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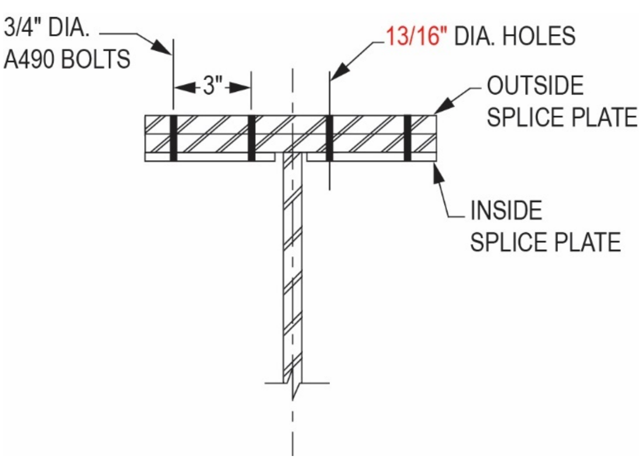
Revisions are shown in red.

Vertical Forces Depth—Bridges, Scenario 1, Question 8

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

Vertical Forces Depth—Bridges, Scenario 1, Question 11



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Vertical Forces Depth—Buildings, Scenario 1, Solution 3

$L_r = 20$ 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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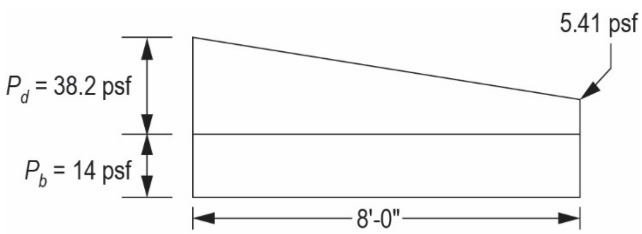
Vertical Forces Depth—Buildings, Scenario 2, Solution 4

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



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

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

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

Vertical Forces Depth—Bridges, Scenario 1, Solution 4

$3n = 3 \times 8.5 = 25.5$

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

$$I_{LT} = 45{,}618.42\text{ in}^4 + 7{,}840.21\text{ in}^4 = 53{,}438.63\text{ in}^4$$

$$S_{LT}^b = \frac{53{,}438.63\text{ in}^4}{30.72\text{ in.}} = 1{,}740.19\text{ in}^3$$

$$f_{b\,LT} = 1{,}200/1{,}740.19\left(\frac{12\text{ in.}}{\text{ft}}\right) = 8.27\text{ ksi}$$

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Vertical Forces Depth—Bridges, Scenario 1, Solution 5

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

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

$A_e = \dfrac{0.8\ (70)}{0.95\ (50)}(19.125) = 22.55 \text{ in}^2$ 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

$F_u = 70$ 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

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Lateral Forces Depth—Buildings, Scenario 1, Figure C

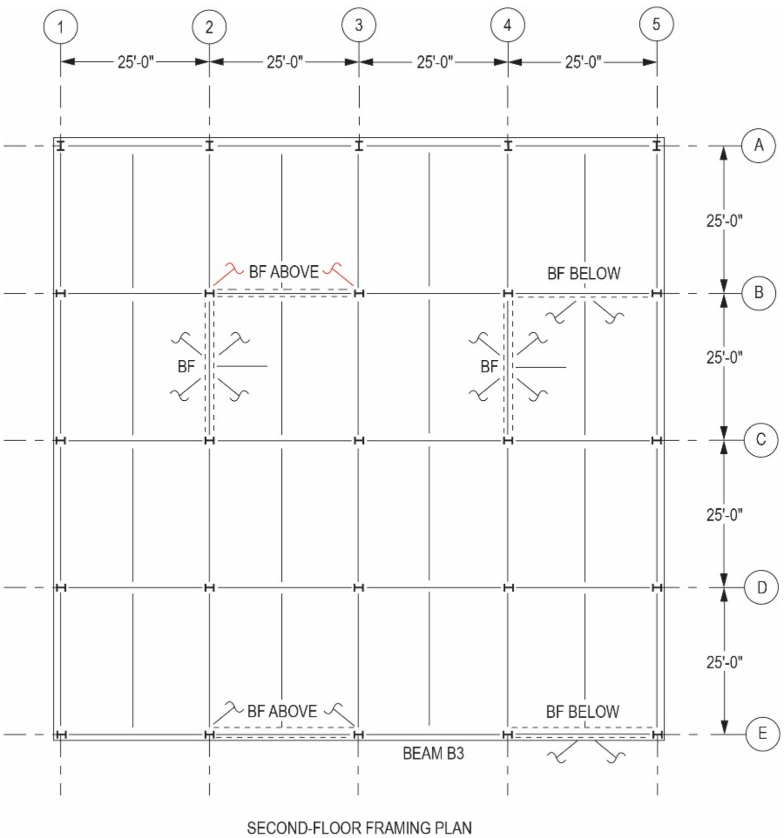

FIGURE C

Lateral Forces Depth—Buildings, Scenario 1, Question 2
Based on the following information, the lightest W36 beam for Beam B1 in **Figure E** is___________.

Lateral Forces Depth—Buildings, Scenario 1, Question 3
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
Which of the elements identified in the figure are required to be designed for overstrength?

Vertical Forces Depth—Buildings, Scenario 1, Question 5
Based on story forces provided in **Table 1**, the governing axial force for Beam B3 in **Figure F** is:

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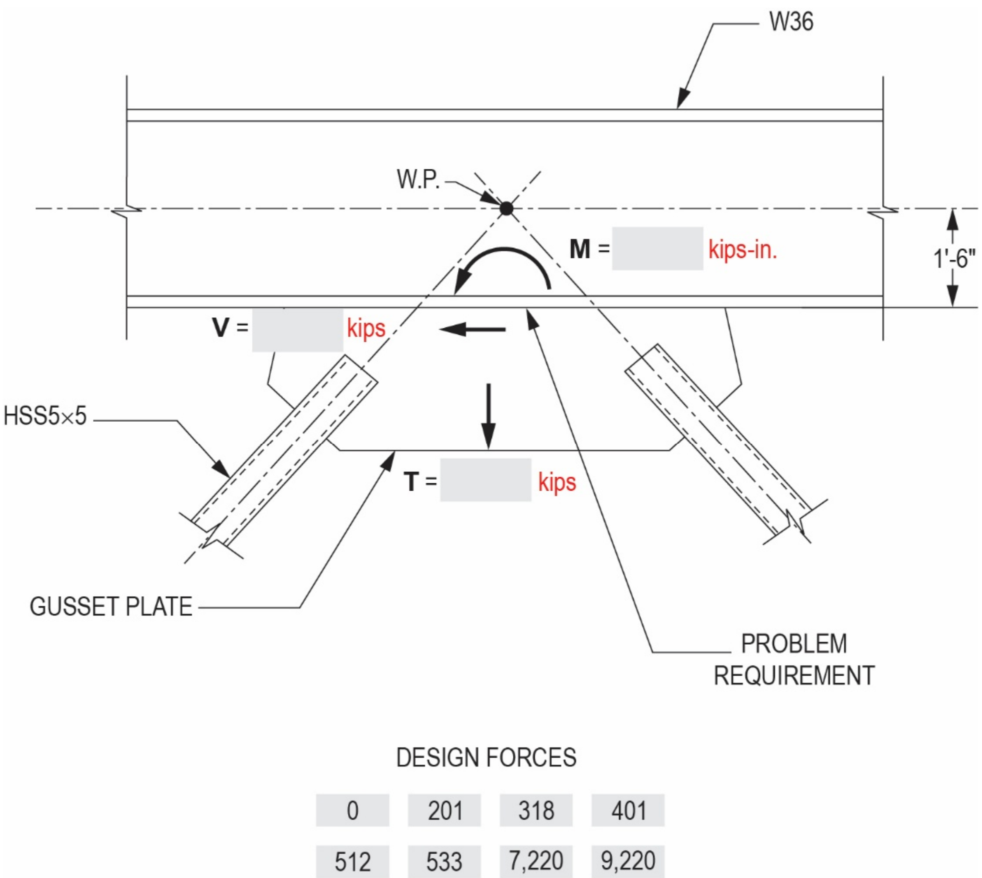
Lateral Forces Depth—Buildings, Scenario 1, Question 6
For this question, consider bolt failure states only. Based on the figure, the number of bolts required to transfer the axial force to the special concentric brace frame is:

Lateral Forces Depth—Buildings, Scenario 1, Question 7



Lateral Forces Depth—Buildings, Scenario 1, Question 10

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

Lateral Forces Depth—Buildings, Scenario 2

Material Specifications:

Steel reinforcement ASTM A615, f_y = 60 ksi

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Lateral Forces Depth—Buildings, Scenario 2, Figure A

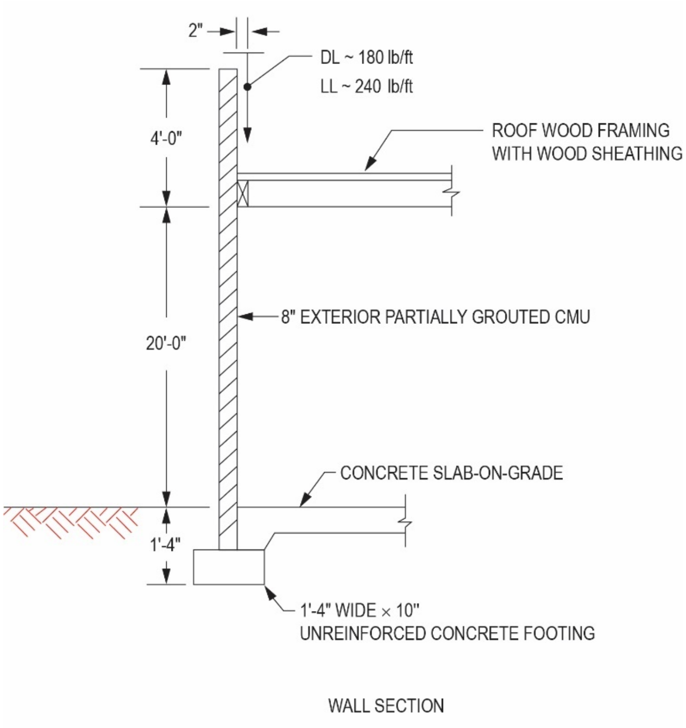

FIGURE A

Lateral Forces Depth—Buildings, Scenario 2, Figure B

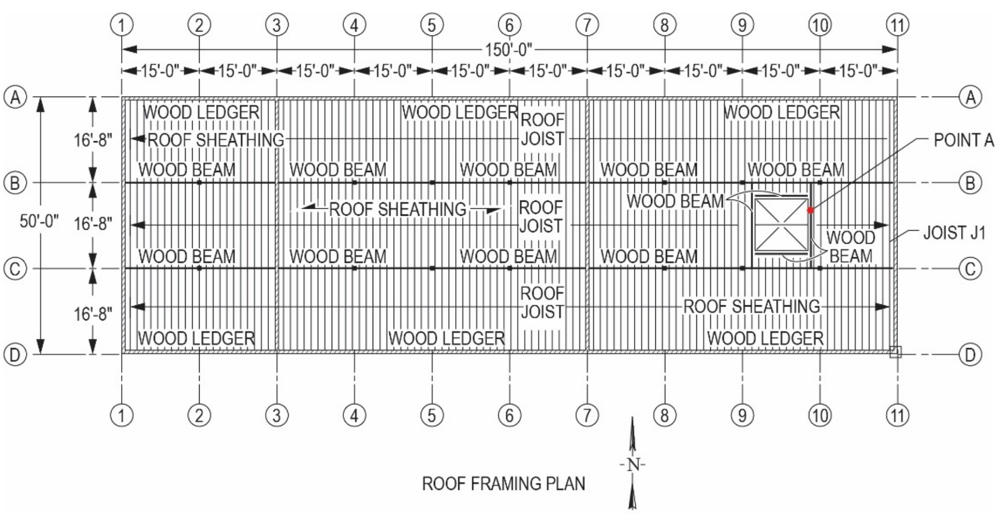

FIGURE B

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Lateral Forces Depth—Buildings, Scenario 2, Figure C

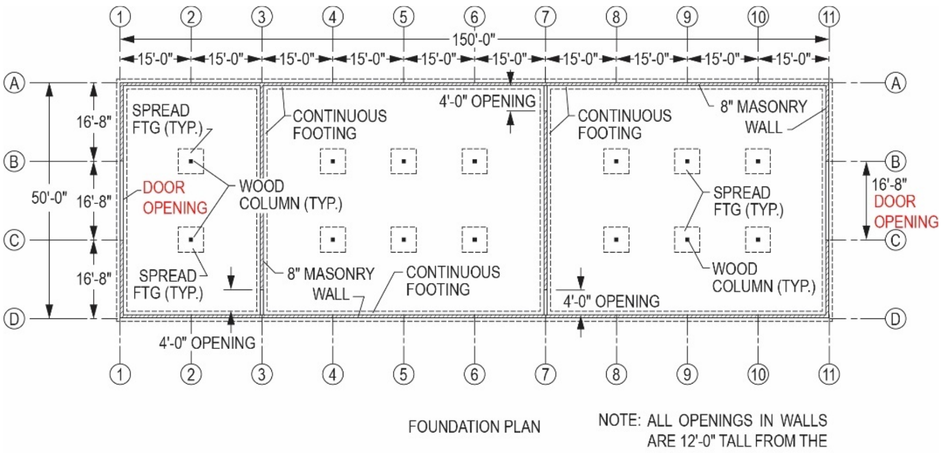

FIGURE C

Lateral Forces Depth—Buildings, Scenario 2, Question 1

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

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

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

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

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

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

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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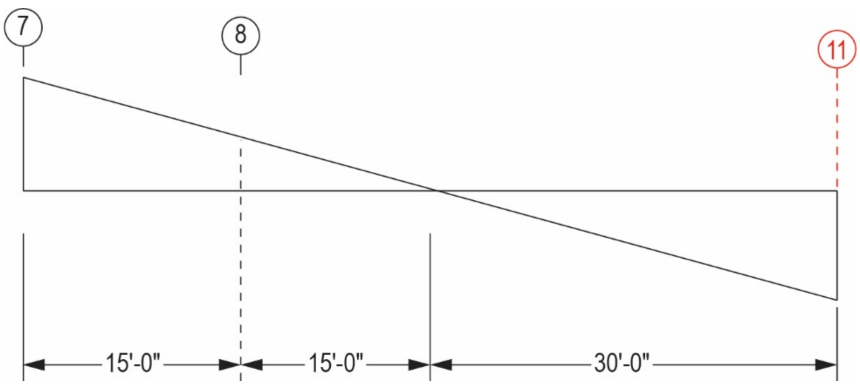
Lateral Forces Depth—Buildings, Scenario 2, Question 8

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

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—Buildings, Scenario 2, Solution 1

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

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

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

Lateral Forces Depth—Buildings, Scenario 2, Solution 6

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

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)

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

$= \dfrac{(0.6)(53{,}063)}{1{,}656 \text{ psi}} = 19.2 \text{ in}^3$

2×8 $S_x = 13.14 < 17.4 \text{ in}^3 \therefore$ NG
2×10 $S_x = 21.39 > 19.2 \text{ in}^3 \therefore$ OK; use 2 × 10

LRFD (Strength Design)

$S_{req} = M/F_b'$

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

to

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

2×8 $S_x = 13.14 < 13.4 \text{ in}^2 \therefore$ NG
2×10 $S_x = 21.39 > 14.8 \text{ in}^2 \therefore$ OK; use 2 × 10

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Lateral Forces Depth—Buildings, Scenario 2, Solution 8

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