	5.06 in^4
Web	(0.625)(45) = 28.125 in^2	24 in.	675 in^3	28.125(24.0 – 30.72)^2 = 1,270.08 in^4	4,746.09 in^4
Bottom Flange	(18)(1.5) = 27 in^2	0.75 in.	20.25 in^3	27(0.75 – 30.72)^2 = 24,251.42 in^4	5.06 in^4
	$\sum$ 106.14 in^2		3,260.49 in^3	45,618.42 in^4	4,877.15 in^4

$3n = 3 \times 8.5 = 25.5$

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

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

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

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

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

$$Z_r = 5.5d^2 \qquad 6.10.10.2\text{-}1$$

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

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

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

$$V_{fat} = V_{sr} \qquad 6.10.10.1.2\text{-}2$$

$$p \le \frac{nZ_r}{V_{sr}} = \frac{4\,(3.09)}{0.938} = 13.17 \text{ in.} \qquad 6.10.10.1.2\text{-}1$$

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

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

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

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

$A_e = \dfrac{0.8\,(65)}{0.95\,(50)}(19.125) = 20.94\text{ in}^2$ AASHTO 6.13.6.1.3 b-2

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

Hole for 3/4-in.-diameter bolt — Table 6.13.3.4-1

$F_{yf} = 50$ ksi — Table 6.4.1-1

$F_u = 65$ ksi — Table 6.4.1-1

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

$I_{t1} = bt_w{}^3\ J$ AASHTO 6.10.11.1.3

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

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

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

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

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

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

$\rho_t = 1.94$

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

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

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

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

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

THE CORRECT ANSWER IS: 94.0 to 96.0

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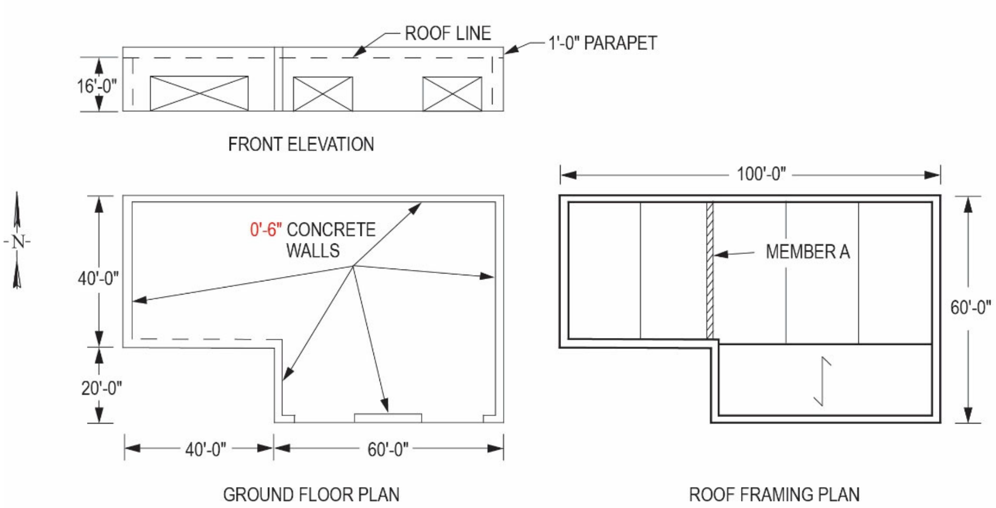
Lateral Forces Breadth—Question 7, p. 160



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

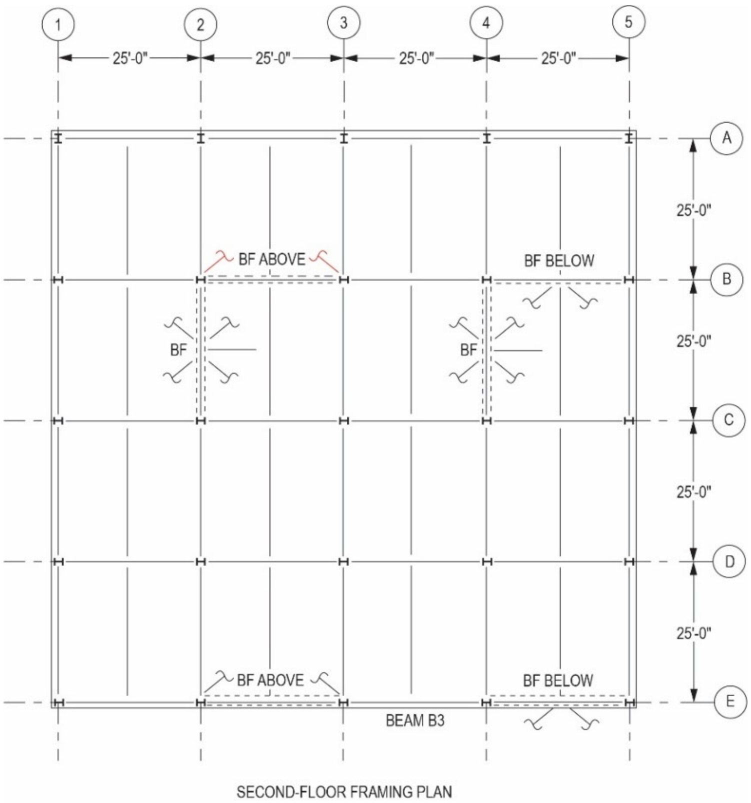

FIGURE C

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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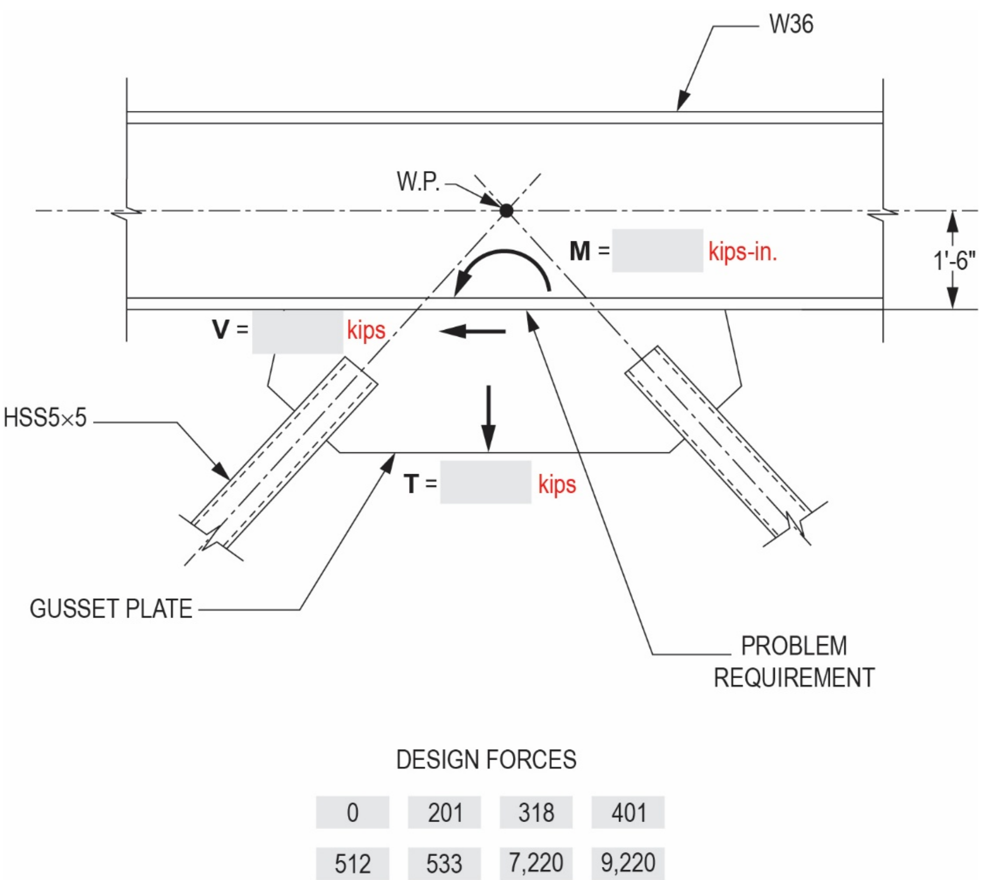
Lateral Forces Depth—Buildings, Scenario 1, Question 6, p. 200
For this question, consider bolt failure states only. Based on the figure, the number of bolts required to transfer the axial force to the special concentric brace frame is:

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

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

Material Specifications:

Steel reinforcement ASTM A615, f_y = 60 ksi

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

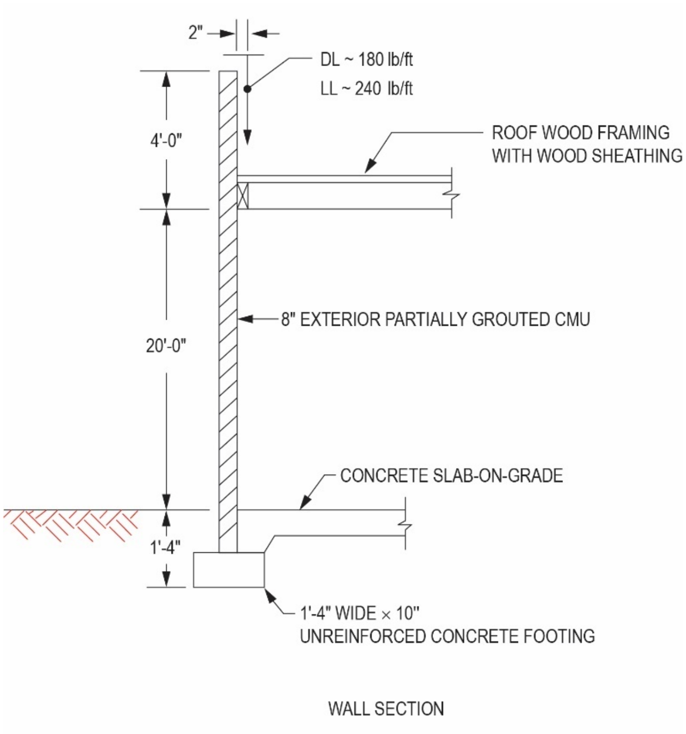

FIGURE A

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

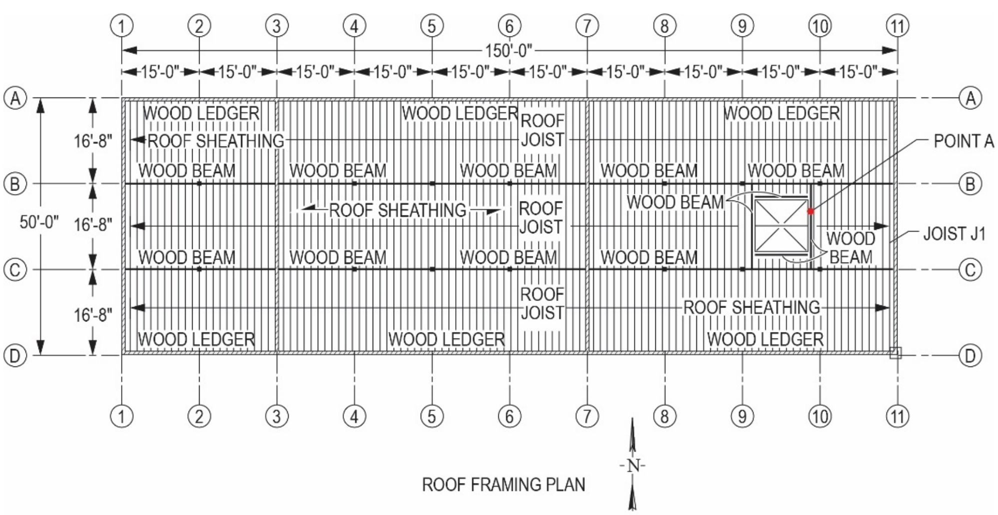

FIGURE B

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

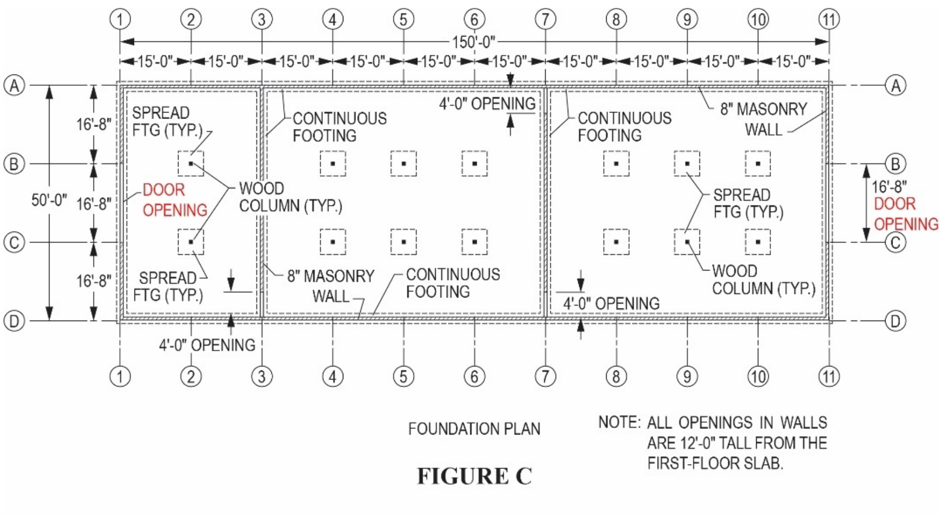

FIGURE C

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 __________.

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 __________.

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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 ft^2.
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)
Mu = 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:

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 ___________.

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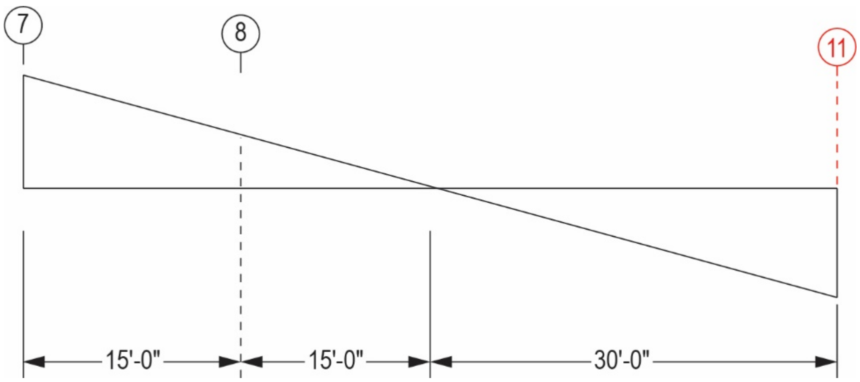
Lateral Forces Depth—Buildings, Scenario 2, Question 8, p. 212

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

Lateral Forces Breadth—Solution 22, p. 240

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

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

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

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

$$\therefore l_d = 91.4 \text{ in.}\left(\frac{1.0 \times 1.0 \times 0.42 \times 1.0}{1.0}\right)$$

$$= 38.39 \text{ in.} \times 1.25 = 47.985 \text{ in.} \sim 48 \text{ in.}$$ AASHTO 5.11.4.3

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

Round HSS 7.500 × 0.375 is correct.

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

ϕP_n = 199 kips (LRFD); P_{all} = 132 kips (ASD)

D/t = < 25.7

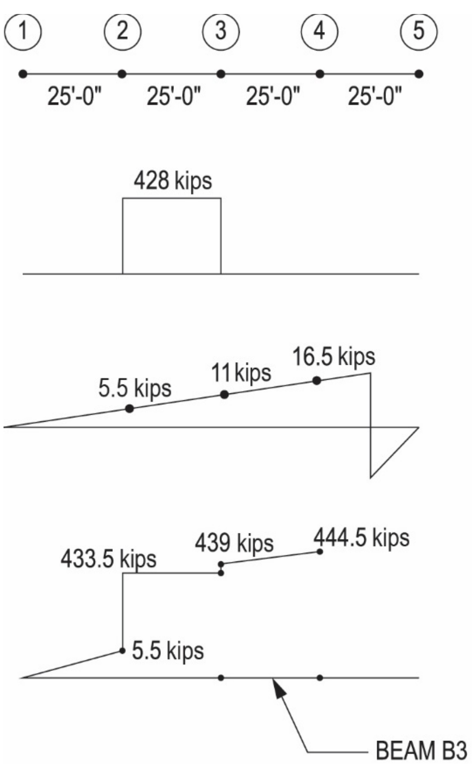
Selecting a round HSS 7.500 × 0.250 or HSS 7.500 × 0.312 is not acceptable.

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

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



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

LRFD min = 439 kips

max = 444.5 kips

ASD min = 0.7 (439) = 307.3 kips

max = 0.7 (444.5) = 311.2 kips

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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 in^2

Using AISC Table 7-1, Group B, V_{ASD} = 41.7 kips, = V_{LRFD} = 62.6 kips
AISC Table 7-17: F_U = 150 ksi, F_V =84 ksi
AISC Table J3.2: $R_n = F_v A_{bolt}$, R_n/Ω = 41.7 kips, ϕR_n = 62.6 kips

ASD (Allowable Design)
ASD = 557 kips (given)/41.7 kips/bolt = 13.4 ≈ 14 bolts

LRFD (Strength Design)
LRFD = 796 kips (given)/62.6 kips/bolt = 12.7 ≈ 13 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

$q_p = 0.00256(0.932)(1.0)(0.85)(1.0)(142)^2 = 40.9$ 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 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

Footnote 5, Figure 30.3-1

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

Figure 30.3-1

Figure 30.3-2A

p = 160 psf

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 ft × 4.0 ft/3 = 5.33 ft^2
GC_p = Figure 30.3-1

Reference Footnote 5, Figure 30.3-1

= $-1.0 \times 0.9 = +0.9 = GC_p$ for p_{roof}
$-1.1 \times 0.9 = -0.99 = GC_p$ for p_{wall}
p_{roof} = 50 psf (0.9 – ∅) = 45 psf
p_{wall} = 50 psf (0.99 – ∅) = 49.5 psf
p_{total} = 94.5 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%

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

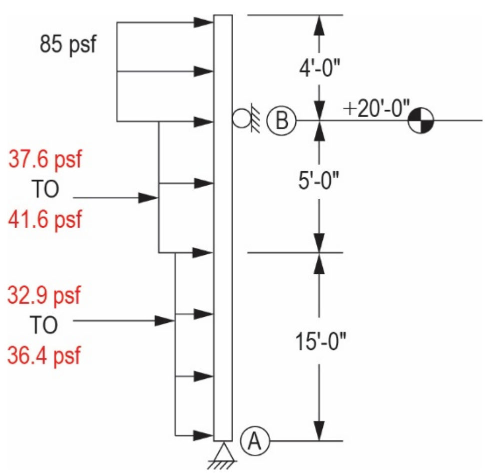
$0.95 \times 0.9 = 0.86 - 0.18 = 1.04$

Design wind pressure P at wall below roof:

$$0\text{–}15 \text{ ft} = 35 \text{ psf} \begin{pmatrix} 0.94 \\ 1.04 \end{pmatrix} = \begin{matrix} 32.9 \text{ psf} \\ \text{to} \\ 36.4 \text{ psf} \end{matrix}$$

$$15\text{–}20 \text{ ft} = 40 \text{ psf} \begin{pmatrix} 0.94 \\ 1.04 \end{pmatrix} = \begin{matrix} 37.6 \text{ psf} \\ \text{to} \\ 41.6 \text{ psf} \end{matrix}$$

Parapet wind pressure given as 85 psf



$\Sigma M_A = 0$:

$$R_B(20 \text{ ft}) = (85 \text{ psf})(4.0 \text{ ft})(22 \text{ ft}) + \begin{pmatrix} 37.6 \\ 41.6 \end{pmatrix}(5.0 \text{ ft})(17.5 \text{ ft}) + \begin{pmatrix} 32.9 \\ 36.4 \end{pmatrix}(15 \text{ ft})(7.5 \text{ ft})$$

$$20R_B = 7{,}480 + \begin{pmatrix} 3{,}290 \\ 3{,}640 \end{pmatrix} + \begin{pmatrix} 3{,}701 \\ 4{,}095 \end{pmatrix}$$

$$20R_B = \begin{pmatrix} 14{,}471 \\ 15{,}215 \end{pmatrix}$$

$R_B = 723$ to 761 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*:

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

$n\rho\ = 0.019$

$n\rho j\ = [16.11(1{,}080)(12)]/[12(3.81)^2(32{,}000)] = 0.037$

$n\rho\ = 0.041$

$0.041 > 0.019$

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

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

32 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*:

$R = M_u/\varnothing bd^2$

$R = 1{,}800/0.9(12)(3.81)^2 = 183 = (1{,}800 \times 12)/[0.9(12)(3.81)^2] = 137.8$

At $R = 137.8$, $\rho = 0.0024$

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

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

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

40 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 ft^2

Therefore, use effective wind area = 93 ft^2

ASCE 7 Figure 30.3-2A, Zone 2: GC_p can be taken with range of –1.7 to –1.9

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

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

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

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 × 8 S_x = 13.14 < 17.4 in^3 ∴ NG

2 × 10 S_x = 21.39 > 19.2 in^3 ∴ OK; use 2 × 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 × 8 S_x = 13.14 < 13.4 in^2 ∴ NG

2 × 10 S_x = 21.39 > 14.8 in^2 ∴ OK; use 2 × 10

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

LRFD (Strength Design)

Shear at Grid Line 7: V_7 = 2,300 plf (30 ft) = 69,000 lb
v_7 = 69,000 lb/50 ft = 1,380 lb/ft

ASD (Allowable Design)

v_7 = 0.6(1,380 lb-ft) = 828 lb-ft
v_8 = 0.6(690 lb-ft) = 414 lb-ft