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Revisions are shown in red.

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

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)		22.55
F_u (ksi)		24
P_{fy} (ksi)		50
ϕ_u		70
ϕ_y		1,127.5

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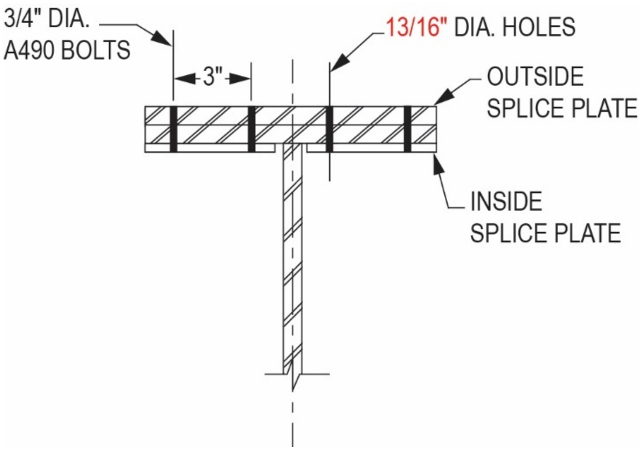
Vertical Forces Depth—Bridges, Scenario 1, Question 11, p. 85



Vertical Forces Breadth—Solution 26, p. 105

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

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

Alternate solution:

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

Slab support condition is both end continuous.

ℓ_n = 30'-0" = 360"

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

THE CORRECT ANSWER IS: 12.5 to 13.5

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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, C4.7.2:

"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 + 20 kips = 494 kips
4) $D + 0.75L + 0.75L_r$ = 474 kips + 0.75(108 kips) + 0.75(20 kips) = 570 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(20 kips) = 751.6 kips (governs)
3) $1.2D + 1.6\,L_r + L$
   = 1.2(474 kips) + 1.6(20 kips) + 108 kips = 708.8 kips

Acceptable range: > 750 kips and < 770 kips

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

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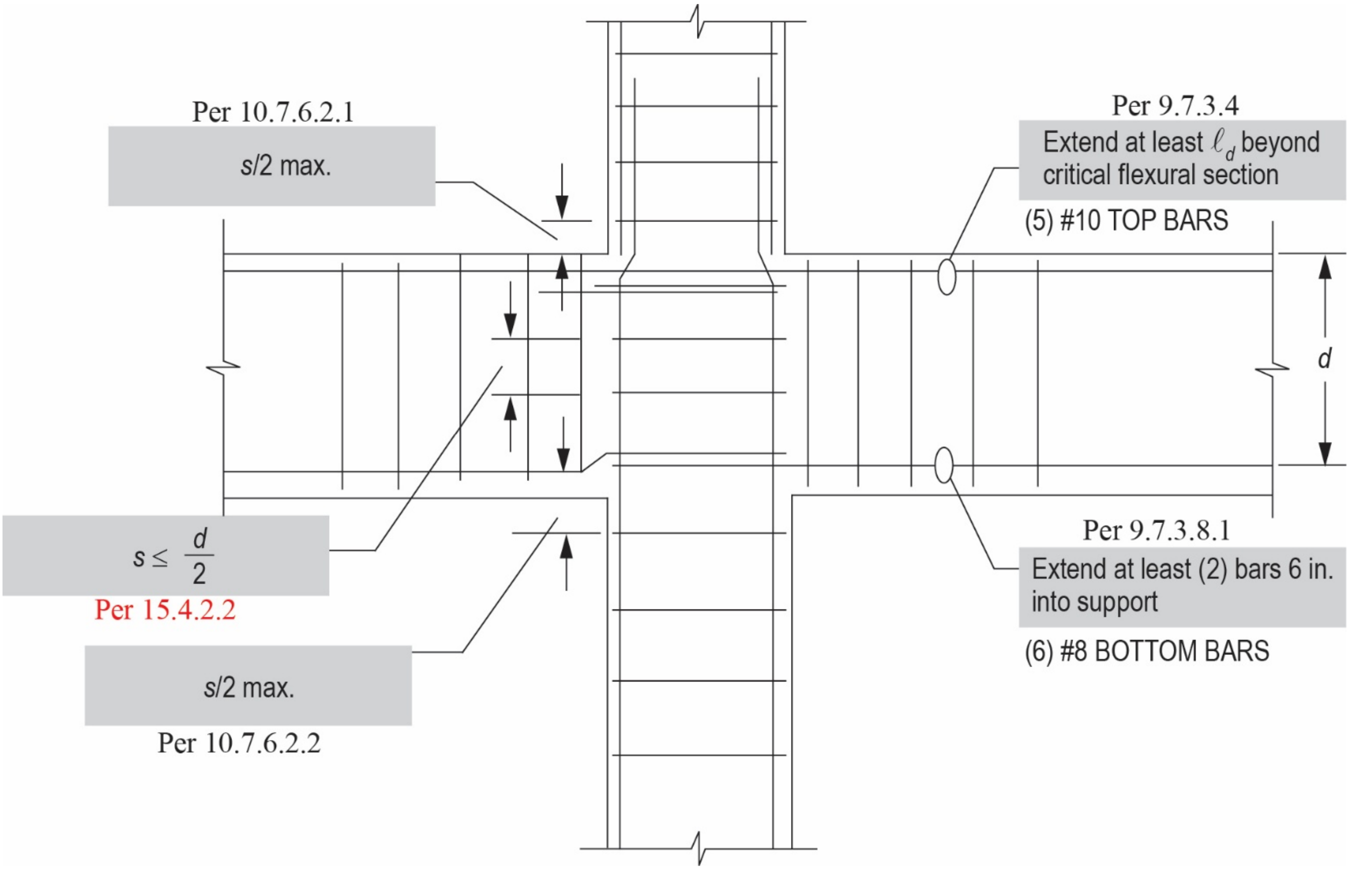
Vertical Forces Depth—Buildings, Scenario 1, Solution 10, p. 117



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

Alternate solution:

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

Beam support condition is both end continuous.

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

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

THE CORRECT ANSWER IS: 17 to 18

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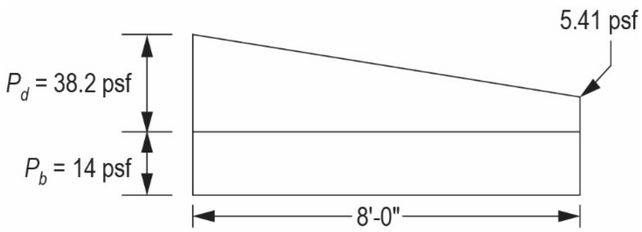
Vertical Forces Depth—Buildings, Scenario 2, Solution 4, p. 125

p_d = (2.33 ft)(16.6 pcf) = 38.2 psf
$W = 4h_d$ = 4(2.33 ft) = 9.32 ft
P_b = 14 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 kips ← Governs snow

$D + L_r$ = 2.09 + 0.48 = 2.57 kips
$D + S$ = 2.09 + 0.990 = 3.08 kips ← Governs

LRFD (Strength Design)

$1.2D + 1.6L_r$ = 3.28 kips
$1.2D + 1.6S$ = 4.09 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

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

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

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

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

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

LRFD (Strength Design)

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

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

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 4, p. 139

Reference: AASHTO 6.10.1.1b

Component	**Area**	Y	Ay	$A(y-\bar{y})^2$	$I_o = \left(\dfrac{bd^3}{12}\right)$ $b = b_E/3n$
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 = 5/16 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 8, p. 143

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

$$A_e = \frac{0.8\ (70)}{0.95\ (50)}(19.125) = 22.55 \text{ in}^2 \qquad \text{AASHTO 6.13.6.1.3 b-2}$$

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

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

$F_{yf} = 50$ ksi — Table 6.4.1-1

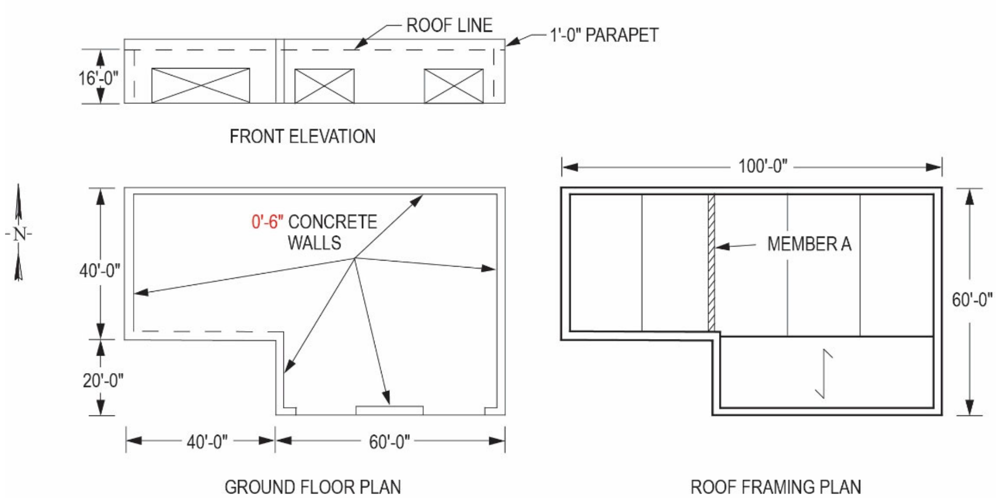
$F_u = 70$ ksi — Table 6.4.1-1

$P_{fy} = A_e F_{yf} = 1{,}127.5$ kips

$\phi_u = 0.8$ — 6.5.4.2

$\phi_y = 0.95$ — 6.5.4.2

Lateral Forces Breadth—Question 7, p. 160



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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 Depth—Buildings, Scenario 1, Figure C

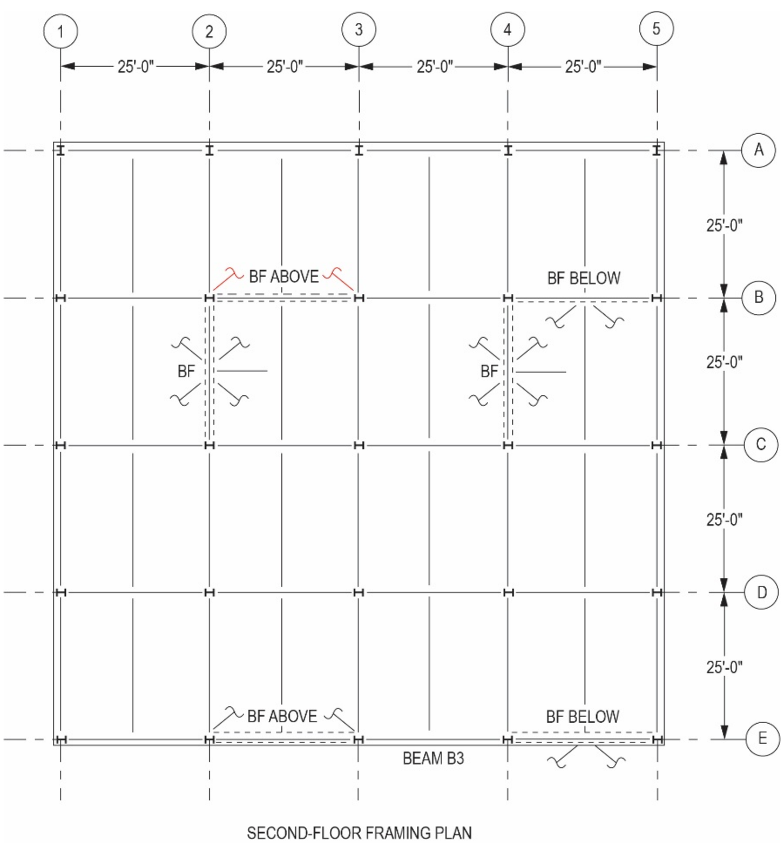

FIGURE C

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___________.

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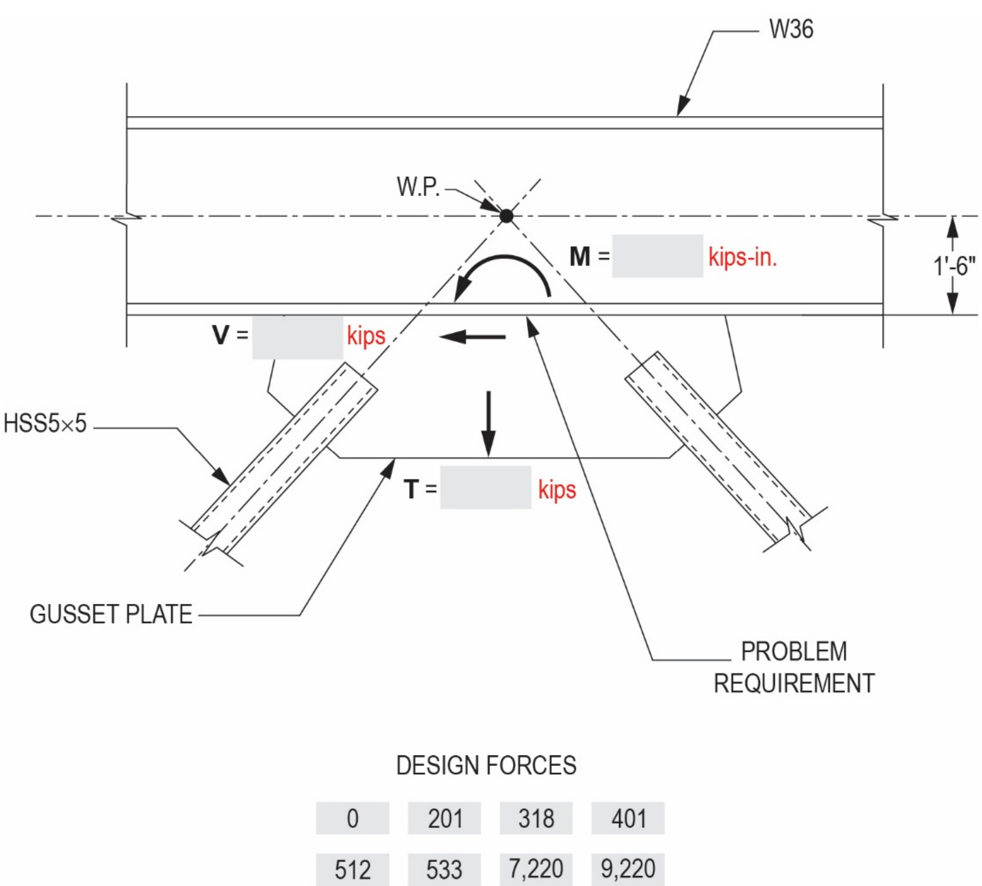
Lateral Forces Depth—Buildings, Scenario 1, Question 3, p. 198
Based on the story forces provided in **Table 1** and effective brace length *KL* = 18 ft, the lightest round hollow steel section using a round HSS 7.500 brace member between the sixth floor and roof is:

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:

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



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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

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

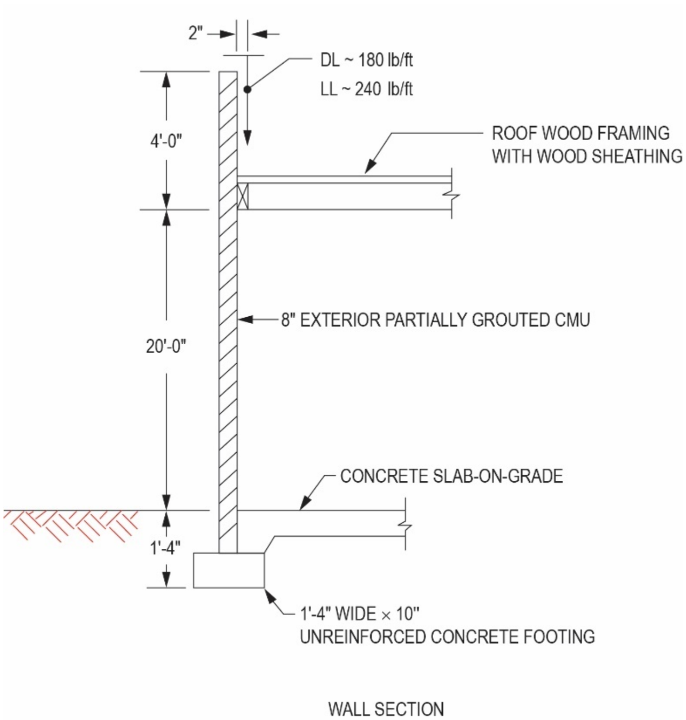

FIGURE A

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

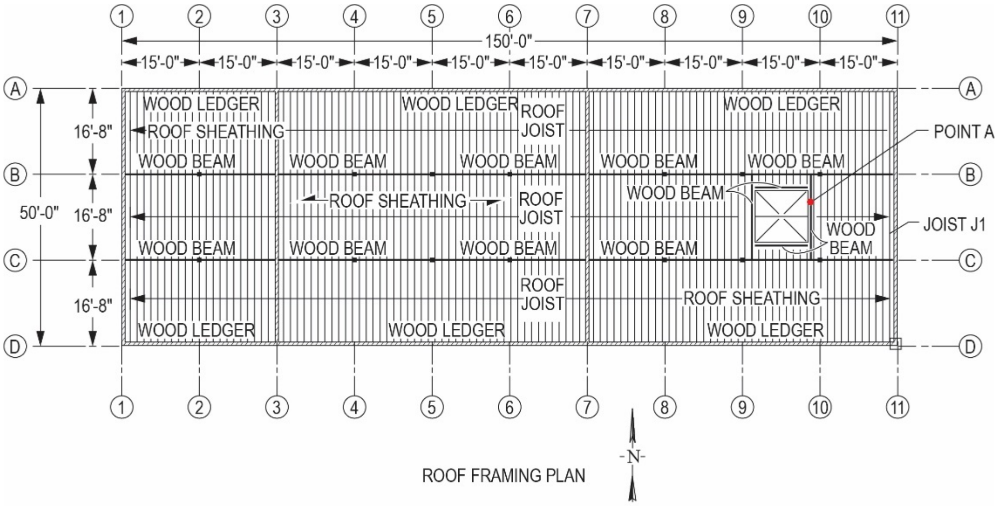

FIGURE B

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

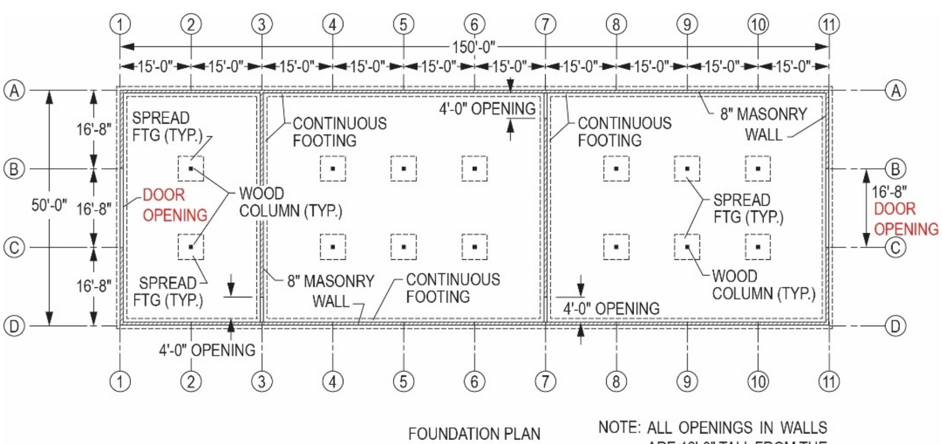

FIGURE C

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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 __________.

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 loads R_B (plf of roof) from the windward wall at Grid Line 5 that are resisted by the roof diaphragm are __________.

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 __________.

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

Assume an ASD moment of 1,080 ft-lb, or a strength level moment of 1,800 ft-lb. 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

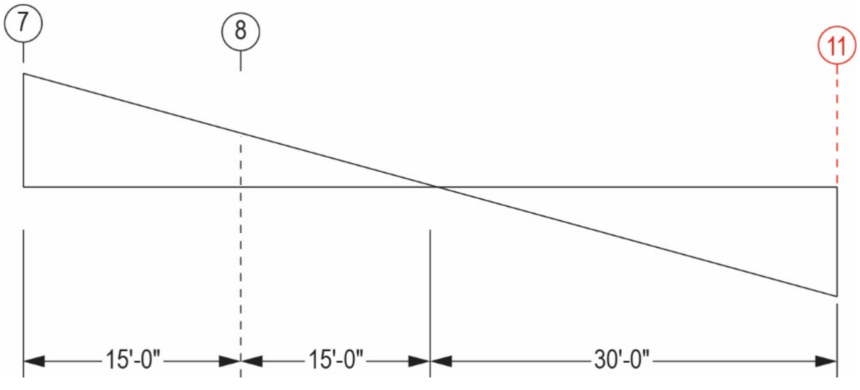
Assumptions:

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 ___________.

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 ________.

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Lateral Forces Breadth—Solution 19, p. 238

R_y = 1.3 HSS ASTM A500 Grade C AISC SDM Table A3.2

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

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

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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 × 0.9 = +0.9 = GC_p for p_{roof}
–1.1 × 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

Lateral Forces Depth—Buildings, Scenario 2, Solution 6, p. 256

From *NCEES PE Structural Reference Handbook*:
$2/jk = (12)(3.81)^2(900)/1{,}080 = 12.1$
$np = 0.019$
$npj = 16.11(1{,}080)(12)/12(3.81)^2(32{,}000) = 0.037$
$np = 0.041$

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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