Fundamentals of Structural Analysis 6th edition Leet Solutions Manual

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FUNDAMENTALS OF STRUCTURAL ANALYSIS

6th Edition

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SOLUTIONS MANUAL

CHAPTER 2: DESIGN LOADS AND STRUCTURAL FRAMING

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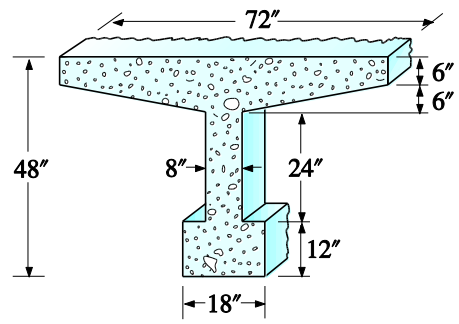
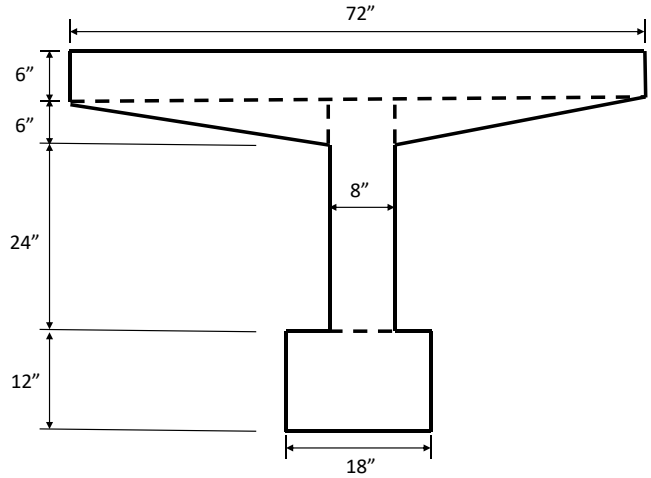
P2.1. Determine the deadweight of a 1-ft-long segment of the prestressed, reinforced concrete tee-beam whose cross section is shown in
Figure P2.1. Beam is constructed with lightweight concrete which weighs 120 lb/ft^3.



Section

P2.1

Compute the weight/ft. of cross section @ 120 lb/ft^3.



Compute cross sectional area:

$$\text{Area} = (0.5)(6) + 2\left(\frac{1}{2}(0.5)(2.67)\right)(0.67)(2.5) + 15(1)$$

$$= 7.5 \text{ ft}^2$$

Weight of member per foot length:

$$\text{wt/ft} = 7.5(120) = \boxed{900 \text{ lb/ft}}$$

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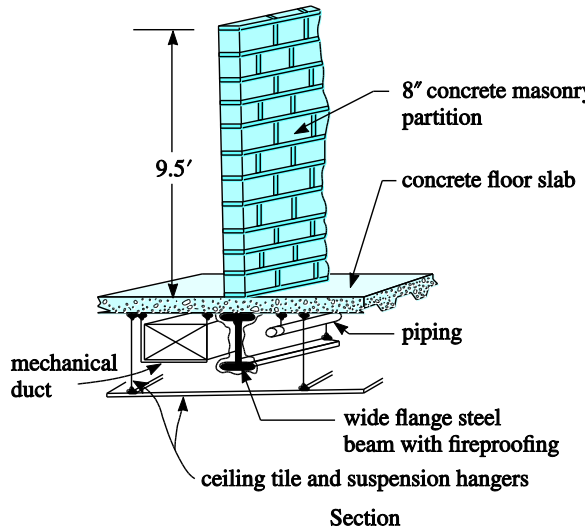
P2.2. A wide flange steel beam shown in Figure P2.2 supports a permanent concrete masonry wall, floor slab, architectural finishes, mechanical and electrical systems. Determine the uniform dead load in kips per linear foot acting on the beam.

The wall is 9.5-ft high, non-load bearing and laterally braced at the top to upper floor framing (not shown). The wall consists of 8-in. lightweight reinforced concrete masonry units with an average weight of 90 psf. The composite concrete floor slab construction spans over simply supported steel beams, with a tributary width of 10 ft, and weighs 50 psf.

The estimated uniform dead load for structural steel framing, fireproofing, architectural features, floor finish, and ceiling tiles equals 24 psf, and for mechanical ducting, piping, and electrical systems equals 6 psf.



P2.2

Uniform Dead Load W_{DL} Acting on the Wide Flange Beam:

Wall Load:

$9.5(0.09) = 0.855$ klf

Floor Slab:

$10(0.05) = 0.50$ klf

Steel Framing, Fireproofing, Architectural Features, Floor Finish, and Ceiling Tiles:

$10(0.024) = 0.24$ klf

Mechanical Ducting, Piping, and Electrical Systems:

$10(0.006) = 0.06$ klf

Total $W_{DL} = 1.66$ klf

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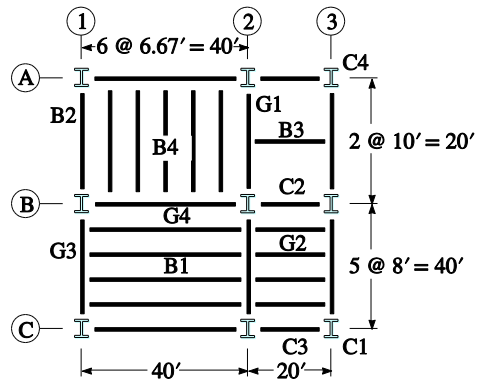
P2.3. Consider the floor plan shown in Figure P2.3. Compute the tributary areas for (*a*) floor beam B1, (*b*) floor beam B2, (*c*) girder G1, (*d*) girder G2, (*e*) corner column C1, and (*f*) interior column C2.



P2.3

(*a*) Method 1: $A_T = \left(\frac{8}{2}+\frac{8}{2}\right)(40) = \boxed{A_T = 320\ \text{ft}^2}$

Method 2: $A_T = 320 - 4\left(\frac{1}{2}4(4)\right) = \boxed{A_T = 288\ \text{ft}^2}$

(*b*) Method 1: $A_T = \left(\frac{6.67}{2}\right)(20) = \boxed{A_T = 66.7\ \text{ft}^2}$

Method 2: $A_T = 66.7 - 2\left(\frac{1}{2}3.33(3.33)\right) = \boxed{A_T = 55.6\ \text{ft}^2}$

(*c*) Method 1: $A_T = \left(\frac{6.67}{2}\right)(20) + 10(10)$

$\boxed{A_T = 166.7\ \text{ft}^2}$

Method 2: $A_T = 166.7 - 2\left(\frac{1}{2}3.33(3.33)\right) + 2\left(\frac{1}{2}5(5)\right)$

$\boxed{A_T = 180.6\ \text{ft}^2}$

(*d*) Method 1: $A_T = \left(\frac{40}{2}+\frac{20}{2}\right)(36)$

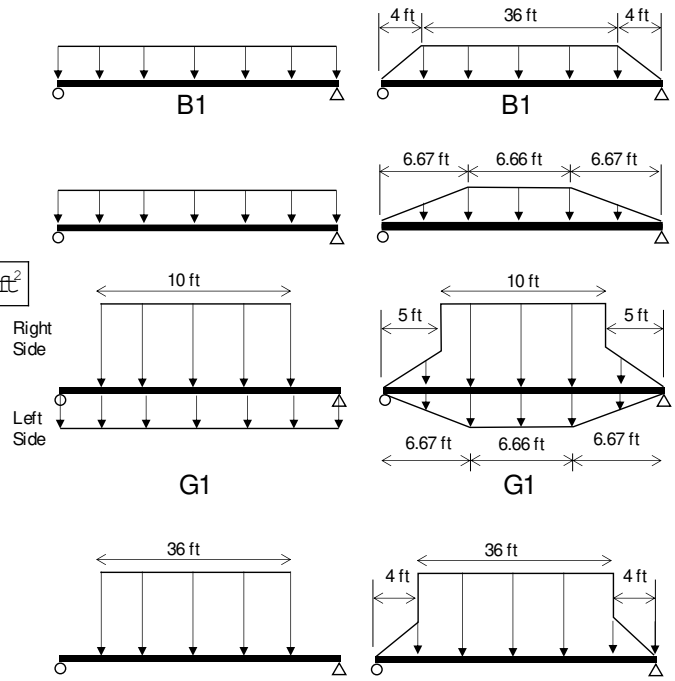
$\boxed{A_T = 1080\ \text{ft}^2}$

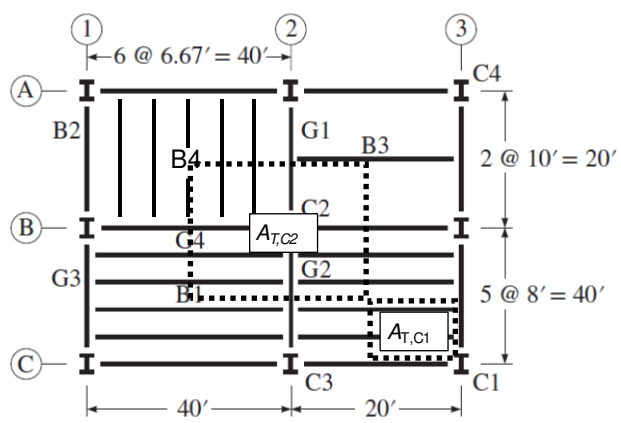
Method 2: $A_T = 1080 + 2\left(\frac{1}{2}4(4)\right)$

$\boxed{A_T = 1096\ \text{ft}^2}$

(*e*) $A_T = \left(\frac{40}{2}\right)\left(\frac{20}{2}\right)$; $\boxed{A_T = 200\ \text{ft}^2}$

(*f*) $A_T = \left(\frac{40}{2}+\frac{20}{2}\right)\left(\frac{40}{2}+\frac{20}{2}\right)$; $\boxed{A_T = 900\ \text{ft}^2}$





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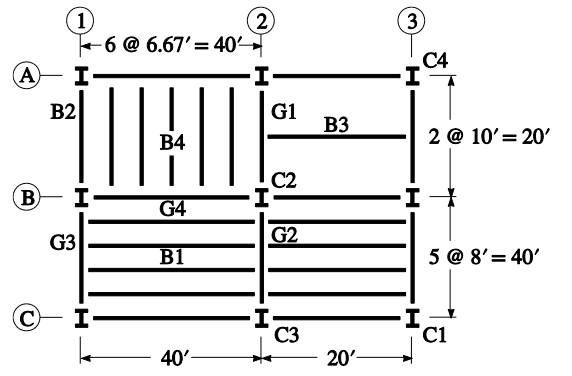
P2.4. Refer to Figure P2.3 for the floor plan. Calculate the tributary areas for (*a*) floor beam B3, (*b*) floor beam B4, (*c*) girder G3, (*d*) girder G4, (*e*) edge column C3, and (*f*) corner column C4.



P2.3

(*a*) Method 1: $A_T = (10)(20)$

$$\boxed{A_T = 200\ \text{ft}^2}$$

Method 2: $A_T = 200 - 4\left(\frac{1}{2}5^2\right)$

$$\boxed{A_T = 150\ \text{ft}^2}$$

(*b*) Method 1: $A_T = (6.67)(20) = \boxed{A_T = 133.4\ \text{ft}^2}$

Method 2: $A_T = 133.4 - 4\left(\frac{1}{2}3.33^2\right)$

$$\boxed{A_T = 111.2\ \text{ft}^2}$$

(*c*) Method 1: $A_T = (36)(20) = \boxed{A_T = 720\ \text{ft}^2}$

Method 2: $A_T = 720 + 2\left(\frac{1}{2}4^2\right) = \boxed{A_T = 736\ \text{ft}^2}$

(*d*) Method 1: $A_T = (4)(40) + 33.33(10)$

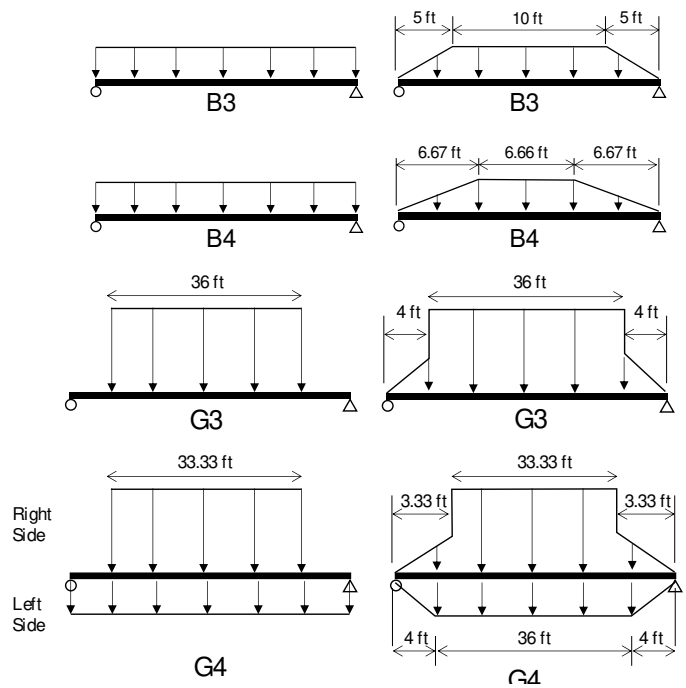
$$\boxed{A_T = 493.4\ \text{ft}^2}$$

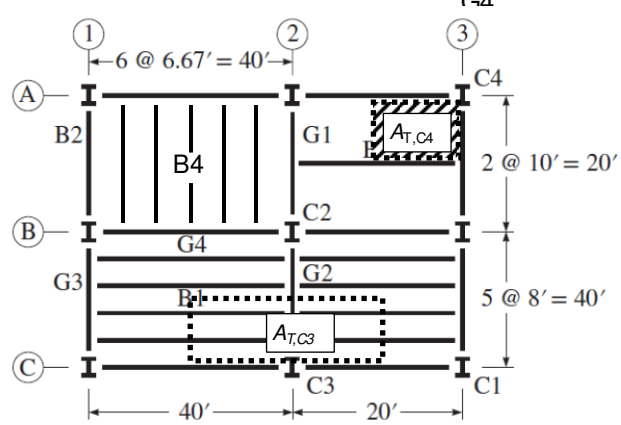
Method 2: $A_T = 493.4 - 2\left(\frac{1}{2}4^2\right) + 2\left(\frac{1}{2}3.33^2\right)$

$$\boxed{A_T = 488.5\ \text{ft}^2}$$

(*e*) $A_T = (30)(20)$; $\boxed{A_T = 600\ \text{ft}^2}$

(*f*) $A_T = (10)(10)$; $\boxed{A_T = 100\ \text{ft}^2}$





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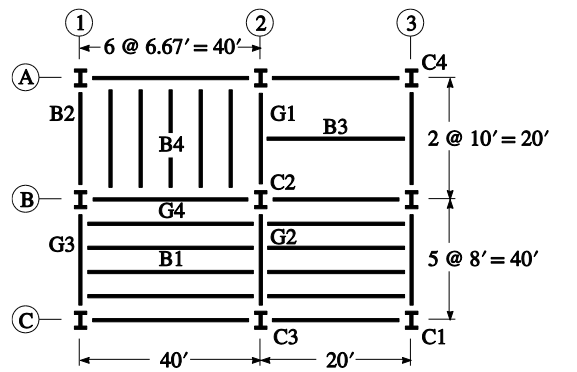
P2.5. The uniformly distributed live load on the floor plan in Figure P2.3 is 60 lb/ft^2. Establish the live loading for members (*a*) floor beam B1, (*b*) floor beam B2, (*c*) girder G1, and (*d*) girder G2. Consider the live load reduction if permitted by the ASCE standard.

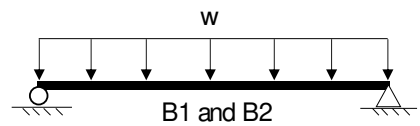


P2.3

(*a*) $A_T = 8(40) = 320 \text{ ft}^2,\ K_{LL} = 2,\ A_T K_{LL} = 640 > 400$

$$L = 60\left(0.25 + \frac{15}{\sqrt{640}}\right) = 50.6 \text{ psf} > \frac{60}{2}, \text{ok}$$

$$\boxed{w = 8(50.6) = 404.8 \text{ lb/ft} = 0.40 \text{ kips/ft}}$$



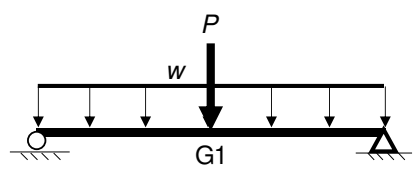
(*b*) $A_T = \frac{6.67}{2}(20) = 66.7 \text{ ft}^2,\ K_{LL} = 2,\ A_T K_{LL} = 133.4 < 400, \text{No Reduction}$

$$\boxed{w = \frac{6.67}{2}(60) = 200.1 \text{ lb/ft} = 0.20 \text{ kips/ft}}$$

(*c*) $A_T = \frac{6.67}{2}(20) + 10(10) = 166.7 \text{ ft}^2,\ K_{LL} = 2,\ A_T K_{LL} = 333.4 < 400, \text{No Reduction}$

$$w = \frac{6.67}{2}(60) = 200.1 \text{ lb/ft} = 0.20 \text{ kips/ft}$$

$$P = \frac{q(W_{trib})(L_{beam})}{2} = \frac{60(10)(20)}{2} = 6000 \text{ lbs} = 6 \text{ kips}$$

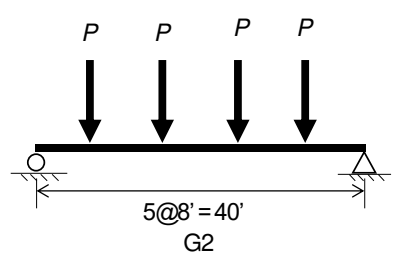


(*d*) $A_T = \left(\frac{40}{2} + \frac{20}{2}\right)36 = 1080 \text{ ft}^2,\ K_{LL} = 2,\ A_T K_{LL} = 2160 > 400$

$$L = 60\left(0.25 + \frac{15}{\sqrt{2160}}\right) = 34.4 > \frac{60}{2}, \text{ok}$$

$L = 34.4$ psf

$$\boxed{P = 8(34.4)\left(\frac{40}{2} + \frac{20}{2}\right) = 8256 \text{ lbs} = 8.26 \text{ kips}}$$



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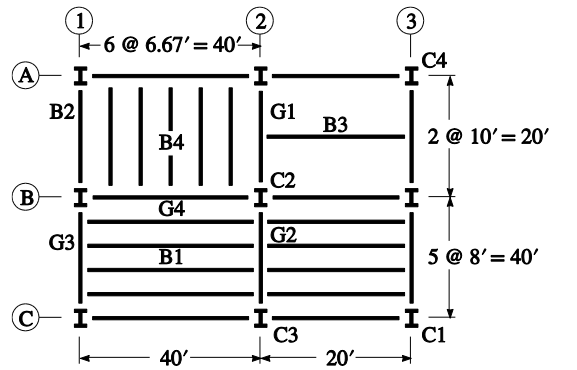
P2.6. The uniformly distributed live load on the floor plan in Figure P2.3 is 60 lb/ft^2. Establish the live loading for members (*a*) floor beam B3, (*b*) floor beam B4, (*c*) girder G3, and girder G4. Consider the live load reduction if permitted by the ASCE standard.

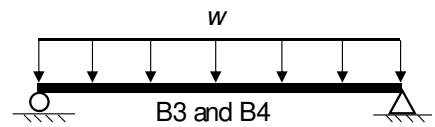


P2.3

(*a*) $A_T = 10(20) = 200\ \text{ft}^2,\ K_{LL} = 2,\ A_T K_{LL} = 400 > 400$

$$L = 60\left(0.25 + \frac{15}{\sqrt{400}}\right) = 60\ \text{psf}$$

$$\boxed{w = 10(60) = 600\ \text{lb/ft} = 0.60\ \text{kips/ft}}$$



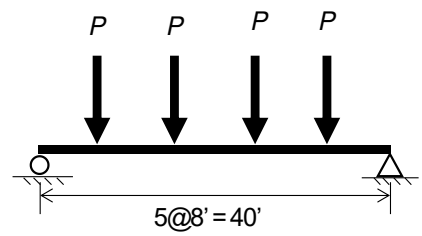
(*b*) $A_T = 6.67(20) = 133.4\ \text{ft}^2,\ K_{LL} = 2,\ A_T K_{LL} = 266.8 < 400,$ No Reduction

$$\boxed{w = 6.67(60) = 400.2\ \text{lb/ft} = 0.40\ \text{kips/ft}}$$

(*c*) $A_T = 36(20) = 720\ \text{ft}^2,\ K_{LL} = 2,\ A_T K_{LL} = 1440 > 400$

$$L = 60\left(0.25 + \frac{15}{\sqrt{1440}}\right) = 38.7\ \text{psf} > \frac{60}{2},\ \text{ok}$$

$$\boxed{P = \frac{q(W_{trib})(L_{beam})}{2} = \frac{38.7(8)(40)}{2} = 6192\ \text{lbs} = 6.19\ \text{kips}}$$



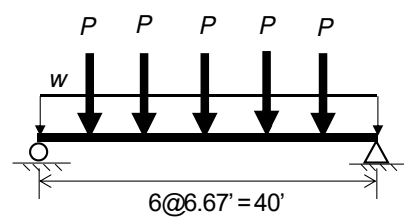
G3

(*d*) $A_T = \left(\frac{8}{2}\right)40 + 33.33(10) = 493.3\ \text{ft}^2,\ K_{LL} = 2,\ A_T K_{LL} = 986.6 > 400$

$$L = 60\left(0.25 + \frac{15}{\sqrt{986.6}}\right) = 43.7 > \frac{60}{2},\ \text{ok}$$

$$w = 43.7(4) = 174.8\ \text{lb/ft} = 0.17\ \text{kips/ft}$$

$$P = \frac{43.7(6.67(20))}{2} = 2914.8\ \text{lbs} = \boxed{2.91\ \text{kips}}$$



G4

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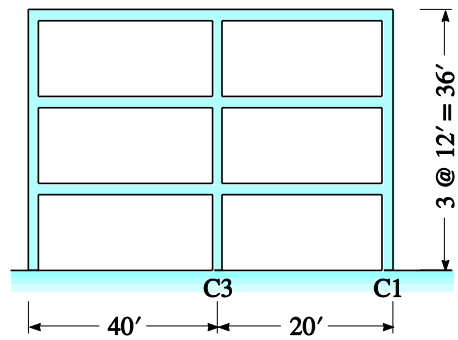
P2.7. The building section associated with the floor plan in Figure P2.3 is shown in Figure P2.7. Assume a live load of 60 lb/ft^2 on all three floors. Calculate the axial forces produced by the live load in column C2 in the third and first stories. Consider any live load reduction if permitted by the ASCE standard.



Building Section

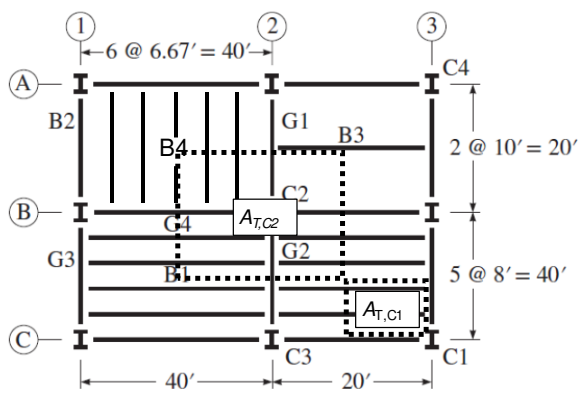
P2.7

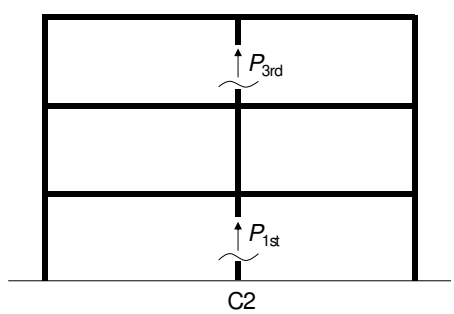
(a) $A_T = \left(\frac{40}{2}+\frac{20}{2}\right)\left(\frac{40}{2}+\frac{20}{2}\right) = 900\ \text{ft}^2,\ K_{LL} = 4,\ A_T K_{LL} = 3600 > 400$

$L = 60\left(0.25+\frac{15}{\sqrt{3600}}\right) = 30\ \text{psf} = \frac{60}{2}$, ok (minimum permitted)

$\boxed{P_{3rd} = 900\,(30) = 27000\ \text{lbs} = 27\ \text{kips}}$

$\boxed{P_{1st} = (3)900\,(30) = 27000\ \text{lbs} = 81\ \text{kips}}$





ELEVATION

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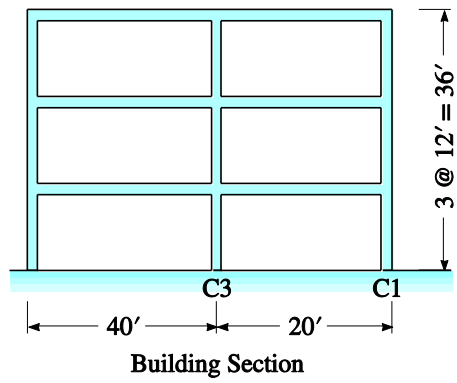
P2.8. The building section associated with the floor plan in Figure P2.3 is shown in Figure P2.7. Assume a live load of 60 lb/ft^2 on all three floors. Calculate the axial forces produced by the live load in column C3 in the third and first stories. Consider any live load reduction if permitted by the ASCE standard.



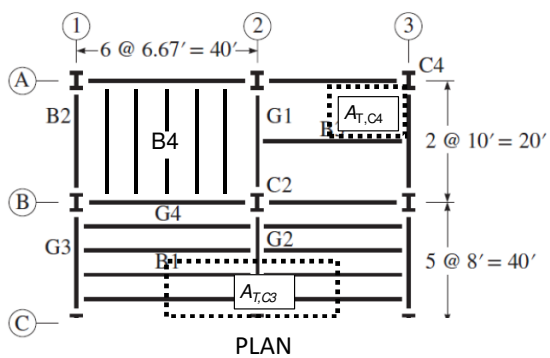
P2.7

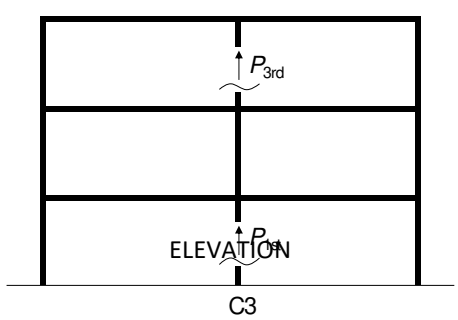
(a) $A_T = \left(\frac{40}{2} + \frac{20}{2}\right)20 = 600 \text{ ft}^2,\ K_{LL} = 4,\ A_T K_{LL} = 2400 > 400\partial$

$L = 60\left(0.25 + \frac{15}{\sqrt{2400}}\right) = 33.4 \text{ psf} = \frac{60}{2}, \text{ok}$

$\boxed{P_{3rd} = 600(33.4) = 20040 \text{ lbs} = 20.0 \text{ kips}}$

$\boxed{P_{1st} = (3)600(33.4) = 60120 \text{ lbs} = 60.1 \text{ kips}}$





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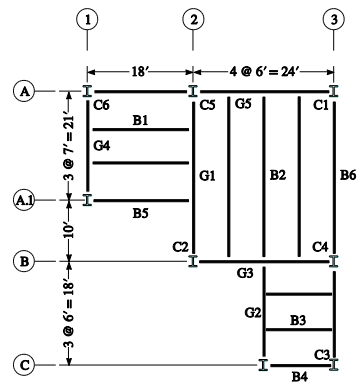
P2.9. Consider the floor plan shown in Figure P2.9. Compute the tributary areas for (*a*) floor beam B1, (*b*) floor beam B2, (*c*) girder G1, (*d*) girder G2, (*e*) corner column C1, and (*f*) corner column C2.



P2.9

(*a*) Method 1: $A_T = \left(\frac{7}{2}+\frac{7}{2}\right)(18) = \boxed{A_T = 126\ ft^2}$

Method 2: $A_T = 126 - 4\left(\frac{1}{2}4(3.5)\right) = \boxed{A_T = 98\ ft^2}$

(*b*) Method 1: $A_T = \left(\frac{6}{2}+\frac{6}{2}\right)(31) = \boxed{A_T = 186\ ft^2}$

Method 2: $A_T = 186 - 4\left(\frac{1}{2}3(3)\right) = \boxed{A_T = 168\ ft^2}$

(*c*) *Method* 1: $A_T = (3)(31) + 14(9)$

$\boxed{A_T = 219\ ft^2}$

Method 2: $A_T = 219 - 2\left(\frac{1}{2}3^2\right) + 2\left(\frac{1}{2}3.5^2\right)$

$\boxed{A_T = 222.25\ ft^2}$

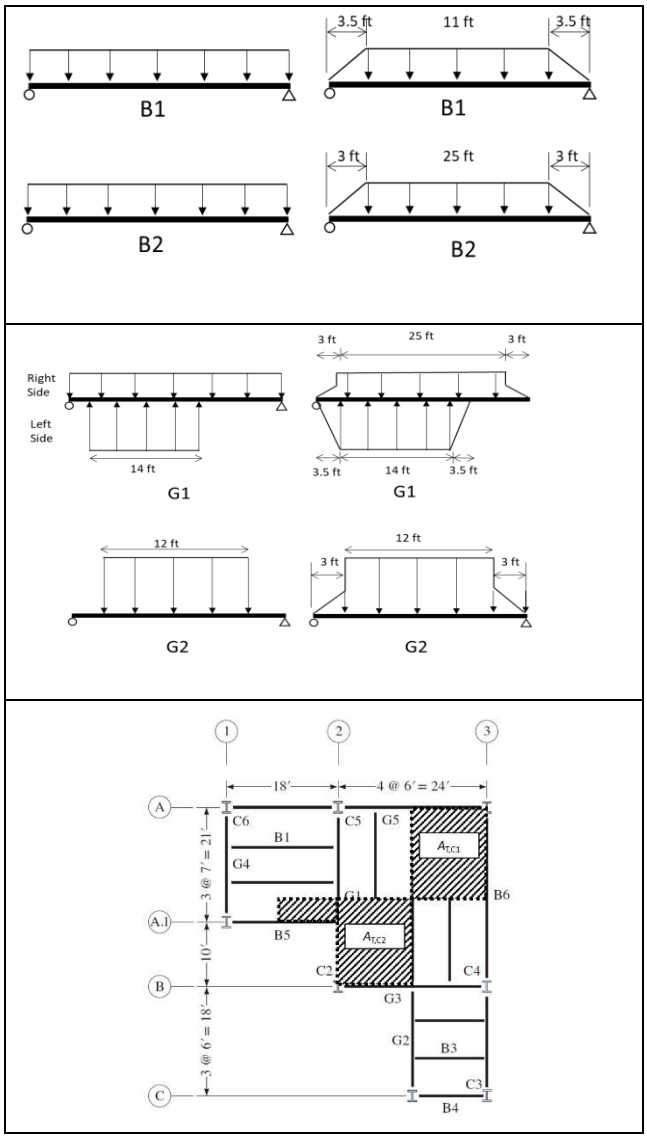
(*d*) Method 1: $A_T = (6)(12) = \boxed{72\ ft^2}$

Method 2: $A_T = 72 + 2\left(\frac{1}{2}3^2\right)$ $\boxed{A_T = 81\ ft^2}$

(*e*) $A_T = (12)\left(\frac{31}{2}\right)$; $\boxed{A_T = 186\ ft^2}$

(*f*) $A_T = (12)\left(\frac{31}{12}\right) + (9)\left(\frac{31}{2}\right) - (9)(10)$;

$\boxed{A_T = 235.5\ ft^2}$



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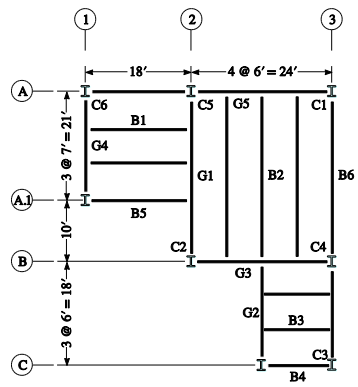
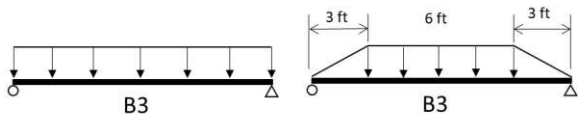
P2.10. Refer to Figure P2.9 for the floor plan. Calculate the tributary areas for (*a*) floor beam B3, (*b*) floor beam B4, (*c*) girder G3, (*d*) girder G4, (*e*) corner column C3, and (*f*) exterior column C4.

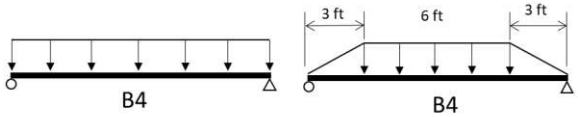


P2.9



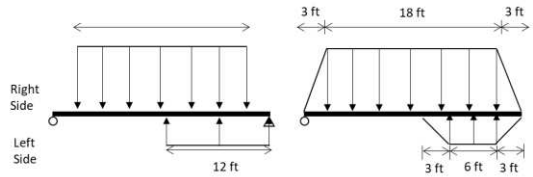
(*a*) Method 1: $A_T = \left(\frac{6}{2}+\frac{6}{2}\right)(6) = \boxed{A_T = 36\ \text{ft}^2}$

Method 2: $A_T = 36 - 4\left(\frac{1}{2}3(3)\right) = \boxed{A_T = 18\ \text{ft}^2}$



(*b*) Method 1: $A_T = \left(\frac{6}{2}\right)(6) = \boxed{A_T = 18\ \text{ft}^2}$

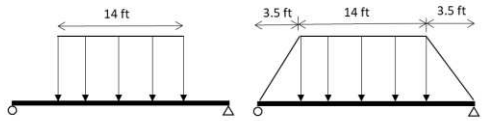
Method 2: $A_T = 18 - 2\left(\frac{1}{2}3(3)\right) = \boxed{A_T = 9\ \text{ft}^2}$



G3 G3

(*c*) Method 1: $A_T = (18)\left(\frac{31}{2}\right) + 12\left(\frac{6}{2}\right) = \boxed{A_T = 315\ \text{ft}^2}$

Method 2: $A_T = 315 - 2\left(\frac{1}{2}3^2\right) + 2\left(\frac{1}{2}3^2\right) = \boxed{A_T = 315\ \text{ft}^2}$



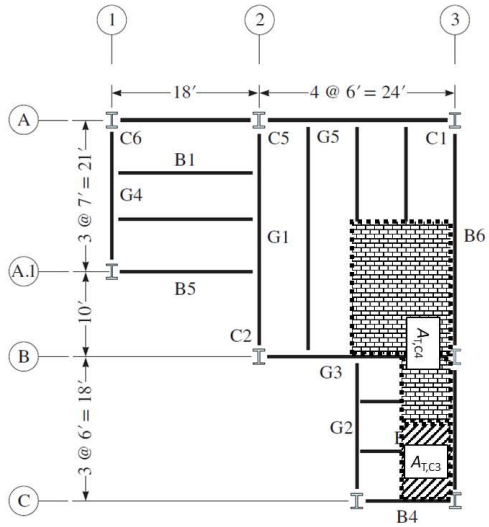
G4 G4

(*d*) Method 1: $A_T = (14)\left(\frac{18}{2}\right)$; $\boxed{A_T = 126\ \text{ft}^2}$

Method 2: $A_T = 126 + 2\left(\frac{1}{2}3.5^2\right) = \boxed{A_T = 138.25\ \text{ft}^2}$

(*e*) $A_T = (6)\left(\frac{18}{2}\right)$; $\boxed{A_T = 54\ \text{ft}^2}$

(*f*) $A_T = (6)\left(\frac{18}{2}\right) + (12)\left(\frac{31}{2}\right) = \boxed{A_T = 240\ \text{ft}^2}$



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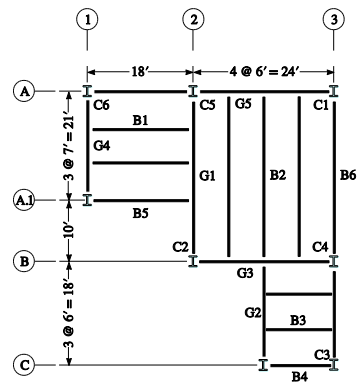
P2.11. Consider the floor plan shown in Figure P2.9. Compute the tributary areas for (*a*) floor beam B5, (*b*) floor beam B6, (*c*) girder G5, (*d*) exterior column C5, and (*f*) corner column C6.



P2.9

(*a*) Method 1: $A_T = \left(\frac{7}{2}\right)(18) = \boxed{A_T = 63\ ft^2}$

Method 2: $A_T = 63 - 2\left(\frac{1}{2}3.5(3.5)\right) = \boxed{A_T = 50.75\ ft^2}$

(*b*) Method 1: $A_T = \left(\frac{6}{2}\right)(31) = \boxed{A_T = 93\ ft^2}$

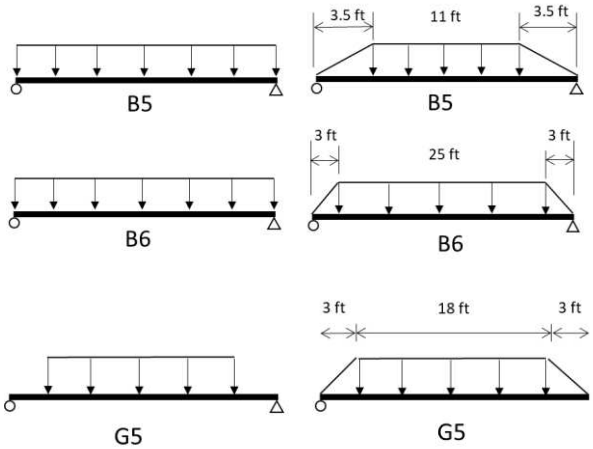
Method 2: $A_T = 93 - 2\left(\frac{1}{2}3(3)\right) = \boxed{A_T = 84\ ft^2}$

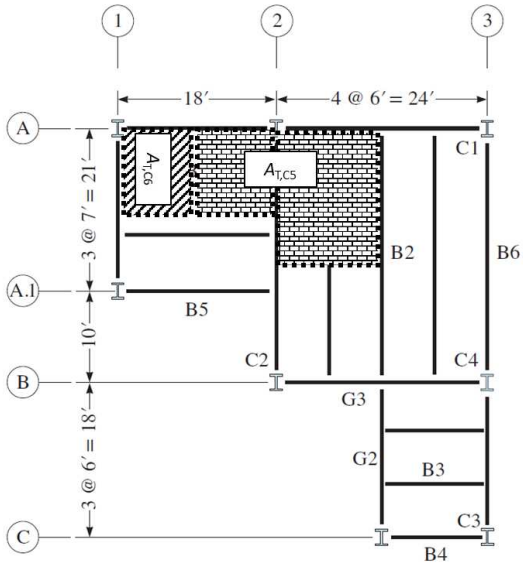
(*c*) Method 1: $A_T = (18)\left(\frac{31}{2}\right); \boxed{A_T = 279\ ft^2}$

Method 2: $A_T = 279 + 2\left(\frac{1}{2}3^2\right); \boxed{A_T = 288\ ft^2}$

(*d*) $A_T = (12)\left(\frac{31}{2}\right) + \left(\frac{21}{2}\right)\left(\frac{18}{2}\right); \boxed{A_T = 280.5\ ft^2}$

(*e*) $A_T = \left(\frac{21}{2}\right)\left(\frac{18}{2}\right); \boxed{A_T = 94.5\ ft^2}$





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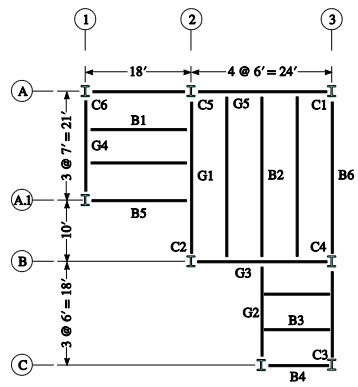
P2.12. The uniformly distributed live load on the floor plan in Figure P2.9 is 100 lb/ft^2. Establish the live loading for members (*a*) floor beam B1, (*b*) floor beam B2, (*c*) girder G1, and (*d*) girder G2. Consider the live load reduction if permitted by the ASCE standard.

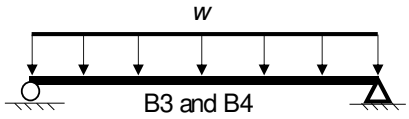


P2.9

(*a*) $A_T = \left(\frac{7}{2}+\frac{7}{2}\right)(18) = 126\ \text{ft}^2,\ K_{LL} = 2,\ A_T K_{LL} = 252 < 400;$

No Reduction

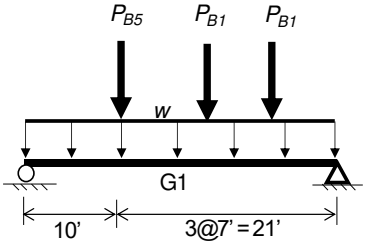
$$\boxed{w = 7(100) = 700\ \text{lb/ft} = 0.70\ \text{kips/ft}}$$



(*b*) $A_T = \left(\frac{6}{2}+\frac{6}{2}\right)(31) = 186\ \text{ft}^2,\ K_{LL} = 2,\ A_T K_{LL} = 372 < 400;$

No Reduction

$$\boxed{w = 6(100) = 600\ \text{lb/ft} = 0.60\ \text{kips/ft}}$$



(*c*) $A_T = \left(\frac{6}{2}\right)31 + \left(\frac{18}{2}\right)(21) = 282\ \text{ft}^2,\ K_{LL} = 2,\ A_T K_{LL} = 564 > 400$

$$L = 100\left(0.25 + \frac{15}{\sqrt{564}}\right) = 88.2 > \frac{100}{2},\ \text{ok}$$

$$w = 88.2(3) = 264.6\ \text{lb/ft} = 0.26\ \text{kips/ft}$$

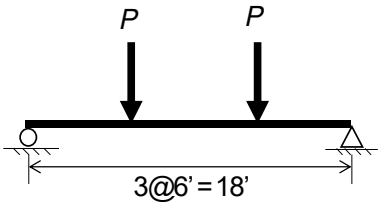
$$P_{B1} = \frac{88.2(7)(18)}{2} = 5556.6\ \text{lbs} = 5.56\ \text{kips}$$

$$P_{B5} = \frac{88.2(3.5)(18)}{2} = 2778.3\ \text{lbs} = 2.78\ \text{kips}$$

(*d*) $A_T = 18(6) = 108\ \text{ft}^2,\ K_{LL} = 2,\ A_T K_{LL} = 216 < 400$

No Reduction

$$P = \frac{100(6)(12)}{2} = 3600\ \text{lbs} = \boxed{3.6\ \text{kips}}$$



G2

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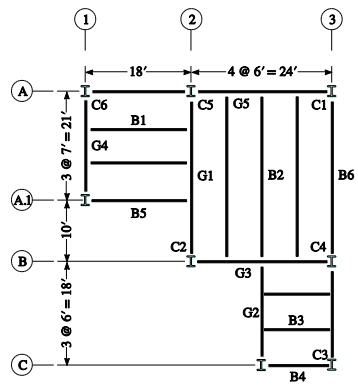
P2.13. The uniformly distributed live load on the floor plan in Figure P2.9 is 100 lb/ft^2. Establish the live loading for members (*a*) floor beam B3, (*b*) floor beam B4, (*c*) girder G3, and girder G4. Consider the live load reduction if permitted by the ASCE standard.

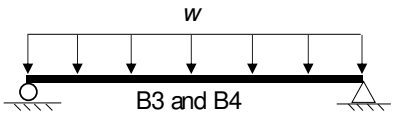


P2.9

(*a*) $A_T = \left(\frac{6}{2}+\frac{6}{2}\right)(12) = 72\ \text{ft}^2,\ K_{LL} = 2,\ A_T K_{LL} = 144 < 400;$

No Reduction

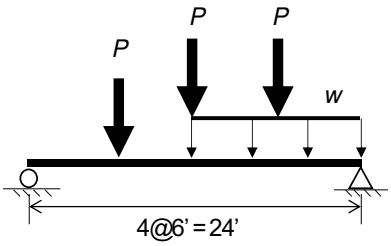
$\boxed{w = 6(100) = 600\ \text{lb/ft} = 0.60\ \text{kips/ft}}$



(*b*) $A_T = \left(\frac{6}{2}\right)(12) = 36\ \text{ft}^2,\ K_{LL} = 2,\ A_T K_{LL} = 72 < 400;$

No Reduction

$\boxed{w = 3(100) = 300\ \text{lb/ft} = 0.30\ \text{kips/ft}}$



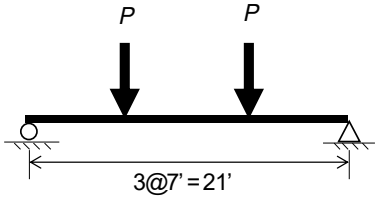
G3

(*c*) $A_T = \left(\frac{6}{2}\right)12 + \left(\frac{31}{2}\right)(18) = 315\ \text{ft}^2,\ K_{LL} = 2,\ A_T K_{LL} = 630 > 400$

$L = 100\left(0.25 + \frac{15}{\sqrt{630}}\right) = 84.76 > \frac{100}{2}$, ok

$w = 84.76(3) = 254.3\ \text{lb/ft} = 0.25\ \text{kips/ft}$

$P = \frac{84.76(6)(31)}{2} = 7882.68\ \text{lbs} = 7.88\ \text{kips}$



G4

(*d*) $A_T = 14\left(\frac{18}{2}\right) = 126\ \text{ft}^2,\ K_{LL} = 2,\ A_T K_{LL} = 252 < 400$

No Reduction

$P = \frac{100(7)(18)}{2} = 6300\ \text{lbs} = \boxed{6.3\ \text{kips}}$

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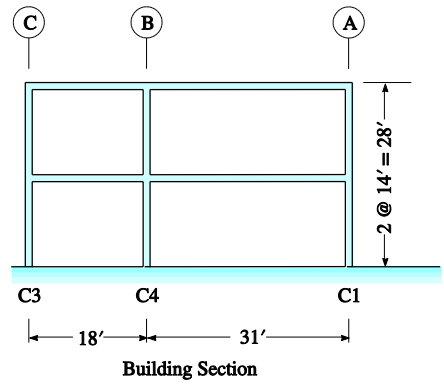
P2.14. The building section associated with the floor plan in Figure P2.9 is shown in Figure P2.14. Assume a live load of 60 lb/ft^2 on the roof and 100 lb/ft^2 on the floor. Calculate the axial forces produced by the live load in column C2 in the first and second stories. Consider any live load reduction if permitted by the ASCE standard.



P2.14

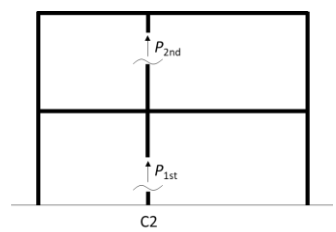
$$A_T = (12)\left(\frac{31}{2}\right) + (9)\left(\frac{31}{2} - 10\right) = 235.5 \text{ ft}^2,\ K_{LL} = 4,\ A_T K_{LL} = 942 > 400$$

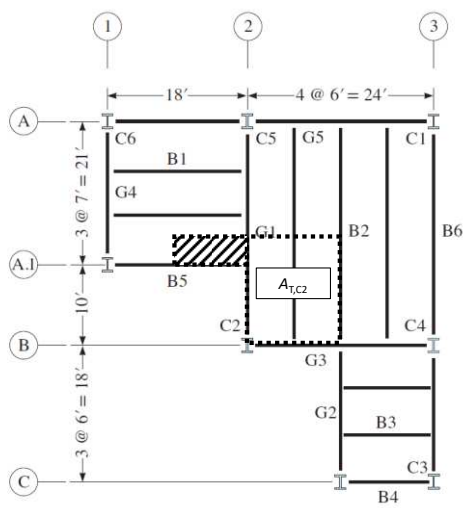
$$L_{roof} = 60\left(0.25 + \frac{15}{\sqrt{942}}\right) = 44.3 > \text{psf} = \frac{60}{2}, \text{ok}$$

$$L_{floor} = 100\left(0.25 + \frac{15}{\sqrt{942}}\right) = 73.9 > \text{psf} = \frac{100}{2}, \text{ok}$$

$$\boxed{P_{2nd} = 235.5(44.3) = 10432.7 \text{ lbs} = 10.4 \text{ kips}}$$

$$\boxed{P_{1st} = 235.5(44.3) + 235.5(73.9) = 27836 \text{ lbs} = 27.8 \text{ kips}}$$





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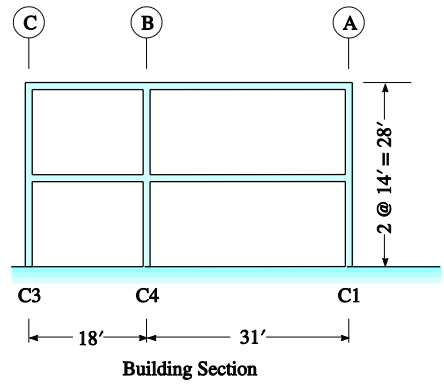
P2.15. The building section associated with the floor plan in Figure P2.9 is shown in Figure P2.14. Assume a live load of 60 lb/ft^2 on the roof and 100 lb/ft^2 on the floor. Calculate the axial forces produced by the live load in column C4 in the first and second stories. Consider any live load reduction if permitted by the ASCE standard.



P2.14

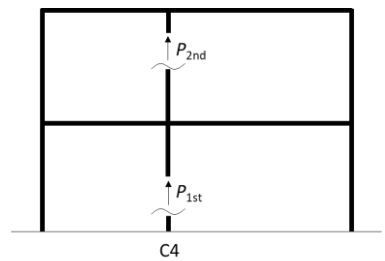
$$A_T = (6)\left(\frac{18}{2}\right) + (12)\left(\frac{31}{2}\right) = 240 \text{ ft}^2,\ K_{LL} = 4,\ A_T K_{LL} = 960 > 400$$

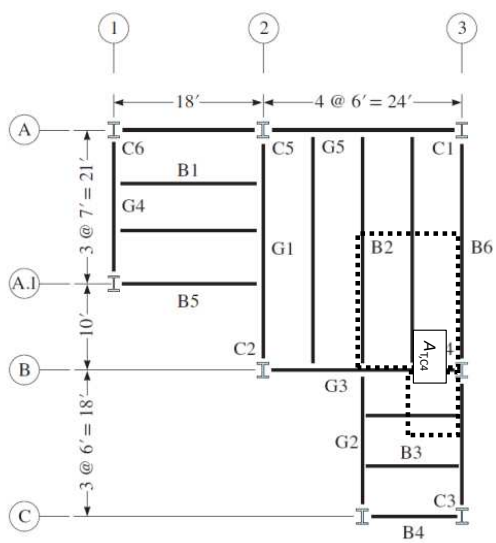
$$L_{roof} = 60\left(0.25 + \frac{15}{\sqrt{960}}\right) = 44.0 > \text{psf} = \frac{60}{2}, \text{ok}$$

$$L_{floor} = 100\left(0.25 + \frac{15}{\sqrt{960}}\right) = 73.4 > \text{psf} = \frac{100}{2}, \text{ok}$$

$$\boxed{P_{2nd} = 240(44.0) = 10560 \text{ lbs} = 10.6 \text{ kips}}$$

$$\boxed{P_{1st} = 240(44.0) + 240(73.4) = 28176 \text{ lbs} = 28.2 \text{ kips}}$$





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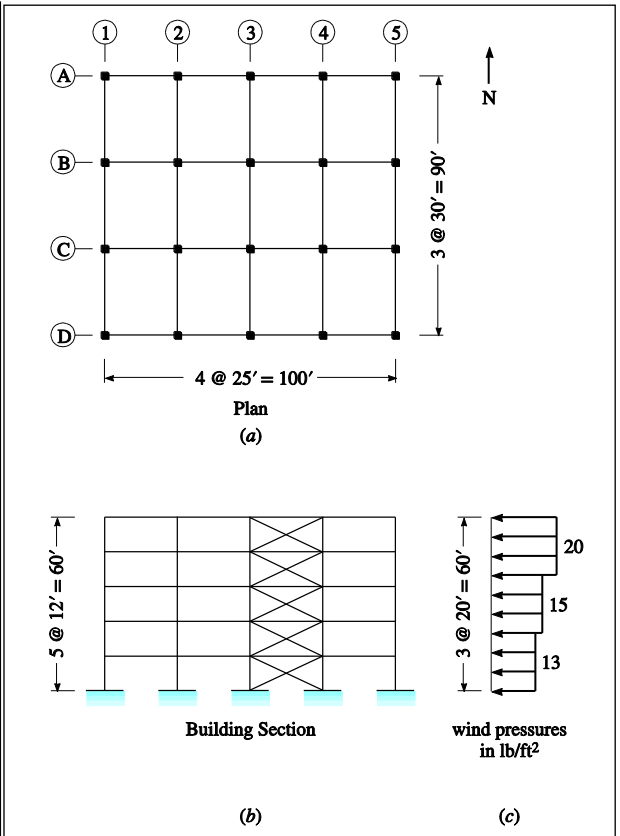
P2.16. A five-story building is shown in Figure P2.16. Following the ASCE standard, the wind pressure along the height on the windward side has been established as shown in Figure P2.16(*c*). (*a*) Considering the windward pressure in the east-west direction, use the tributary area concept to compute the resultant wind force at each floor level. (*b*) Compute the horizontal base shear and the overturning moment of the building.



P2.16

Resultant Wind Forces:

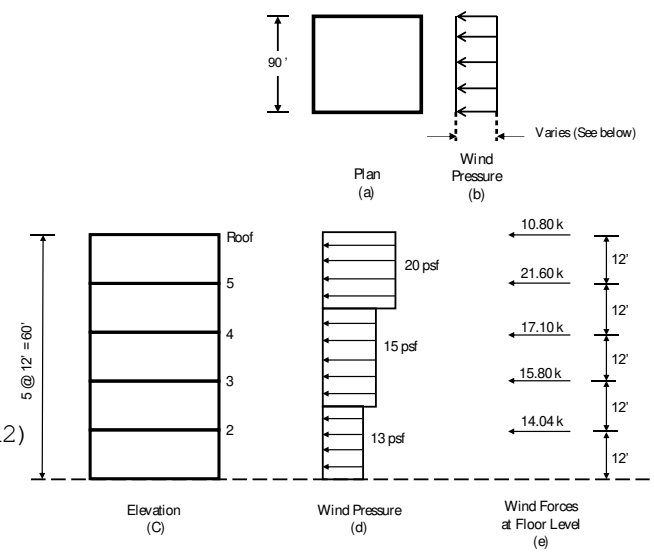
Roof: $20(6)(90) = 10{,}800$ lb

5th floor: $20(12)(90) = 21{,}600$ lb

4th floor: $20(2)(90) + 15(10)(90) = 17{,}100$ lb

3rd floor: $15(10)(90) + 13(2)(96) = 15{,}996$ lb

2nd floor: $13(12)(90) = 14{,}040$ lb



Horizontal Base Shear

$V_{BASE} = 10.8 + 21.6 + 17.1 + 15.8 + 14.04$

$\boxed{V_{BASE} = 79.34 \text{ kips}}$

Overturning Moment of the Building:

$M = 10.8(60) + 21.6(48) + 17.1(36) + 15.8(24) + 14.04(12)$

$\boxed{M = 2848 \text{ kips.ft}}$

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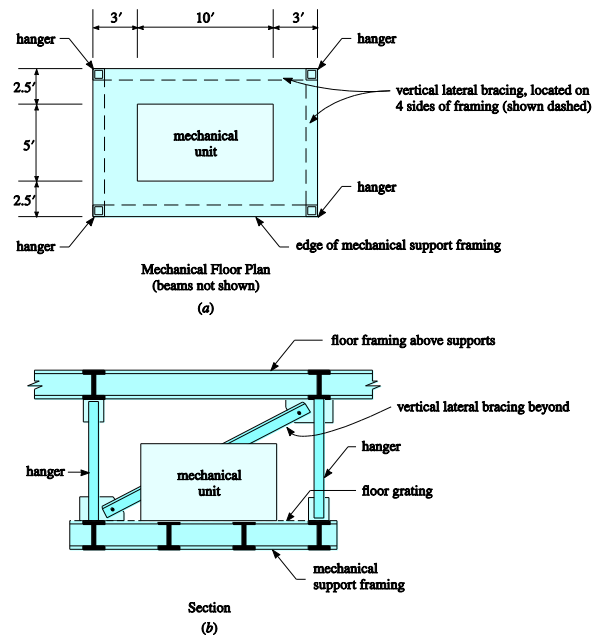
P2.17. A mechanical support framing system is shown in Figure P2.17. The framing consists of steel floor grating over steel beams and entirely supported by four tension hangers that are connected to floor framing above it. It supports light machinery with an operating weight of 4000 lb, centrally located. (*a*) Determine the impact factor *I from the Live Load Impact Factor,* Table 2.3. (*b*) Calculate the total live load acting on one hanger due to the machinery and uniform live load of 40 psf around the machine. (*c*) Calculate the total dead load acting on one hanger. The floor framing dead load is 25 psf. Ignore the weight of the hangers. Lateral bracing is located on all four edges of the mechanical floor framing for stability and transfer of lateral loads.



P2.17

a) <u>Live Load Impact Factor</u> = 20%

b) <u>Total LL</u>

$$\text{Machinery} = 1.20(4) = 4.8 \text{ kips}$$

$$\text{Uniform LL} = [10(16) - 5(10)](0.04) = 4.4 \text{ kips}$$

$$\text{Total LL} = 9.2 \text{ kips}$$

$$\text{Total LL acting on one Hanger} = \frac{9.2}{4} = \boxed{2.3 \text{ kips}}$$

c) <u>Total DL</u>

$$\text{Floor Framing} = 10(16)(0.025) = 4 \text{ kips}$$

$$\text{Total DL acting on one Hanger} = \frac{4}{4} = \boxed{1 \text{ kips}}$$

$$\text{Total DL + LL on one Hanger} = \boxed{3.3 \text{ kips}}$$

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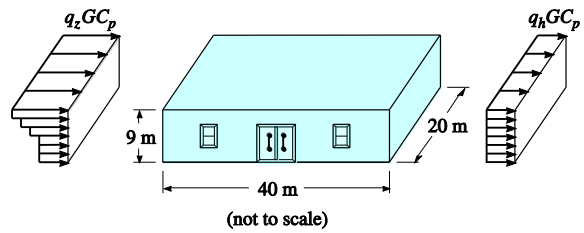
P2.18. The dimensions of a 9-m-high warehouse are shown in Figure P2.18. The windward and leeward wind pressure profiles in the long direction of the warehouse are also shown. Establish the wind forces based on the following information: basic wind speed = 40 m/s, wind exposure category = C, K_d = 0.85, K_{zt} = 1.0,
G = 0.85, and C_p = 0.8 for windward wall and −0.2 for leeward wall. Use the K_z values listed in Table 2.4. What is the total wind force acting in the long direction of the warehouse?



P2.18

Use I = 1

$q_s = 0.613V^2$ (Eq. 2.4b)

$= 0.613(40)^2 = \boxed{980.8 \text{ N/m}^2}$

$q_z = q_s I K_z K_{zt} K_d$

$q_z = 980.8(1)(K_z)(1)(0.85) = \boxed{833.7 K_z}$

$0 - 4.6\text{m}: q_z = 833.7(0.85) = 708.6 \text{ N/m}^2$

$4.6 - 6.1\text{m}: q_z = 833.7(0.90) = 750.3 \text{ N/m}^2$

$6.1 = 7.6\text{m}: q_z = 833.7(0.94) = 783.7 \text{ N/m}^2$

$7.6 = 9\text{m}: q_z = 833.7(0.98) = 817.1 \text{ N/m}^2$

For the Windward Wall

$p = q_z GC_p$ (Eq. 2.7)

where $GC_p = 0.85(0.8) = \boxed{0.68}$

$p = 0.68 q_z$

$\boxed{0 - 4.6\text{m}}$ $p = 481.8 \text{ N/m}^2$

$\boxed{4.6 - 6.1\text{m}}$ $p = 510.2 \text{ N/m}^2$

$\boxed{6.1 - 7.6\text{m}}$ $p = 532.9 \text{ N/m}^2$

$\boxed{7.6 - 9\text{m}}$ $p = 555.6 \text{ N/m}^2$

Total Wind force, F_W, Windward Wall

$F_W = 481.8(4.6)(20) + 510.2(1.5)(20) + 532.9(1.5)(20) + 555.6(1.4)(20)$

$\boxed{F_W = 91180 \text{ N}}$

For Leeward Wall

$p = q_h GC_p = q_h (0.85)(-0.2)$

$q_h = q_z$ at 9m $= 817.1 \text{ N/m}^2$ (above)

$p = 817.1(0.85)(-0.2) = \boxed{-138.9 \text{ N/m}^2}$

Total Wind force on Leeward Wall

$F_L = (20)(9)(-138.9) = \boxed{-25{,}003 \text{ N}^*}$

Total Force $= F_W + F_L$

$= 91{,}180 \text{ N} + 25{,}003$

$= \boxed{116{,}183.3 \text{ N}}$

*Both F_L and F_N Act in Same Direction.

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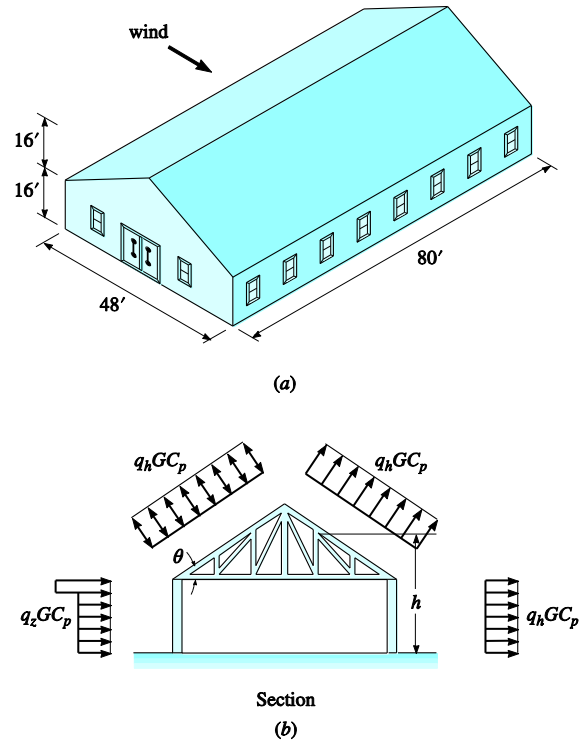
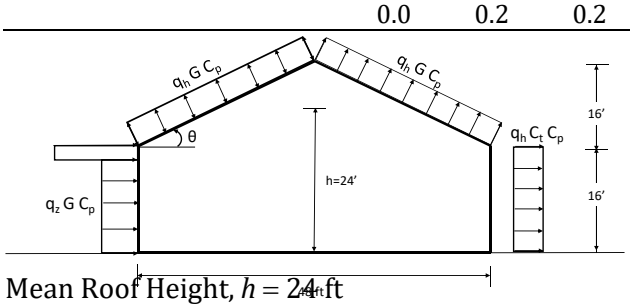
P2.19. The dimensions of an enclosed gabled building are shown in Figure P2.19*a*. The external pressures for the wind load perpendicular to the ridge of the building are shown in Figure P2.19*b*. Note that the wind pressure can act toward or away from the windward roof surface. For the particular building dimensions given, the C_p value for the roof based on the ASCE standard can be determined from Table P2.19, where plus and minus signs signify pressures acting toward and away from the surfaces, respectively. Where two values of C_p are listed, this indicates that the windward roof slope is subjected to either positive or negative pressures, and the roof structure should be designed for both loading conditions. The ASCE standard permits linear interpolation for
the value of the inclined angle of roof ϑ. But interpolation should only be carried out between values of the same sign. Establish the wind pressures on the building when positive pressure acts on the windward roof. Use the following data: basic wind speed = 100 mi/h, wind exposure category = B, K_d = 0.85, K_{zt} = 1.0, G = 0.85, and C_p = 0.8 for windward wall and –0.2 for leeward wall.



P2.19

TABLE P2.13 Roof Pressure Coefficient C_p *θ defined in Figure P2.13

	Windward								Leeward		
Angle θ	10	15	20	25	30	35	45	≥60	10	15	≥20
C_p	−0.9	−0.7	−0.4	−0.3	−0.2	−0.2	0.0	0.01θ*	−0.5	−0.5	−0.6
			0.0	0.2	0.2	0.3	0.4				



Mean Roof Height, $h = 24$ ft

$$\theta = \tan^{-1}\left(\frac{16}{24}\right) = 33.69^\circ \quad \text{(for Table 2.10)}$$

<u>Consider Positive Windward Pressure on Roof, i.e. left side.</u>

Interpolate in Table P2.10

$$C_p = 0.2 + \frac{(33.69-30)}{(35-30)}(0.1)$$

$$C_p = 0.2738 \text{ (Roof only)}$$

From Table 2.4 (see p51 of text)

$$K_z = 0.57, \boxed{0-15}$$
$$= 0.62, \boxed{15-20}$$
$$= 0.66, \boxed{20-25}$$
$$= 0.70, \boxed{25-30}$$
$$= 0.76, \boxed{30-32}$$

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P2.19. Continued

$K_{zt} = 1.0, K_d = 0.85, I = 1$

$q_s = 0.00256\,V^2$ (Eq 2.4a)

$q_s = 0.00256(100)^2 = \boxed{25.6 \text{ lb/ft}^2}$

$q_z = q_s\, I K_z\, K_{zt}\, K_d$

$\boxed{0\text{–}15}$ $q_z = 25.6\,(1)(0.57)(1)(0.85)$

$= 12.40 \text{ lb/ft}^2$

$\boxed{15\text{–}16}$ $q_z = 13.49 \text{ lb/ft}^2$

$\boxed{24}$ $q_z = 14.36 \text{ lb/ft}^2$

Wind Pressure on Windward Wall & Roof

$P = q_z\, G C_p$

Wall $\boxed{0\text{–}15}$;

$P = 12.40(0.85)(0.80) = \boxed{8.43 \text{ lb/ft}^2}$

Wall $\boxed{15-16}$;

$P = 13.49(0.85)(0.8) = \boxed{9.17 \text{ lb/ft}^2}$

Roof;

$P = 14.36(0.85)(0.2738) = \boxed{3.34 \text{ lb/ft}^2}$

Wind Pressure on Leeward Side

For Wall $P = q_h\, G\, C_p$

For $h = 24'$; $q_h = q_z = 14.36 \text{ lb/ft}^2$

$C_p = -0.2$ for wall $= 0.6$ for roof

For Wall $P = 14.36\,(0.85)(0.2)$

$P = 2.44 \text{ lb/ft}^2$

For Roof $P = 14.36\,(0.85)(-0.6)$

$= \boxed{-7.32 \text{ lb/ft}^2 \text{ (uplift)}}$

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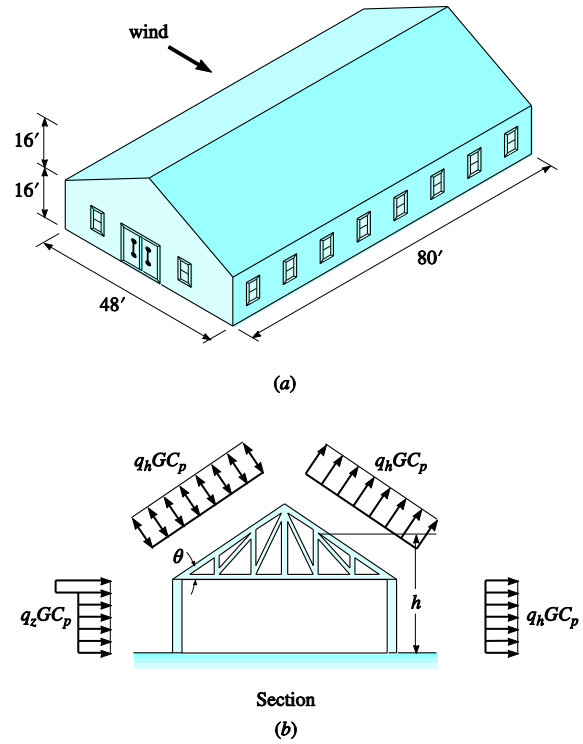
P2.20. Establish the wind pressures on the building in Problem P2.19 when the windward roof is subjected to an uplift wind force.



P2.19

TABLE P2.13 Roof Pressure Coefficient C_p *θ defined in Figure P2.11

	Windward								Leeward		
Angle θ	10	15	20	25	30	35	45	≥60	10	15	≥20
C_p	−0.9	−0.7	−0.4	−0.3	−0.2	−0.2	0.0	0.01θ*	−0.5	−0.5	−0.6
			0.0	0.2	0.2	0.3	0.4				

See P2.13 Solution

Windward Roof (Negative Pressure)

$\theta = 33.7°$

Interpolate between 30° and 35° for negative C_p value in Table P2.12

$$C_p = -0.274$$

$$p = q_h\ GC_p = 21.76\,(0.66)\ 0.85\,(-0.274)$$

$$= \boxed{-3.34\ \text{lb/ft}^2}\ \text{(Suction)}$$

Note: Wind pressures on other 3 surfaces are the same as in P2.13.

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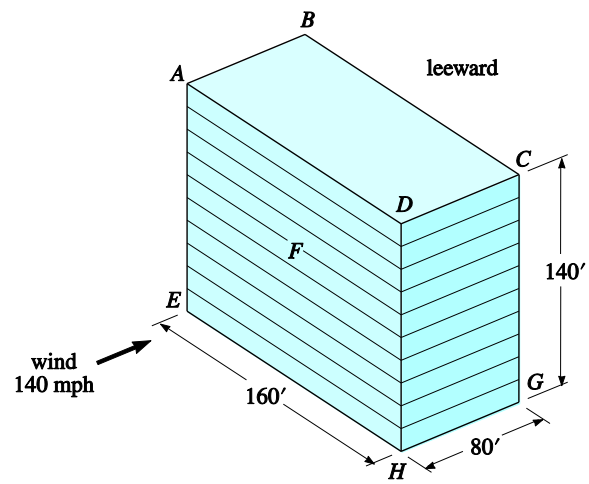
P2.21. (*a*) Determine the wind pressure distribution on the four sides of the 10-story hospital shown in Figure P2.21. The building is located near the Georgia coast where the wind velocity contour map in the ASCE Standard specifies a design wind speed of 140 mph. The building, located on level flat ground, is classified as *stiff* because its natural period is less than 1 s. On the windward side, evaluate the magnitude of the wind pressure every 35 ft in the vertical direction. (*b*) Assuming the wind pressure on the windward side varies linearly between the 35-ft intervals, determine the total wind force on the building in the direction of the wind. Include the negative pressure on the leeward side.



P2.21

(a) Compute Variation of Wind Pressure on Windward Face

$q_z = q_s\, I K_z K_{zt} K_d$ Eq 2.8

$q_s = 0.00256 V^2$ Eq 2.6a

$= 0.00256(140)^2$

$q_s = 50.176$ psf; Round to 50.18 psf

$I = 1.15$ *for hospitals*

$K_{zt} = 1;\ K_d = 0.85$

<u>K_z, Read in Table 2.4</u>

Elev. (ft)	0	35	70	105	140
K_z	1.03	1.19	1.34	1.44	1.52

$q_z = 50.18\ (1.15)(K_z)\ 1\ (0.85)$

$q_z = 49.05\ K_z$

<u>Compute Wind Pressure "*p*" on Windward Face</u>

$p = q_z G C_p = 49.05\ K_z\ G C_p$

where $G = 0.85$ for natural period less than 1 sec.

$C_p = 0.8$ on windward side

$p = 49.05\ k_z\ (0.85)(0.8) = \boxed{33.354\ K_z}$

<u>Compute "*p*" for Various Elevations</u>

Elev. (ft)	0	35	70	105	140
p (psf)	34.36	39.69	44.69	48.03	50.70

<u>Compute Wind Pressure on Leeward Wall</u>

$p = q_z\, GC_p$; Use Value of q_z at 140 ft. i.c. $K_z = 152$

$C_p = -0.5$ $\quad q_z = 49.05(1.52) = \boxed{74.556}$

$p = 74.556\ G C_p = 74.556(0.85)(-0.5)$

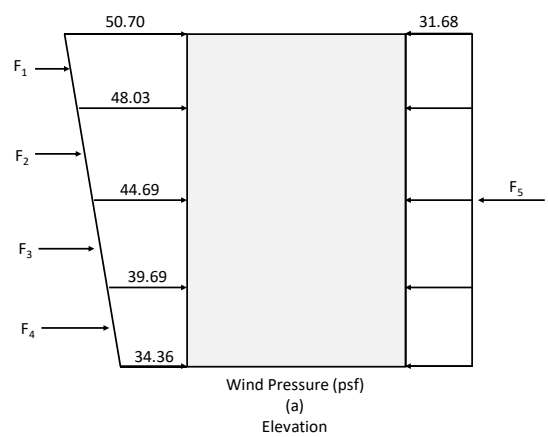
$p = \boxed{-31.68 \text{ psf}}$ ANS.

<u>Wind Pressure on Side Walls</u>

$p = q_z G C_p = 49.05(1.52)(0.85)(-0.7)$

$p = \boxed{-44.36 \text{ psf}}$

(b) Variation of Wind Pressure on Windward and Leeward Sides



P2.21. Continued

<u>Compute Total Wind Force (kips)</u>

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$$F_1 = \frac{50.7 + 48.02}{2}\left[\frac{35(1.60)}{1000}\right] = 276.42\text{ kips}$$

$$F_2 = \frac{48.03 + 44.69}{2}\left[\frac{35(1.60)}{1000}\right] = 259.62\text{ k}$$

$$F_3 = \frac{44.69 + 39.69}{2}\left[\frac{35(1.60)}{1000}\right] = 236.26\text{ k}$$

$$F_4 = \frac{39.69 + 34.36}{2}\left[\frac{35(1.60)}{1000}\right] = 207.39\text{ k}$$

$$F_5 = \frac{31.68\big((1.40)(1.60)\big)}{1000} = 709.63\text{ k}$$

$$\text{Total Wind Force} = \Sigma F_1 + F_2 + F_3 + F_4 + F_5$$

$$= \boxed{1689.27\text{ kips}}$$

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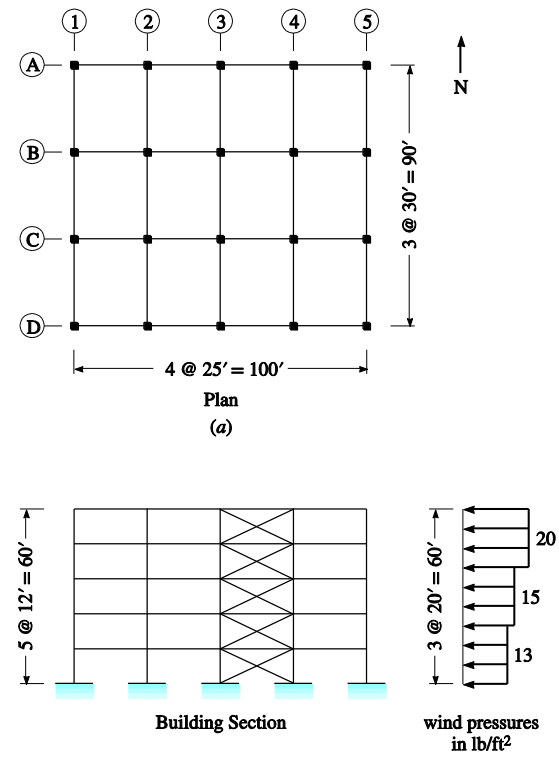
P2.22. Consider the five-story building shown in Figure P2.16. The average weights of the floor and roof are 90 lb/ft^2 and 70 lb/ft^2, respectively. The values of S_{DS} and S_{D1} are equal to 0.9*g* and 0.4*g*, respectively. Since *steel* moment frames are used in the north-south direction to resist the seismic forces, the value of *R* equals 8. Compute the seismic base shear *V*. Then distribute the base shear along the height of the building.



P2.16

Fundamental Period

$T = C_t h_n^x$; $C_t = 0.035$ and x = 0.75 for steel moment frames.

$T = 0.035(60)^{3/4}$

$T = \boxed{0.75 \text{ sec.}}$

$W = 4(100)(90)(90) + (100)(90)(70)$

$= 3{,}870{,}000 \text{ lbs} = \boxed{3{,}870 \text{ kips}}$

$V = \dfrac{S_{D1} W}{T(R/I)}$ $I = 1$ for office bldgs.

$V = \dfrac{0.4(3870)}{0.75(8/1)} = \boxed{258 \text{ kips}}$

$V_{max} = \dfrac{S_{DS} W}{R/I} = \dfrac{0.9(3870)}{8/1}$

$= \boxed{435 \text{ kips}}$

$V_{min} = 0.0441\, I\, S_{DS}\, W$

$= 0.0441\,(1)(0.9)(3870)$

$= \boxed{153.6 \text{ kips}}$

Therefore, $\boxed{\text{Use } V = 258 \text{ kips}}$

$k = 1 + \dfrac{T - 0.5}{2} = 1.125$

$F_x = \dfrac{W_x h_x^k}{\sum_{i=1}^{n} W_i h_i^k} V$

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P2.22. Continued

Forces at Each Floor Level

Floor	Weight W_i (kips)	Floor Height h_i (ft)	$w_i h_i^k$ (kips.ft)	$\frac{w_x h_x^k}{\Sigma w_i h_i^k}$	F_x (kips)
Roof	630	60	63,061	0.295	76.1
5th	810	48	63,079	0.295	76.1
4th	810	36	45,638	0.213	56.0
3rd	810	24	28,922	0.135	34.8
2nd	810	12	13,261	0.062	16.0
	$\sum = 3{,}870$		$\sum = 213{,}961$		$\sum = 258$

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P2.23. When a moment frame does not exceed 12 stories in height and the story height is at least 10 ft, the ASCE standard provides a simpler expression to compute the approximate fundamental period:

$$T = 0.1N$$

where N = number of stories. Recompute T with the above expression and compare it with that obtained from Problem P2.22. Which method produces a larger seismic base shear?

<u>ASCE Approximate Fundamental Period:</u>

$T = 0.1N$

$N = 5 \qquad \therefore T = 0.5 \text{ seconds}$

$$V = \frac{0.3(6750)}{0.5(5/1)} = 810 \text{ kips}$$

The simpler approximate method produces a larger value of base shear.

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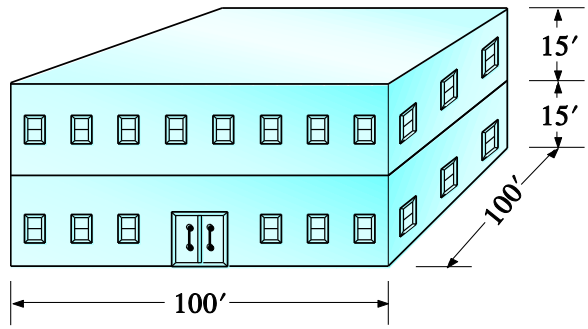
P2.24. (*a*) A two-story hospital facility shown in Figure P2.24 is being designed in New York with a basic wind speed of 90 mph and wind exposure *D*. The importance factor *I* is 1.15 and K_z = 1.0. Use the simplified procedure to determine the design wind load, base shear, and building overturning moment. (*b*) Use the equivalent lateral force procedure to determine the seismic base shear and overturning moment. The facility, with an average weight of 90 lb/ft^2 for both the floor and roof, is to be designed for the following seismic factors: S_{DS} = 0.27*g* and S_{D1} = 0.06*g*; reinforced concrete frames with
an *R* value of 8 are to be used. The importance factor *I* is 1.5. (*c*) Do wind forces or seismic forces govern the strength design of the



P2.24

(a) Wind Loads Using Simplified Procedure:
Design Wind Pressure $P_s = \lambda\, K_{zt}\, I P_{S30}$
$\lambda = 1.66$ Table 2.8, Mean Roof Height = 30'

Zones	P_{S30}	$P_S = 1.66(1)1.15P_{S30} = 1.909\,P_{S30}$
A	12.8 psf	24.44 psf
C	8.5 psf	16.22 psf

<u>Resultant Force at Each Level:</u> Where Distance $a = 0.1(100') = 10'$; $0.4(30') = 12'$; $3'$
$a = 10'$ Controls & $2a = 20'$ Region (A)

$$F_{roof}: \text{Zone (A)}: \frac{15}{2}(24.44)\frac{20}{1000} = 3.67 \text{ kips}$$

$$\text{Zone (C)}: \frac{15}{2}(16.3)\frac{80}{1000} = 9.78 \text{ kips}$$

$$\boxed{F_{roof}\ \text{Resultant} = 13.45 \text{ kips}}$$

$$F_{2nd}: \text{Zone (A)}: 15(24.44)\frac{20}{1000} = 7.33 \text{ kips}$$

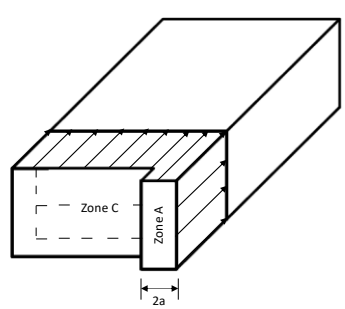
$$\text{Zone (C)}: 15(16.3)\frac{80}{1000} = 19.56 \text{ kips}$$

$$\boxed{F_{2nd}\ \text{Resultant} = 26.89 \text{ kips}}$$

Base Shear $V_{base} = F_{roof} + F_{2nd} = \boxed{40.34 \text{ kips}}$

Overturning Moment $M_{O.T.} = \Sigma F_i h_i$

$M_{O.T.} = 13.45(30) + 26.89(15) = \boxed{806.9 \text{ ft.kips}}$



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P2.24. Continued

(b) Seismic Loads by Equivalent Lateral Force Procedure Given: $W = 90$ psf Floor & Roof; $S_{DS} = 0.27$g, $S_{D1} = 0.06$g; $R = 8$, $I = 1.5$

$$\text{Base Shear } V_{base} = \frac{S_{D1} W}{T\left(R/I\right)}$$

Where W Total Building Dead Load:

$$W_{roof} = 90(10)^2 = 900$$
$$W_{2nd} = 90(10)^2 = 900$$
$$W_{total} = \boxed{1800 \text{ kips}}$$

$T = C_T h_n^x = 0.342$ sec.

$C_T = 0.016$ Reinf. Concrete Frame

$X = 0.9$ Reinf. Concrete Frame

$h = 30'$ Building Height

$$V_{base} = \frac{0.06(1800)}{(0.342)(8/1.5)} = 0.033W = 59.2 \text{ kips (controls)}$$

$$V_{max.} = \frac{S_{DS} W}{R/I} = \frac{0.27(1800)}{(8/1.5)} = 0.051W = 91.1 \text{ kips}$$

$$V_{min.} = 0.044 S_{DS} IW = 0.044(0.27)(1.5)(1800)$$
$$= 0.0178W = 32.1 \text{ kips}$$

$$\text{Force @ Each Level } F_x = \frac{w_x h_x^k}{\Sigma w_i h_i^k} V_{base}, \text{ where } V_{base} = 59.2 \text{ kips}$$

$T < 0.5$ Sec. Thus $K = 1.0$

Level	w_i (kips)	h_i (ft)	$w_i h_i^k$ (kips-ft)	$w_x h_x^k / w_i h_i^k$	Force @ Ea. Level (kips)
Roof	900	30	27,000	0.667	$F_{roof} = 39.5$
2nd	900	15	13,500	0.333	$F_{2nd} = 19.76$
			$\Sigma w_i h_i^k = 40{,}500$		$\Sigma F_x = V_{base} = 59.2$ kips

Overturning Moment $M_{OT.} = \Sigma F_x h_i$

$$M_{OT.} = 39.5(30) + 19.76(15) = \boxed{1{,}481.4 \text{ kips.ft}}$$

(c) Seismic Forces Govern the Lateral Strength Design.

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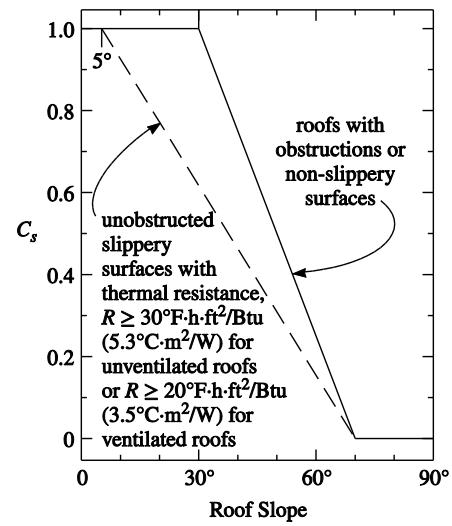
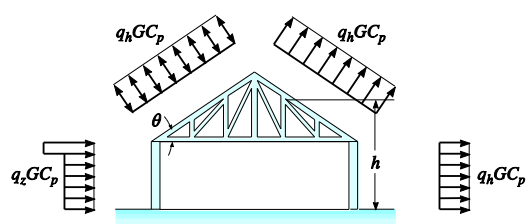
P2.25. In the gabled roof structure shown in Figure P2.19, determine the sloped roof snow load P_s. The building is heated and is located in a windy area in Boston. Its roof consists of asphalt shingles. The building is used for a manufacturing facility, placing it in a type II occupancy category. Determine the roof slope factor, C_s using the ASCE graph shown in Figure P2.25. If roof trusses are spaced at 16 ft on center, what is the uniform snow load along a truss?



Roof slope factor C_s with warm roofs and $C_t \leq 1.0$

P2.25



Section

P2.19

Sloped Roof Snow Load $P_s = C_s p_f$

Where p_f Flat Roof Snow Load

$p_f = 0.7 C_e C_t I p_g$

$C_e = 0.7$ Windy Area

$C_t = 1.0$ Heated Building

$I = 1.0$ Type II Occupancy

$p_g = 40$ psf for Boston

$C_s =$ Based on Roof Slope $\theta = \tan^{-1}\left(\frac{16}{24}\right) = 33.7°$

From Fig. P2.17 C_s is approximately 0.9 (Non-Slippery Surface)

$p_f = 0.7\,(0.7)(1.0)(1.0)(40\text{ psf}) = 19.6\text{ psf}$

$p_s = C_s P_s = 0.9\,(19.6\text{ psf}) = \boxed{17.64\text{ psf}}$

<u>Uniform Load Acting on Trusses Spaced @ 16′o.c.</u>

$W_{snow} = 17.64\,(16) = \boxed{282.2\text{ kips/ft}}$

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P2.26. A beam that is part of a rigid frame has end moments and mid-span moments for dead, live, and earthquake loads shown below. Determine the governing load combination for both negative and positive moments at the ends and mid-span of the beam. Earthquake load can act in either direction, generating both negative and positive moments in the beam.

End Moments (ft-kip)	Mid-Span Moments (ft-kip)
Dead Load −180	+90
Live Load −150	+150
Earthquake ±80	0

<u>Load Combinations-Factored Strength</u>

End Moments

$$1.4D = 1.4(-180\ \text{ft}\cdot\text{k}) \qquad = -252\ \text{kips-ft}$$

$$1.2D + 1.6L = 1.2(-180) + 1.6(-150) \qquad = -456\ \text{kips-ft}$$

$$1.2D \pm 1.0E + L = 1.2(-180) + (-80) + (-150) \qquad = -446\ \text{kips-ft}$$

Mid-Span Moments

$$1.4D = 1.4(+90) \qquad = +126\ \text{kips-ft}$$

$$1.2D + 1.6L = 1.2(+90) + 1.6(+150) \qquad = +348\ \text{kips-ft}$$

$$1.2DL \pm 1.0E + L = 1.2(90) + 0 + (150) \qquad = +258\ \text{kips-ft}$$

Beam Needs to be Designed for Max. End Moment = −456 kips-ft

Max. Mid-Span Moment = +348 kips-ft

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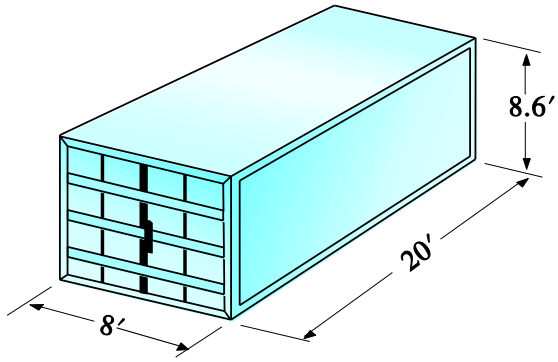
P2.27. Calculate the vertical hydrostatic load on the 5100-lb empty shipping container in Figure P2.27 subjected to a tsunami inundation height of 3′. Assuming the container is water-tight, will the tsunami wave be capable of carrying away the container as debris?



P2.27

$$F_V = \gamma_s V_W = 70.4(3)(8)(20)$$

$$F_V = 33792 \text{ lbs}$$

$$\boxed{F_V = 33.8 \text{ kips}}$$

$$33.8 \text{ kips} > W_{container} = 5.1 \text{ kips}$$

$$\boxed{\text{Yes, the container will be carried away.}}$$

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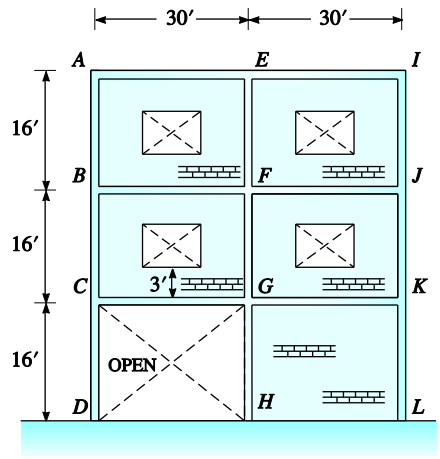
P2.28. Consider the building in Figure P2.28, which has a width into the page of 35 ft. Maximum inundation height, h_{max}, and flow velocity, u_{max}, have been determined as 33 ft and 20 ft/sec, respectively. Calculate the hydrodynamic load applied at floor level on the wall *IJKL* for Load Cases 2 and 3, due to outflow. If windows are inundated, calculate the expected hydrostatic loading on the adjacent outside walls due to water retained by the floor, or floors. Finally, calculate the debris impact load to be applied to the free-standing column CD. Assume I_{tsu} = 1.0 and C_d = 1.25.



P2.28

Load Case 2:

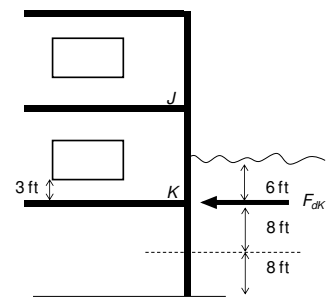
$$\frac{2}{3}h_{max} = h_{des} = 22 \text{ ft}$$

$$u_{max} = u_{des} = 20 \text{ ft/sec}$$

Load Case 3:

$$h_{max} = h_{des} = 33 \text{ ft}$$

$$\frac{1}{3}u_{max} = u_{des} = 6.67 \text{ ft/sec}$$



Load Case 2

Hydrodynamic, Load Case 2

$$h_{des,K} = \text{Trib height} = 8 + 6 = 14 \text{ ft}$$

$$F_{dK} = \frac{1}{2}\gamma_s (I_{tsu})(C_d)(C_{cx})(B)(h_{des,K})(20^2)$$

$$F_{dK} = \frac{1}{2}70.4(1.0)(1.25)(1.0)(35)(14)(20^2)$$

$$\boxed{F_{dK} = 8624 \text{ kips}}$$

Hydrodynamic, Load Case 3

$$h_{des,J} = \text{Trib height} = 1 + 8 = 9 \text{ ft}$$

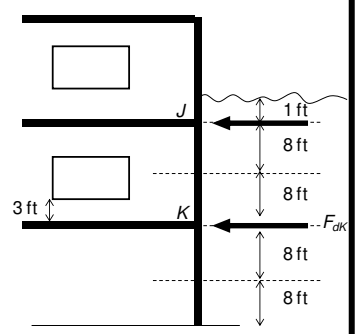
$$F_{dJ} = \frac{1}{2}70.4(1.0)(1.25)(1.0)(35)(9)(6.67^2)$$

$$\boxed{F_{dJ} = 616 \text{ kips}}$$

$$h_{des,K} = \text{Trib height} = 8 + 8 = 16 \text{ ft}$$

$$F_{dK} = \frac{1}{2}70.4(1.0)(1.25)(1.0)(35)(16)(6.67^2)$$

$$\boxed{F_{dK} = 1090.2 \text{ kips}}$$



Load Case 3

Hydrostatic on interior walls

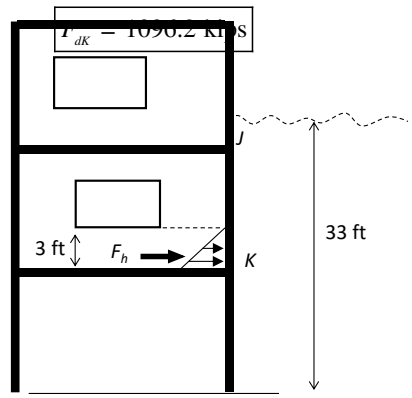
$$F_h = \frac{1}{2}\gamma_s b h_{des}^2 = \frac{1}{2}70.4(35)3^2$$

$$\boxed{F_h = 11.1 \text{ kips}}$$

Debris Impact on *CD*

$$F_i = 330 C_o I_{tsu} = 330(0.65)(1.0)$$

$$\boxed{F_h = 214.5 \text{ kips}}$$



Hydrostatic on inside walls

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