Solution Manual for Fundamentals of Structural Analysis 5th Edition by Leet Uang Lanning

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FUNDAMENTALS OF STRUCTURAL ANALYSIS

5th Edition

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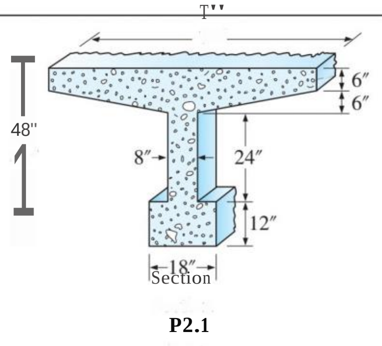
SOLUTIONS MANUAL

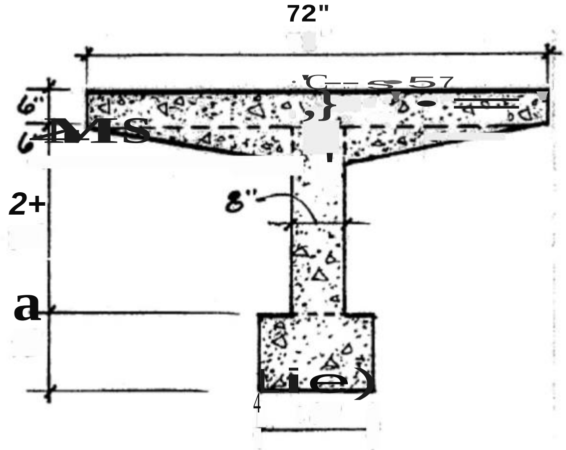
CHAPTER 2: DESIGN LOADS AND STRUCTURAL FRAMING

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 lbs/ft.^{3}



P2.1

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



Compute cross sectional area:

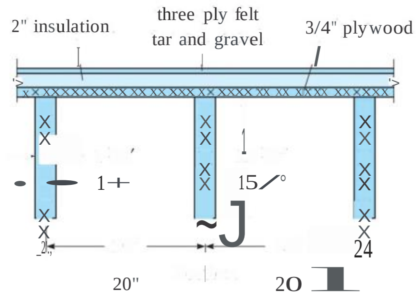
Area = (0.5' × 6') + 2[½ × 0.5' × 2.67'] + (0.67' × 2.5') + (1.5' × 1')

= 7.5 ft^2

Weight of member per foot length:

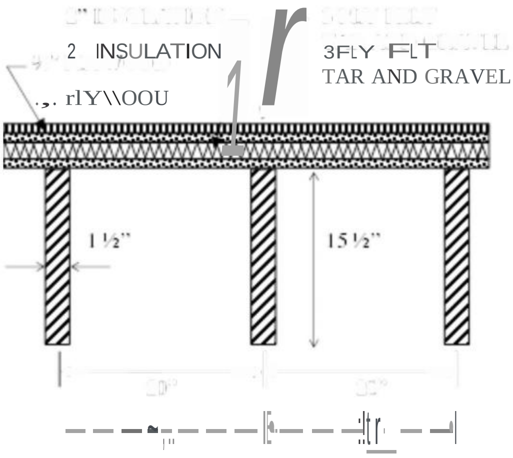
wt/ft = 7.5 ft^2 × 120 lb/ft^3 = 900 lb/ft.

P2.2. Determine the deadweight of a 1-ft-long segment of a typical 20-in-wide unit of a roof supported on a nominal 2 x 16 in. southern pine beam (the actual dimensions are ½ in. smaller). The ¾-in. plywood weighs 3 lb/ft.



Section

P2.2



See Table 2.1 for weights

wt/20" unit

Plywood: $3\text{ psf} \times \frac{20''}{12} \times 1' = 5\text{ lb}$

Insulation: $3\text{ psf} \times \frac{20''}{12} \times 1' = 5\text{ lb}$

Roof'g Tar &G: $5.5\text{ psf} \times \frac{20''}{12} \times 1' = 9.17\text{ lb}$

= 19.17 lb

Wood Joist = $37 \frac{1}{ft} \frac{(1.5'' \times 15.5)}{144\text{ in}/f} 1' = 5.97$

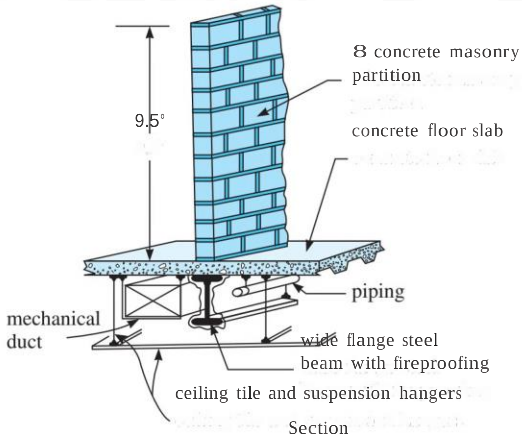
Total wt of 20" unit = 19.17 + 5.97

= [25.14 lb. Ans.]

P2.3. A wide flange steel beam shown in Figure P2.3 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.



P2.3

Uniform Dead Load W Acting on the Wide Flange Beam:

Wall Load:

9.5'(0.09 ksf) = 0.855 kif

Floor Slab:

10'(0.05 ksf) = 0.50 klf

Steel Frmg, Fireproof'g, Arch'l Features, Floor Finishes, & Ceiling:

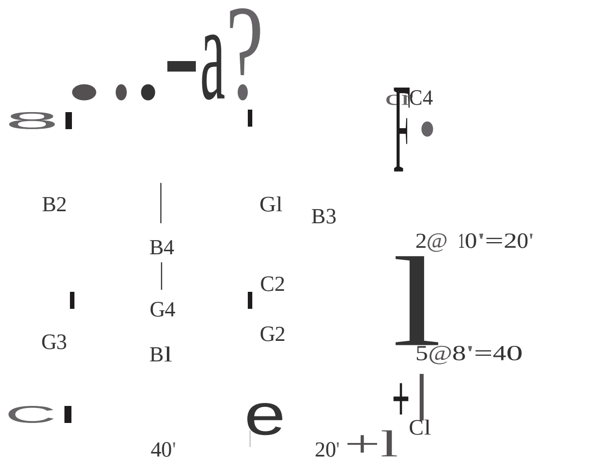
10'(0.024 ksf) = 0.24 klf

Mech'l, Piping & Electrical Systems:

10'(0.006 ksf) = 0.06 kf

Total W_u = 1.66 kf

P2.4. Consider the floor plan shown in Figure P2.4. Compute the tributary areas for *(a)* floor beam B1, *(b)* floor beam B2, (c) girder G1, *(d)* girder G2, *(e)* comer column C1, and *(J)* interior column C



P2.4

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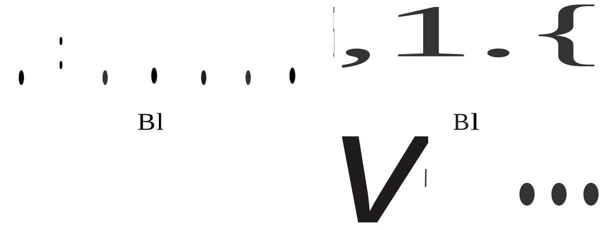
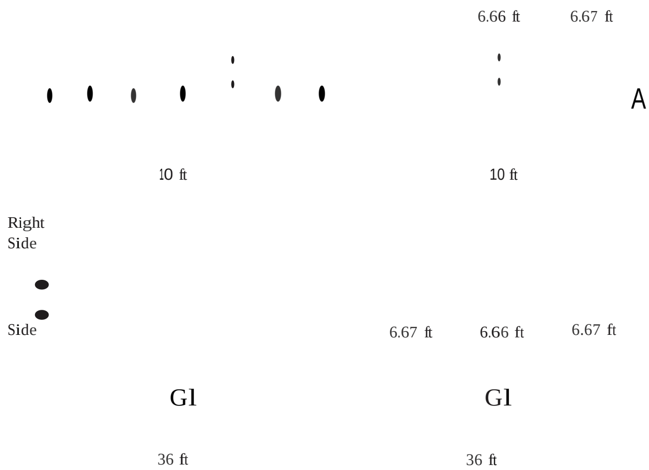


6 7

(b) Method 1: A_T= (·:· } 20) = |A_T = 66.7 ft' |

Method2:A_T =320−4(;4(4))=1A_T =288ft'I

Method 2: A_T = 66.7 − 2 (; 3.33(3.33)) = |A_T = 55.6 ft' |



(c) Method 1: A_T = (⁓} 20) + 10(10)

[s,−166.7n]

«a−−[▾ao»)e2[,s»)]

[,−so.6n]

(d) Method 1:A_ = (40 , 20) [(36)

2 2

[s,−ioson]

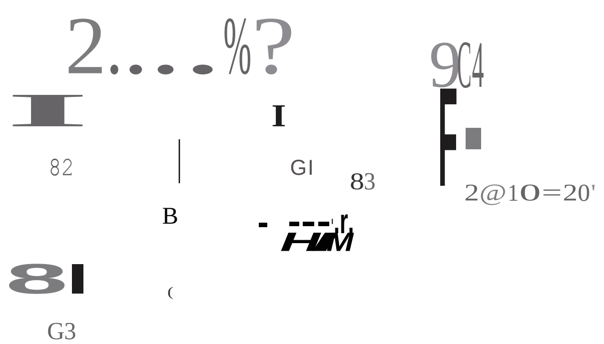
Method2:A_T =1080+2(;4(4))

[,−too6n]

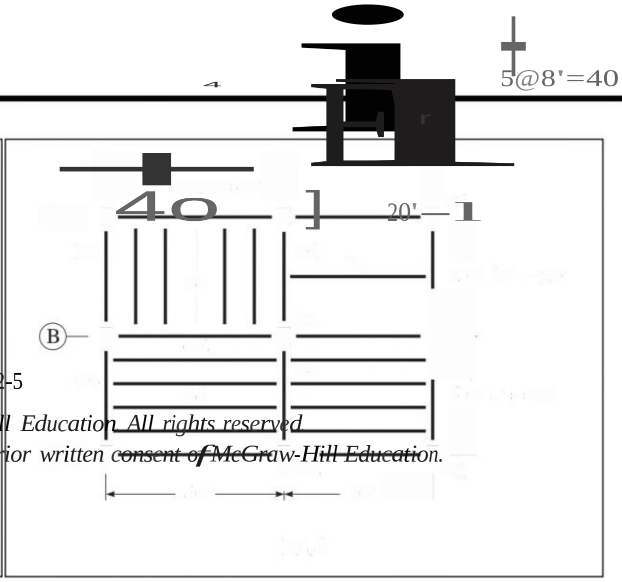
5 Z s»|

G2

G2



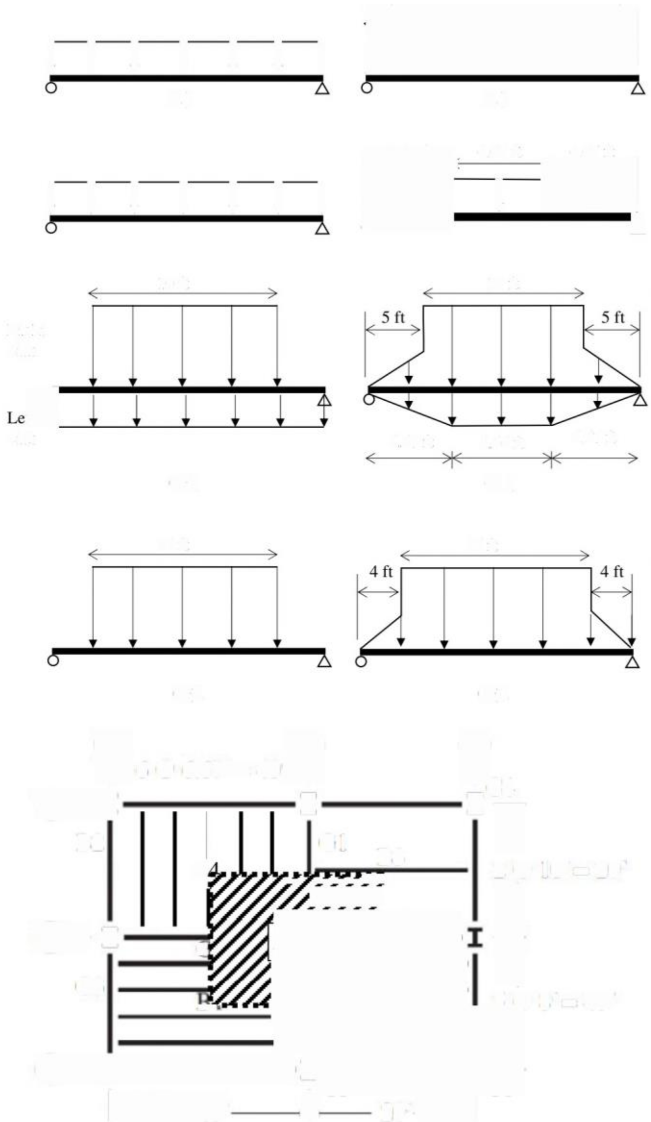
GM– 40/3 20/3 (40 20/3) 1 — 900n



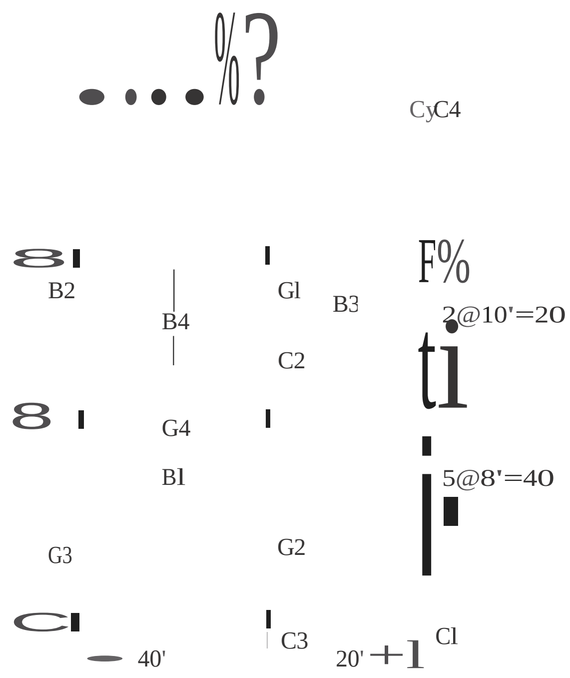
2: A_T 66.

$\left(\frac{20}{2}\right)$

2 2 2 A_T



P2.5. Refer to Figure P2.4 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 *(J)* corner column C4.



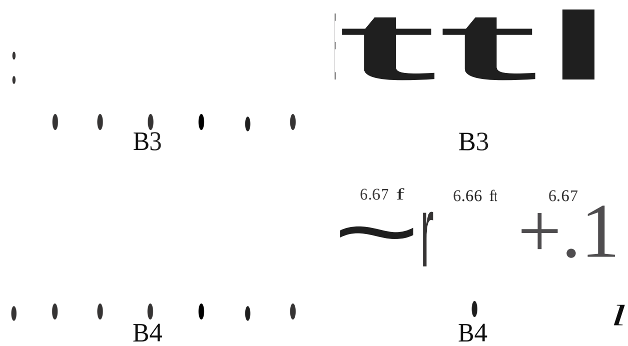
P2.4

(a) Method 1:A, = (10)(20)

[s,-zoo]

Method 2: A_T = 200−4(; 5')

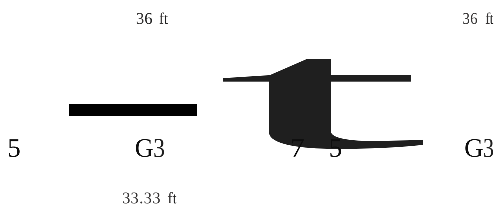
[s-so]



(6) Method 1:*A*, = (6.67)(20)=-[4, =133.4 f]

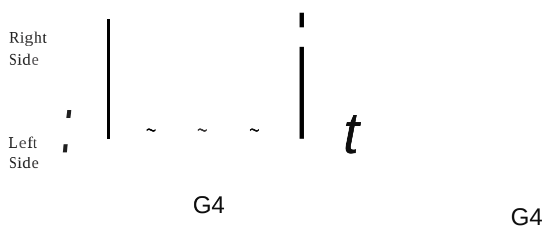
Method 2: A_T = 133.4 – 4 (; 3.33 ,

[-u]



«e» Memhod 1:4, – (36)(20)–[4, – 720 t]

Method2:A_T =720+2(;4')=1A_T =736ft'I

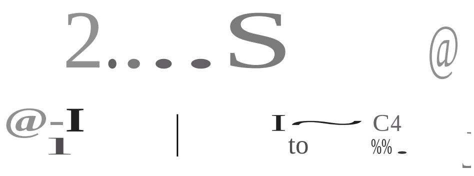


(d) Method 1:*A*, = (4)(40) + 33.33010)

[s,- «01.+n]

a – 0W-[r);r)

[s,- us«s«]

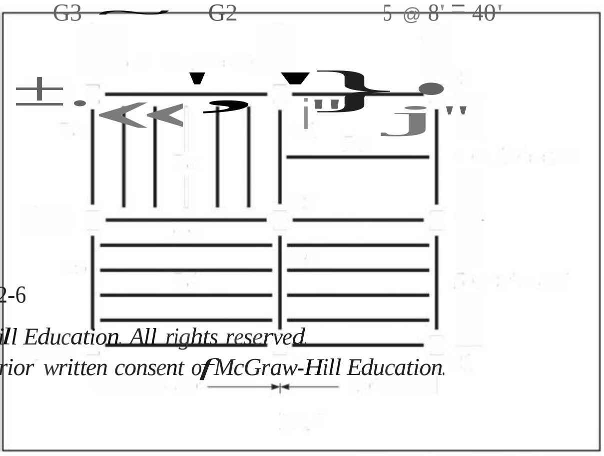


GA, -(0)(10) [4,=- too ne]

«)A, -(30)(20): [4,=-60on]

B4

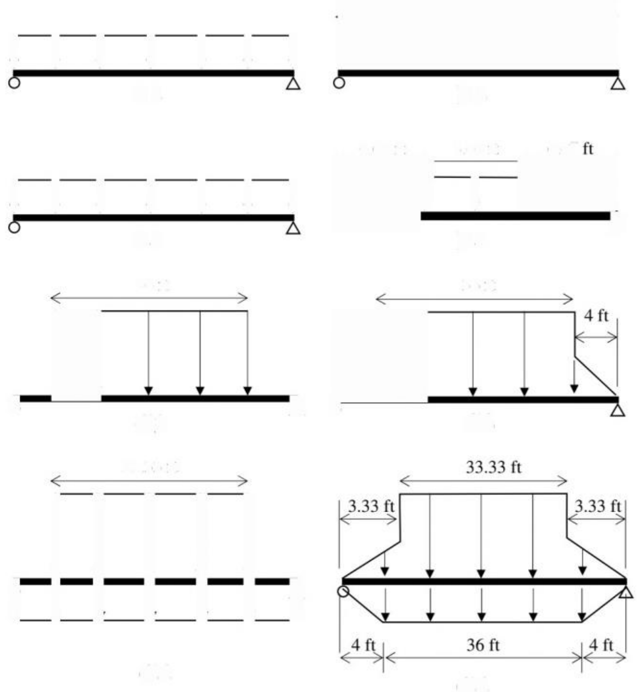
2@1O=20'



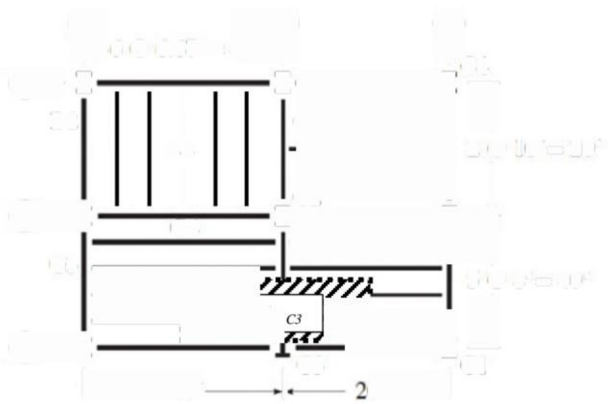
t

ft

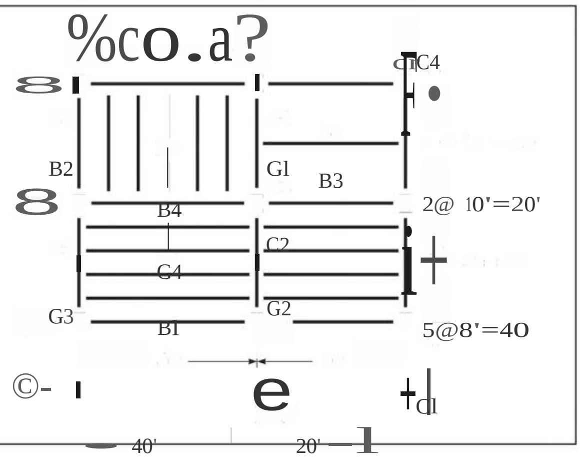
1.2



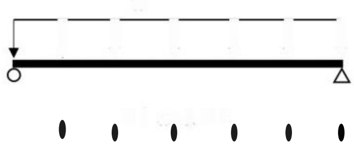
d 2: 2



P2.6. The uniformly distributed live load on the floor plan in Figure P2.4 is 60 lb/ft. Establish the 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.



P2.4



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

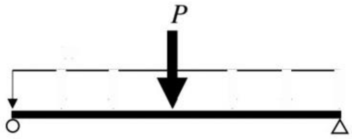
(a) $A_T = 8(40) = 320$ ft^2, $K_{LL} = 2$, $A_I = K_{LL}A_T = 640 > 400$

$w = 8(50.6) = 404.8$ lb/ft $= 0.40$ kips/ft

w
B1 and B2

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

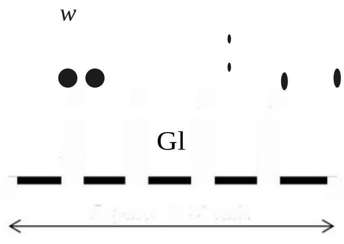
$w = \frac{6.67}{2}(60) = 200.1$ lb/ft $= 0.20$ kips/ft



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

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

$P = \frac{60(10)(20)}{2} = 6000$ lbs $= 6$ kips

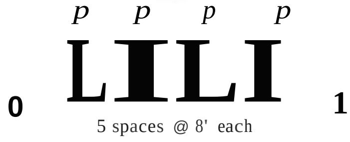


(d) $A_T = \left(\frac{40}{2} + \frac{20}{2}\right)36 = 1080$ ft^2, $K_{LL} = 2$, $A_I = K_{LL}A_T = 2160 > 400$

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

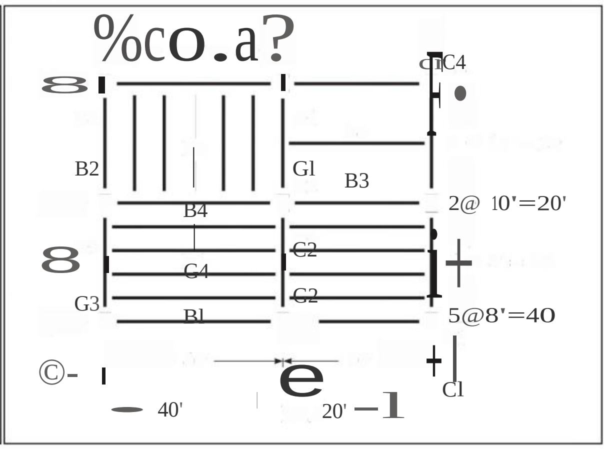
$L = 34.4$ psf

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



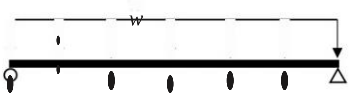
G2

P2.7. The uniformly distributed live load on the floor plan in Figure P2.4 is 60 lb/ft². Establish the 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.



P2.4

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



B3 and B4

$w = 10(60) = 600$ lb/ft $= 0.60$ kips/ft

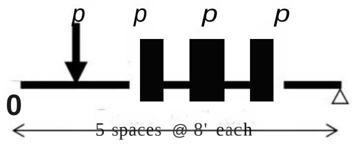
(b) $A_T = 6.67(20) = 133.4\ ft^2$, $K_{LL} = 2$, $A_T K_{LL} = 266.8 < 400$, No Reduction

$w = 6.67(60) = 400.2$ lb/ft $= 0.40$ kips/ft

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

$L = 60[0.25 + \frac{15}{\sqrt{1440}}] = 38.7$ psf $> \frac{60}{2}$

$P = \frac{W_{trib}}{2} = \frac{38.7(840)}{2} = 6192$ lbs $= 6.19$ kips



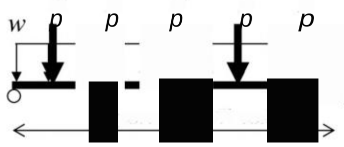
G3

(d) $A_T = (\frac{3}{4})40 + 33.33(10) = 493.3\ ft^2$, $K_{LL} = 2$, $A_T K_{LL} = 986.6 > 400$

$L = 60[0.25 + \frac{15}{\sqrt{986.6}}] = 43.7 > \frac{60}{2}$, ok

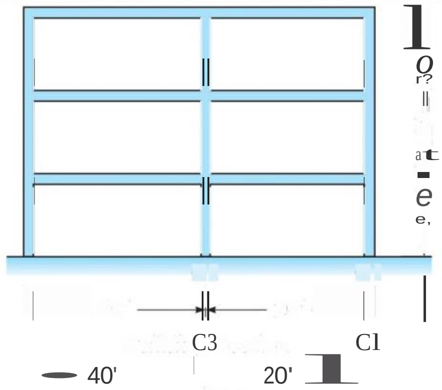
$w = 43.7(4) = 174.8$ lb/ft $= 0.17$ kips/ft

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



G4

P2.8. The building section associated with the floor plan in Figure P2.4 is shown in Figure P2.8. Assume a live load of 60 lb/ft2 on all three floors. Calculate the axial forces produced by the live load in column C1 in the third and first stories. Consider any live load reduction if permitted by the ASCE standard.



Building Section

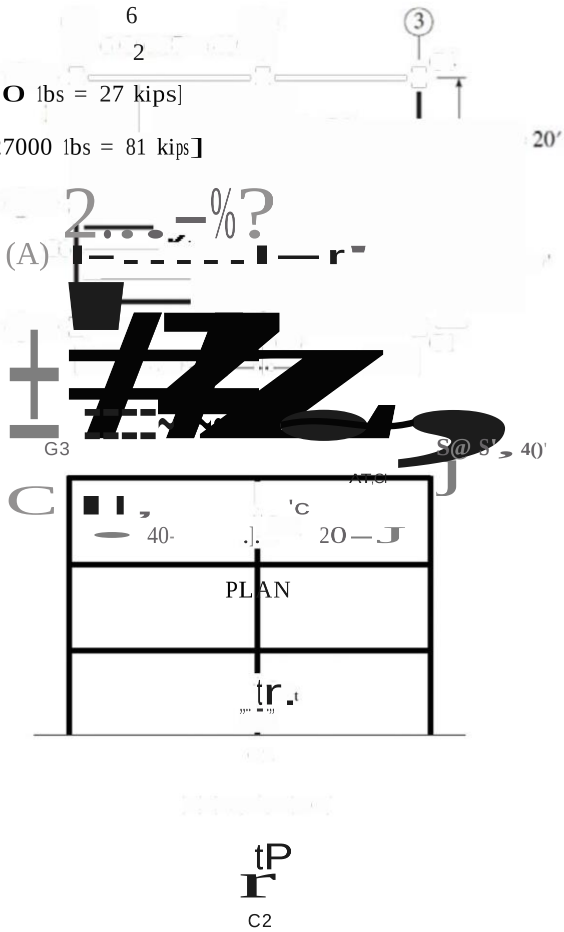
P2.8

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

$L = 60\left(0.25 + \frac{15}{\sqrt{3600}}\right) = 30 \text{ psf} = 0.5(60)$ ok (minimum permitted)

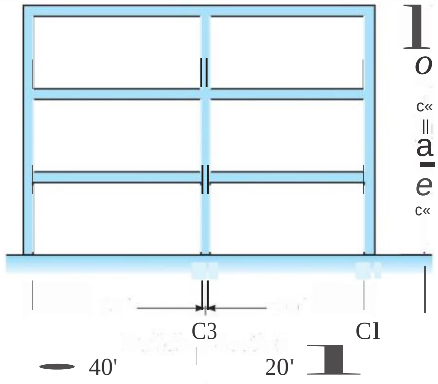
$P_3 = 900(30) = 27000 \text{ lbs} = 27 \text{ kips}$

$P_1 = (\;)900(30) = 27000 \text{ lbs} = 81 \text{ kips}$



ELEVATION

P2.9. The building section associated with the floor plan in Figure P2.4 is shown in Figure P2.8. Assume a live load of 60 lb/ft2 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.



Building Section

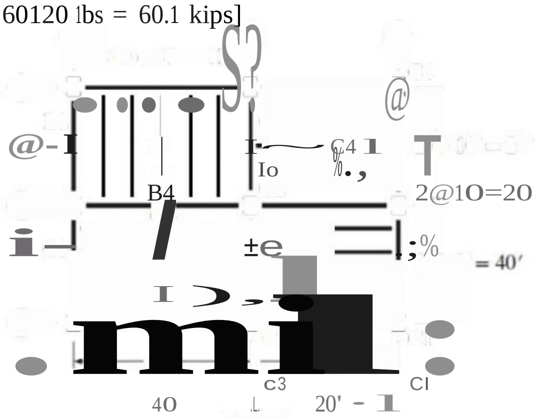
P2.8

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

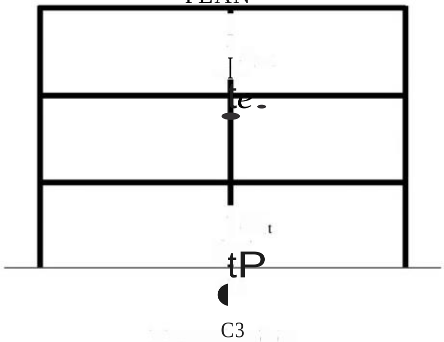
$$L = 60\left[0.25 + \frac{15}{\sqrt{2400}}\right] = 33.4 \text{ psf} > \frac{60}{2} \text{ ok}$$

$P_3 = 600(33.4) = 20040$ lbs = 20.0 kips

$P_1 = 3(600)(33.4) = 60120$ lbs = 60.1 kips

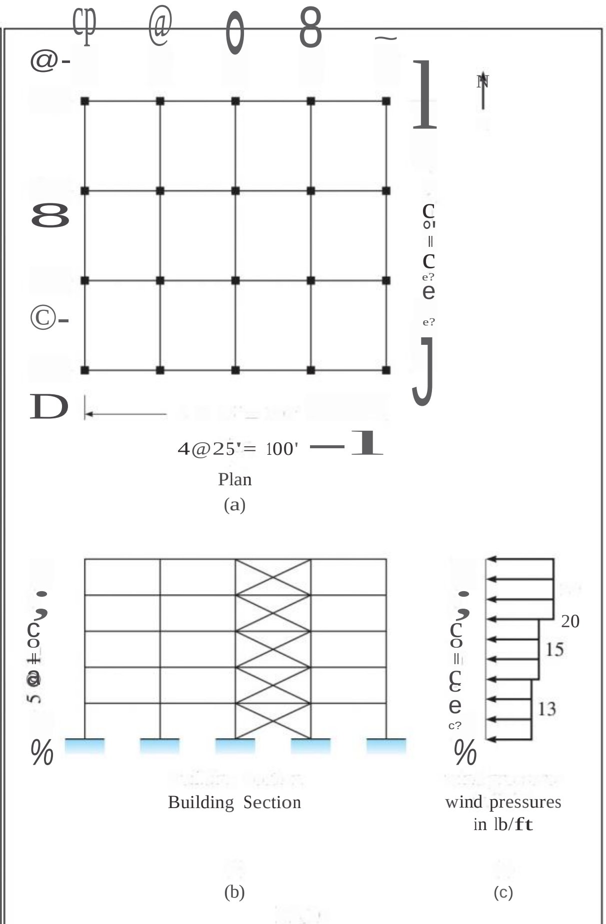


PLAN



ELEVATION

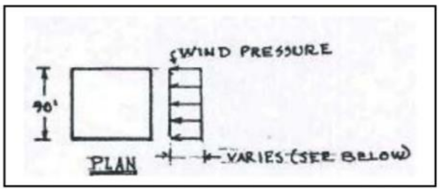
P2.10. A five-story building is shown in Figure P2.10. Following the ASCE standard, the wind pressure along the height on the windward side has been established as shown in Figure P2.10(c). *(a)* Considering the windward pressure in the east-west direction, use the tributary area concept to compute the resultant wind force at each floor level. *(b)* Compute the horizontal base shear and the overturning moment of the building.



P2.10

a) Resulant Wind Forces

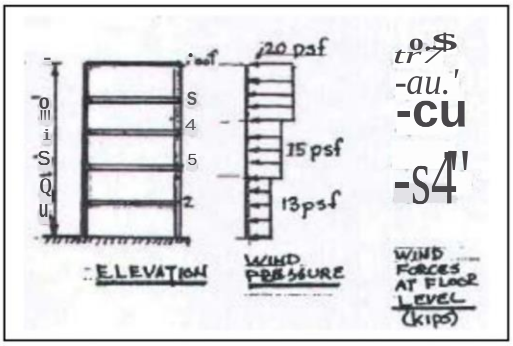
Roof	20 psf (6 x 90) = 10,800 lb
5" floor	20 psf (12 x 90) = 21,600 lb
4" floor	20 psf (2 x 90) + 15 (10 x 90) = 17,100 lb
3" floor	15 psf (10 x 90) + 13 (2 x 96) = 15,800 lb
2" floor	13 psf (12 x 90) = 14,040 lb



b) Horizontal Base Shear V~ = 2 Forces at Each Level =

10.8' + 21.6' + 17.1' + 15.8' + 14.04'=

(a)



...=[9.3@]

Overturning Moment of the Building =

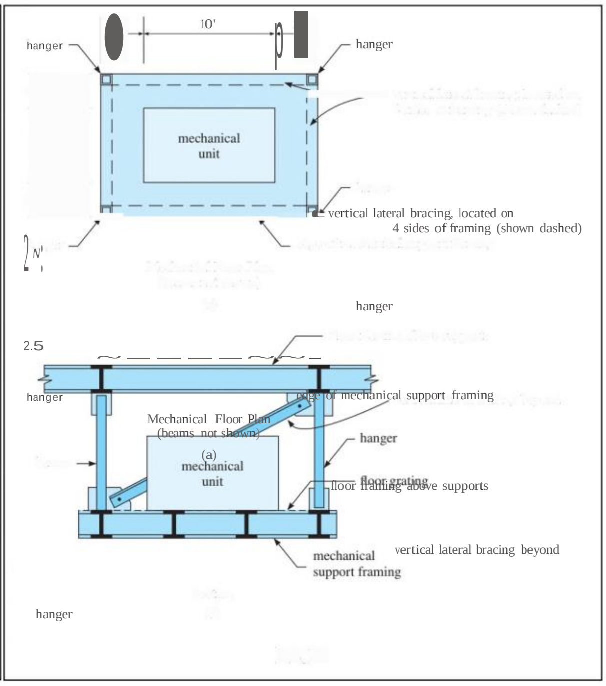
2(Force @ Ea. Level x Height above Base)

10.8' (60')+ 21.6 (48) + 17.1 (36) +

15.8\(24')+ 14.04'(12') =

M overturning = [2, 848''

P2.11. A mechanical support framing system is shown in Figure P2.11. 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 lbs, 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.



Section
(b)

P2.11

$= 4.8^k$

a) Live Load Impact Factor = 20%

b) Total LL

Machinery = 1.20 (4 kips)

Uniform LL = ((10' 16) − (5' 10'))(0.04ksf) = 4.4

Total LL = 9.2'

= 4

∴ Total' LL Acting on One Hanger = 9.2'/4 Hangers = [2.3]

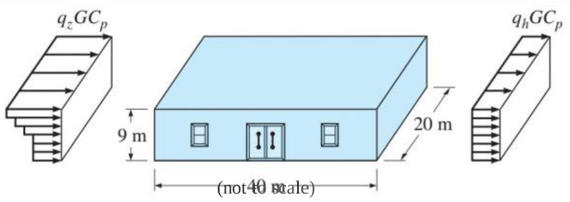
c) Total DL

Floor Framing = 10' x 16' (0.025 ksf)

∴ Total DL Acting on one Hanger = 4/4 Hangers = [1 kip]

∴ Total DL + LL on One Hanger = 2.3' + 1' = 3.3 kips

P2.12. The dimensions of a 9-m-high warehouse are shown in Figure P2.12. 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 mis, 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?



P2.12

Use I= I

q_s =0.613V (Eq. 2.4b)

–0.613@O – [9so.sMkl

•=, IKKK,

q_z=980.8((K,K(0.85) = [833.7K_z]

0–4.6m: q_z = 833.7(0.85) = 708.6 N/m

4.6–6.1m: q_z =833.7(0.90) = 750.3 N/m

6.1 =7.6 m: q_z =833.7(0.94) = 783.7 N/m

7.6 =9m: q_z =833.7(0.98) = 817.1 N/m

For the Windward Wall

$p = q_z GC_p$ (Eq. 2.7)

where GC_p = 0.85(0.8) = [0.68]

p =0.68 q_z

0–4.6m p=481.8N/m

4.6–6.1m p=510.2 N/m'

6.1–7.6m p=532.9 N/m

7.6--9m p=555.6 N/m

Total Windforce, F_W, Windward Wall

F_w = 481.8[4.6 20] + 510.2[1.5 x 20]

+ 532.9[1.5 x 20] + 555.6[1.4 x 20]

F_w =91,180 N

For Leeward Wall

$p = q_h GC_p = q_h$ (0.85)(–0.2)

$q_h = q_z$ at9m =817.1 Nm (above)

p = 817.1 (0.85)(–0.2) = 138.9 N/m

Total Windforce, F_L, on Leeward Wall

F_L =(20x9)(–138.9) =[--25,003N']

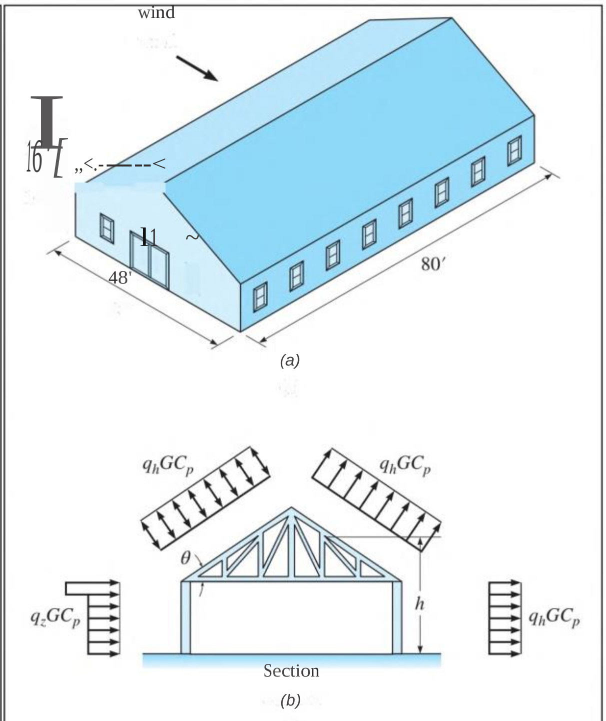
Total Force=$F_w + F_L$

= 91,180N + 25,003

= [116.183.3 N]

Both F_w and F_L Act in Same Direction.

P2.13. The dimensions of an enclosed gabled building are shown in Figure P2.13a. The external pressures for the wind load perpendicular to the ridge of the building are shown in Figure P2.13b. 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 P2.13, 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 pressure, 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.

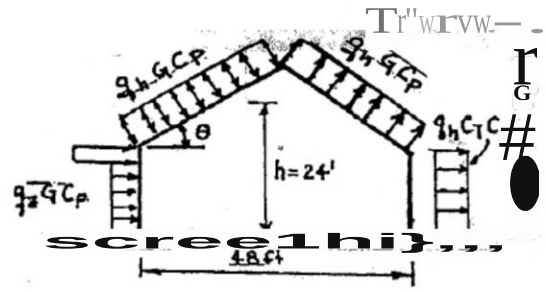


P2.13

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

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

Consider Positive Windward Pressure on Roof, i.e. left side.

Interpolate in Table P2.10

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

$C_p = 0.2738$ (Roof only)

From Table 2.4 (see p48 of text)

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

P2.13. Continued

$K_z = 1.0$, $K_d = 0.85$, $I = 1$

$q_z = 0.00256\, V$ (Eq 2.4a)

·, =0.002560oOy – [3sorrel

$\bullet = 0, IK_z K_{zt} K_d$

$0–15; q_z = 25.6(1)(0.57)(1)(0.85)$

$= 12.40$ lb/ft

$15–16'; q_z = 13.49$ lb/ft

$h = 24; q_z = 14.36$ lb/ft

Wind Pressure on Windward Wall & Roof

$P = q_z\, GC_p$

Wall 0–15' $P = 12.40 \times 0.85 \times 0.80$

$P = $ [8.43 psf]

Wall, 15'–16' $P = 13.49 \times 0.85 \times 0.8 =$ [9.17 psf]

Roof, $P = 14.36 \times 0.85 \times 0.2738$

$P = $ [3.34psf]

Wind Pressure on Leeward Side

For Wall $P = q_h\, GC_p$

For $h = 24'$; $q_z = q_h = 14.36$ lb/ft

$C_p = 0.2$ for wall = 0.6 for roof

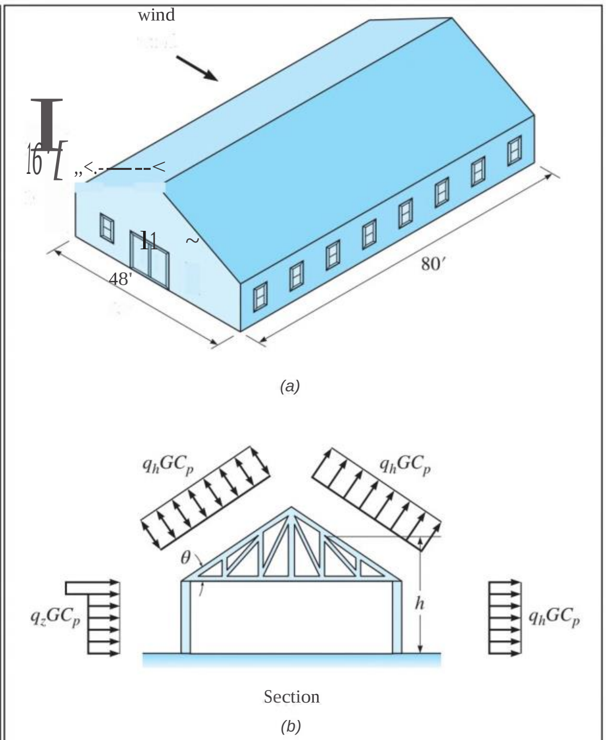
For Wall $P = 14.36\,(0.85)(0.2)$

$P = 2.44$ lb/ft

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

= [–7.32 1/f(plifp]

P2.14. Establish the wind pressures on the building in Problem P2.13 when the windward roof is subjected to an uplift wind force.



P2.13

TABLEP2.13 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

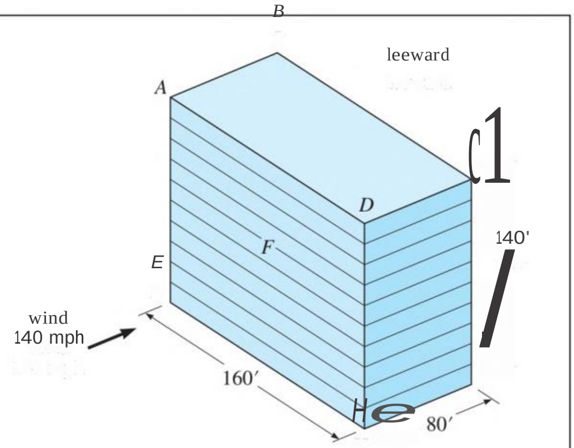
$C_p = -0.274$

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

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

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

P2.15. *(a)* Determine the wind pressure distribution on the four sides of the 10-story hospital shown in Figure P2.15. 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.



P2.15

(a) Compute Variation of Wind Pressure on Windward Face

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

$q_s = 0.00256V^2$ Eq 2.6a

$= 0.00256(140)^2$

$q_s = 50.176$ psf; Round to 50.18 psf

$I = 1.15$ for hospitals

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

K_z Read in Table 2.4

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

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

$q_z = 49.05K_z$

Compute Wind Pressure "p" on Windward Face

$p = q_z GC_p = 49.05\ K_z GC_p$

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

$C_p = 0.8$ on windward side

$p = 49.05\ K_z\ (0.85)(0.8) = [33.354K_z]$

Compute "p" for Various Elevations

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

Compute Wind Pressure on Leeward Wall

$p = qGC_p$; Use Value of q_z at 140 ft. i.e. $K_z = 1.52$

$C_p = -0.5$ $q_z = 49.05(1.52) = [74.556]$

$p = 74.556\ GC_p = 74.556(0.85)(-0.5)$

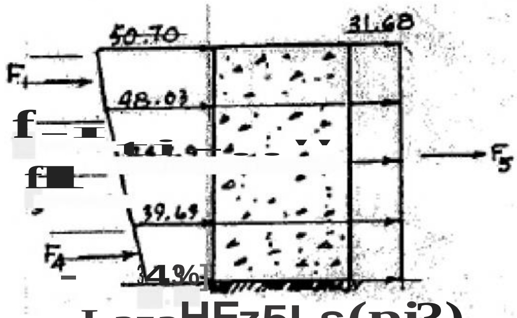
$p = [-31.68$ psf$]$ ANS.

Wind Pressure on Side Walls $p = qGC_p$

$= 49.05(1.52)(0.85)(-0.7)$

$p = [-44.36$ psf$]$

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



P2.15. Continued

Compute Total Wind Force (kips)

$$F_1 = \frac{50.7+48.02}{2}\left[\frac{35\times160}{1000}\right] = 276.42 \text{ kips}$$

$$F_2 = \frac{48.03+44.69}{2}\left[\frac{35\times160}{1000}\right] = 259.62 \text{ k}$$

$$F_3 = \frac{44.69+39.69}{2}\left[\frac{35\times160}{1000}\right] = 236.26 \text{ k}$$

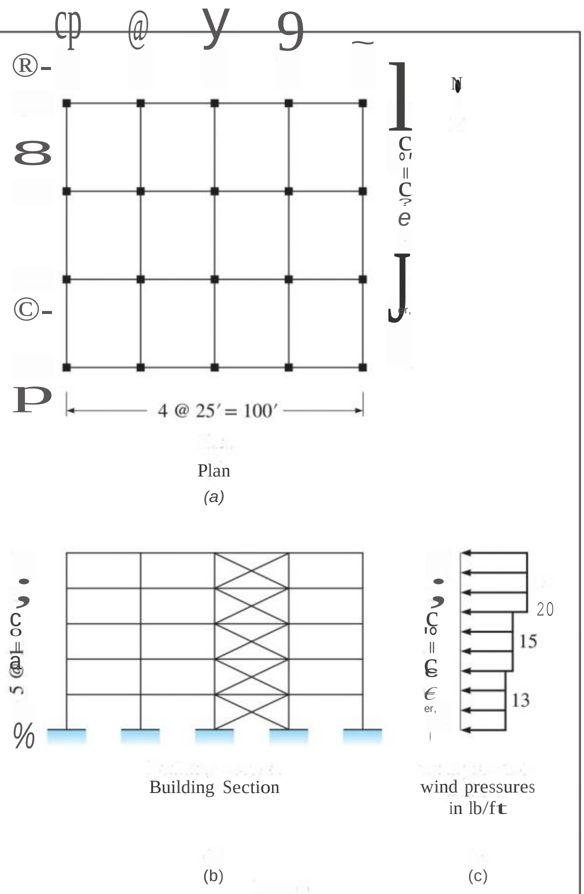
$$F_4 = \frac{39.69+34.36}{2}\left[\frac{35\times160}{1000}\right] = 207.39 \text{ k}$$

$$F_5 = \frac{31.68(140\times160)}{1000} = 709.63 \text{ k}$$

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

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

P2.16. Consider the five-story building shown in Figure P2.10. The average weights of the floor and roof are 90 lb/ft and 70 lb/ft, respectively. The values of S%, and S% are equal to 0.9g and 0.4g, 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.



P2.10

Fundamental Period

$T = C_t h^{3/4}$ [$C_t = 0.035$ for steel moment frames]

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

$T = [0.75 \text{ sec}]$

$W = 4(100 \times 90)\ 90\ \text{lb/ft}^2 + (100 \times 90)\ 70\ \text{lb/ft}$

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

$V = \frac{S_1 W}{T(R/I)}$ $I = 1$ for office bldgs.

$V = \frac{0.4(3870)}{0.75(8/1)} = [258\ \text{kips}]$

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

$= [435\ \text{kips}]$

$V_{min} = 0.044 S_{DS} W$

$= 0.044(1)(0.9)(3870)$

$= [153.6\ \text{kips}]$

Therefore, [Use $V = 258$ kips]

$= 1 + \frac{0.5 - 0.5}{2} = 1.125$

$F = \frac{W_x h_x^k}{\sum_{i=1} W_i h_i^k} V$

P2.16. Continued

Forces at Each Floor Level

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

P2.17. 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 P2.16. Which method produces a larger seismic base shear?

ASCE Approximate Fundamental Period:

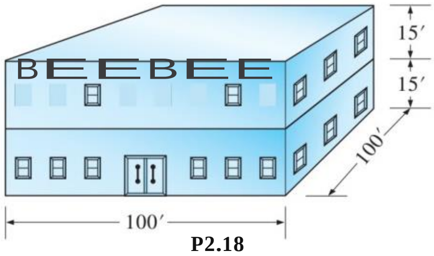
$T = 0.1N$

$N = 5 \quad \therefore T = 0.5$ seconds

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

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

P2.18. *(a)* A two-story hospital facility shown in Figure P2.18 is being designed in New York with a basic wind speed of 90 mi/h and wind exposure *D*. The importance factor *I* is 1.15 and K_t = 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 for both the floor and roof, is to be designed for the following seismic factors: S_s = 0.27g and S_1 = 0.06g; 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 building?



P2.18

(a) Wind Loads Using Simplified Procedure:

Design Wind Pressure $P_s = \lambda K_{zt} I P_{s30}$

λ = 1.66 Table 2.8, Mean Roof Height = 30'

Zones	P_{s30}	$P_s = 1.66(1)1.15 P_{s30} = 1.909 P_{s30}$
A	12.8 psf	24.44 psf
C	8.5 psf	16.22 psf

Resultant Force at Each Level; Where Distance a = 0.1(100') = 10'; 0.4(30') = 12'; 3'

a = 10' Controls & 2a = 20' Region(A)

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

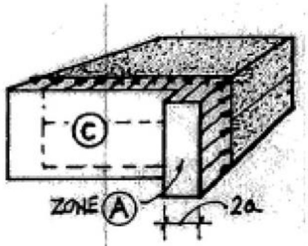
Zone (C): $\frac{15'}{2}$(16.3 psf) $\frac{80'}{1000}$ = 9.78'

$F_{Resultant}$ = 13.45'

F_2 Zone (A): 15'(24.44 psf) $\frac{20'}{1000}$ = 7.33

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

F_2 Resultant = 26.89'



Base Shear $V = F_{roof} + F_2$ = 40.34

Overturning Moment $M_{ot} = \Sigma Fh$

M_{ot} = 13.45'(30') + 26.89'(15') = 806.9

2-22

P2.18. Continued

(b) Seismic Loads by Equivalent Lateral Force Procedure Given: W = 90 psf Floor & Roof;

$S_s = 0.27g$, $S_1 = 0.06g$; $R = 8$, $I = 1.5$

Base Shear $V_{base} = \frac{S_{DS} W}{R/I}$

Where W Total Building Dead Load =

W_{2f} = 90 psf (100')^2 = 900'

W_{roof} = 90 psf (100')^2 = 900'

W_t = 1800'

And $T = C_t h_n^x$ = 0.342 sec.

C_t = 0.016 Reinf. Concrete Frame

X = 0.9 Reinf. Concrete Frame

h = 30' Building Height

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

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

$$V_{min} = 0.044 S_{DS} I W = 0.044(0.27)(1.5)(1800')$$
$$= 0.0178W = 32.1$$

Force @ Each Level $F_x = \frac{W_x h_x^k}{\Sigma W h^k} V_{base}$, Where V_{base} = 59.2'

T < 0.5 Sec. Thus K = 1.0

Level	W_x	H_x	$W_x h_x^k$	$W_x h_x^k / \Sigma W_i h_i^k$	Force @ Ea. Level:
Roof	900'	30'	27000	0.667	F_r = 39.5'
2°	900'	15'	13500	0.333	F_2 = 19.76
					$\Sigma F_x = V_{base} = 59.2$

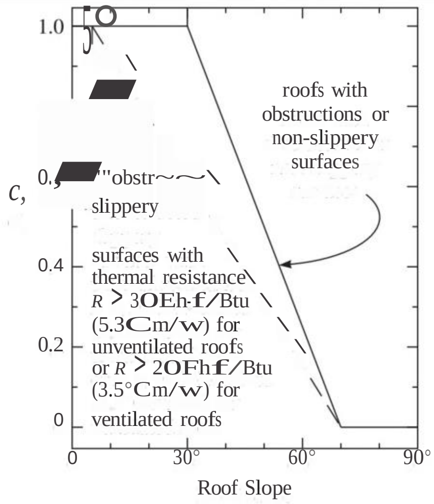
$\Sigma W_i h_i^k = 40500$

Overturning Moment $M_{OT} = \Sigma F_x h_x$

M_{OT} = 39.5'(30')+19.76'(15') = [1,481.4 ft]

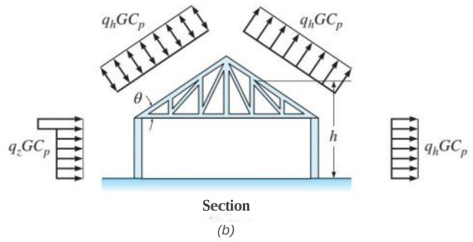
(c) Seismic Forces Govern the Lateral Strength Design.

P2.19. In the gabled roof structure shown in Figure P2.13, determine the sloped roof snow load P. 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 P2.19. 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 < 1.0

P2.19



P2.13

Sloped Roof Snow Load $P = C_s\, p_f$

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 $\theta = \mathrm{Tan}^{-1}\left(\frac{16}{24}\right) = 33.7°$

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

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

$P_s = C_s P_f = 0.9\,(19.6\text{psf}) =$ [17.64 psf]

Uniform Load Acting on Trusses Spaced @ 16'o.c.

$W_u = 17.64\text{psf}\,(16') =$ [282.2 plf]

P2.20. A beam that is part of a rigid frame has end moments and mid-span moments for dead, live, and earth-quake 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

Load Combinations-Factored Strength

End Moments

$1.4DL = 1.4(-180 ft.k) \qquad = 252 ft.k$

$1.2DL + 1.6LL + 0.5$ —— $= 1.2(-180) + 1.6(-150) \qquad = -456 ft\text{-}k'$

$1.2DL \pm LOE + LL + 0.2$(S $= 12(--180) + (-80) + (-150) \qquad = 446 ft.k$

Mid-Span Moments

$1.4DL = 14(+90 ft.k) \qquad = +126 ft.k$

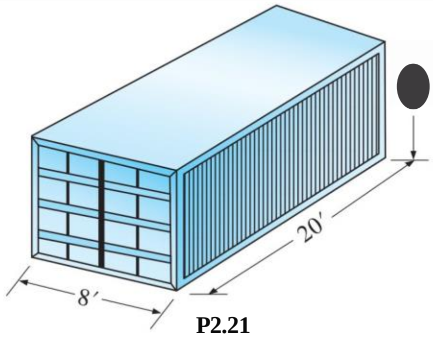
$1.2DL + 1.6LL + 0.5(L, ors) = 1.2(+90) + 1.6(+150) \qquad = +348 ft.k$

$1.2DL \pm 1.OE + LL + 0.2$(f $= 12(90) + 0 + (150) \qquad = +258 ft \cdot k$

Beam Needs to be Designed for Max. End Moment= –456 ft·k

Max. Mid-Span Moment = +348 ftk

P2.21. Calculate the vertical hydrostatic load on the 5100-lb empty shipping container in Figure P2.19 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?



P2.21

$F_v = \gamma_s V = 70.4(3)(8)(20)$

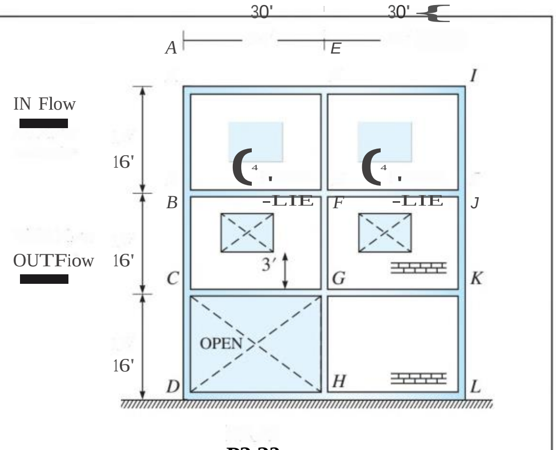
$F_v = 33792$ lbs

[$F_v = 33.8$ kips]

33.8 kips > $W_t = 5.1$ kips

[Yes, the container will be carried away.]

P2.22. Consider the building in Figure P2.22, 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 and hydrostatic resultant load and location on the walls *ABC and IJKL* for Load Cases 2 and 3, due to both inflow and outflow directions. 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.



P2.22

Load Case 2:

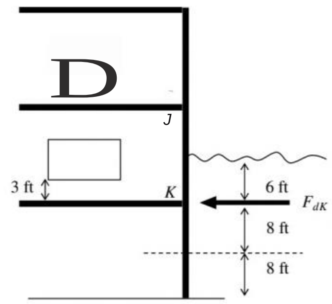
$$\frac{2}{3}h_{max} = h_{des} = 22 \text{ ft}$$

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

Load Case 3:

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

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



Hydrodynamic, Load Case 2

h = Trib height = 8 + 6 = 14 ft

E=, 1 - KC,KC,MB«Gh,,,O20

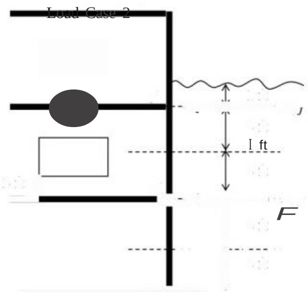
$$F = \frac{1}{2}70.4(1.0)(1.25)(1.0)(35)(14)(20)$$

Hydrodynamic, Load Case 3

h_{dJ} = Trib height = 1 + 8 = 9 ft

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

[.-a«o.»

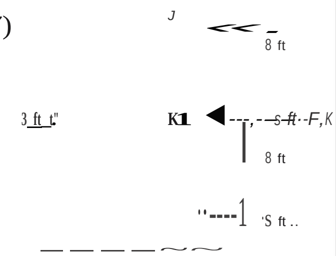


Z. -»a.

h = Trib height = 8 + 8 = 16 ft

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

[— 1096.2 kips]



Load Case 3

Hydrostatic on interior walls

$$F = \frac{1}{2}\gamma b h^2 = \frac{1}{2}70.4(35)3^2$$

[r,=1 - 1 ops]

Debris Impact on CD

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

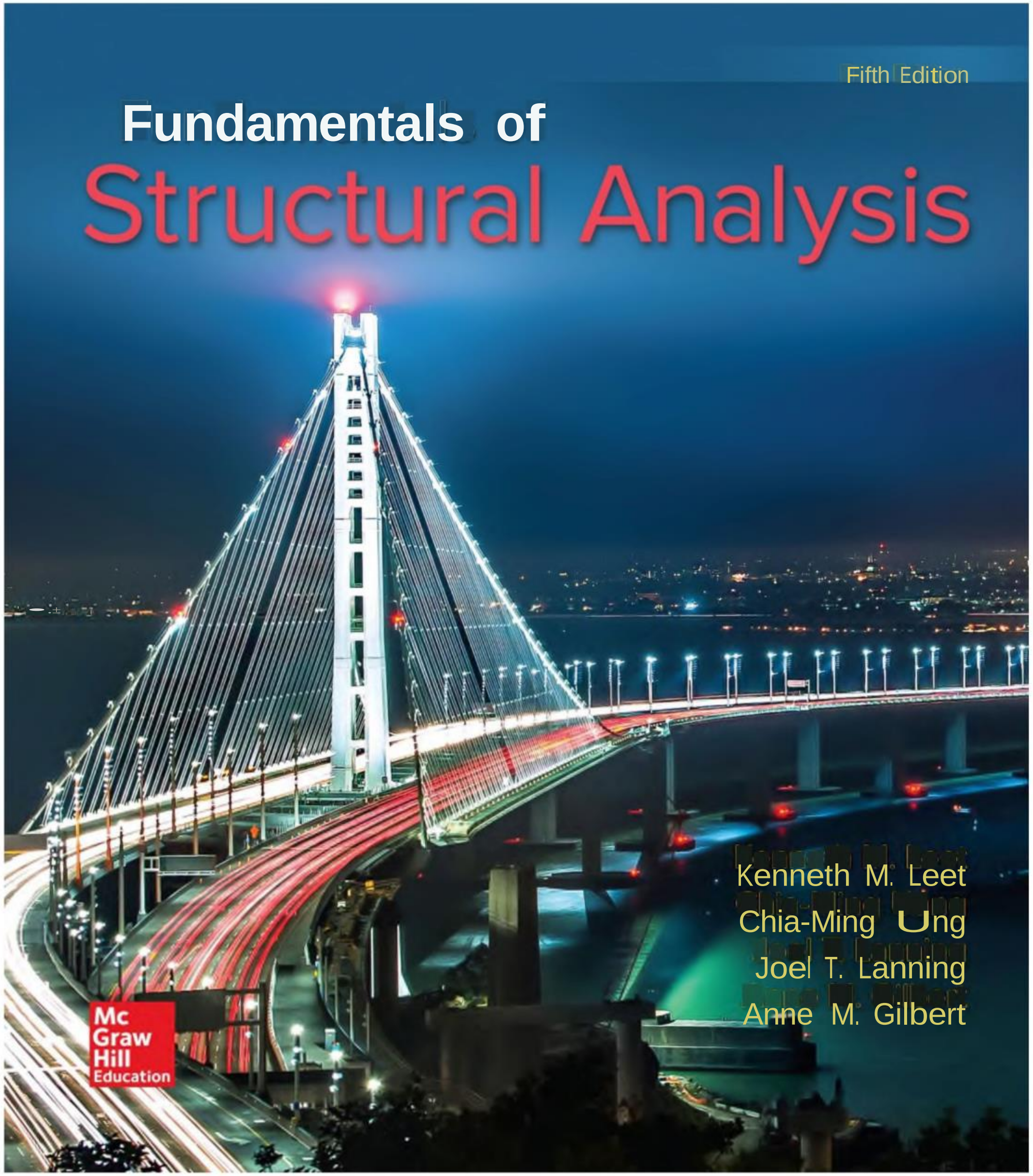
[F_i = 214.5 kips]

33 ft

Hydrostatic on inside walls

Design Loads and Structural Framing

Chapter 2



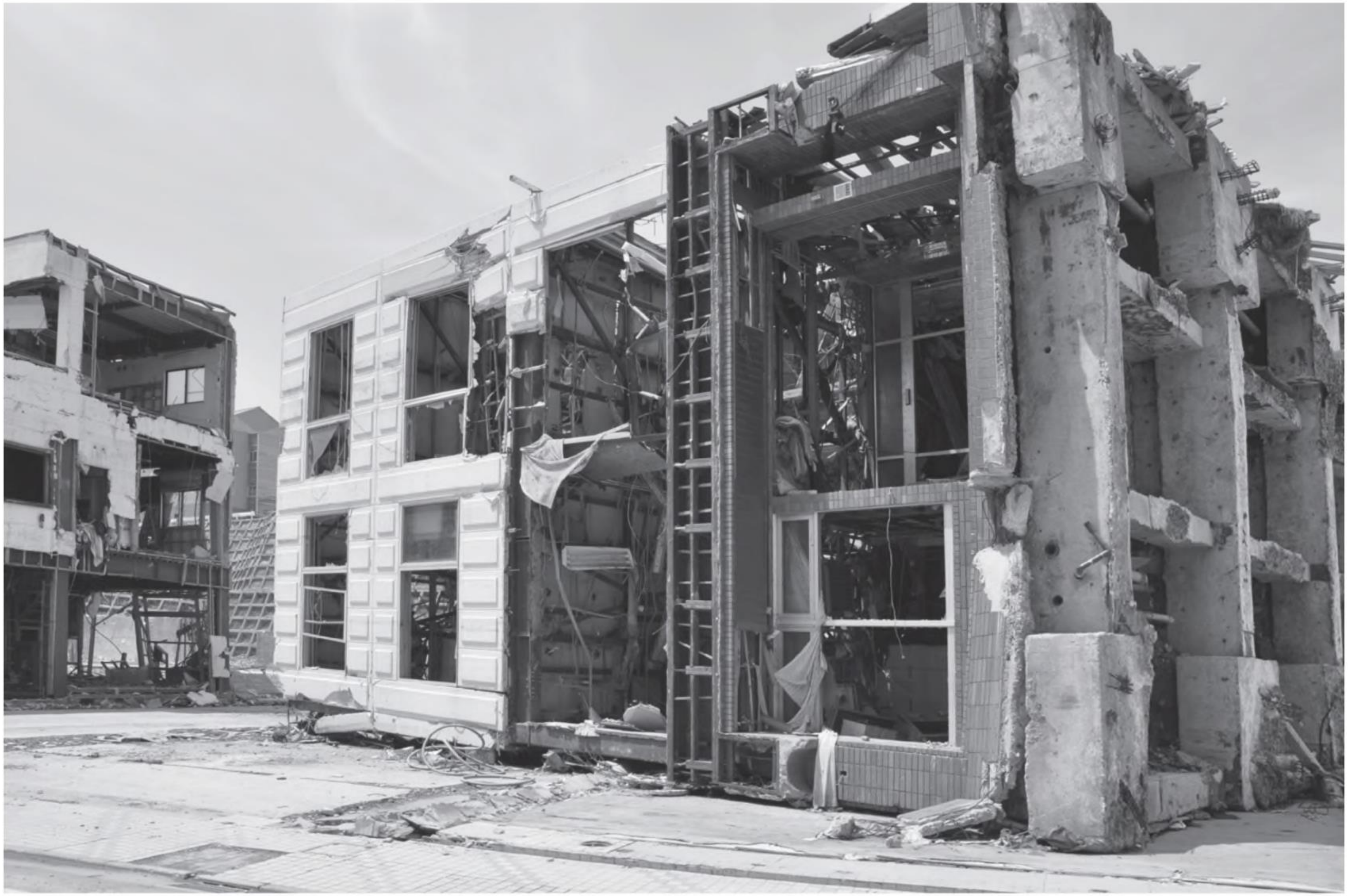


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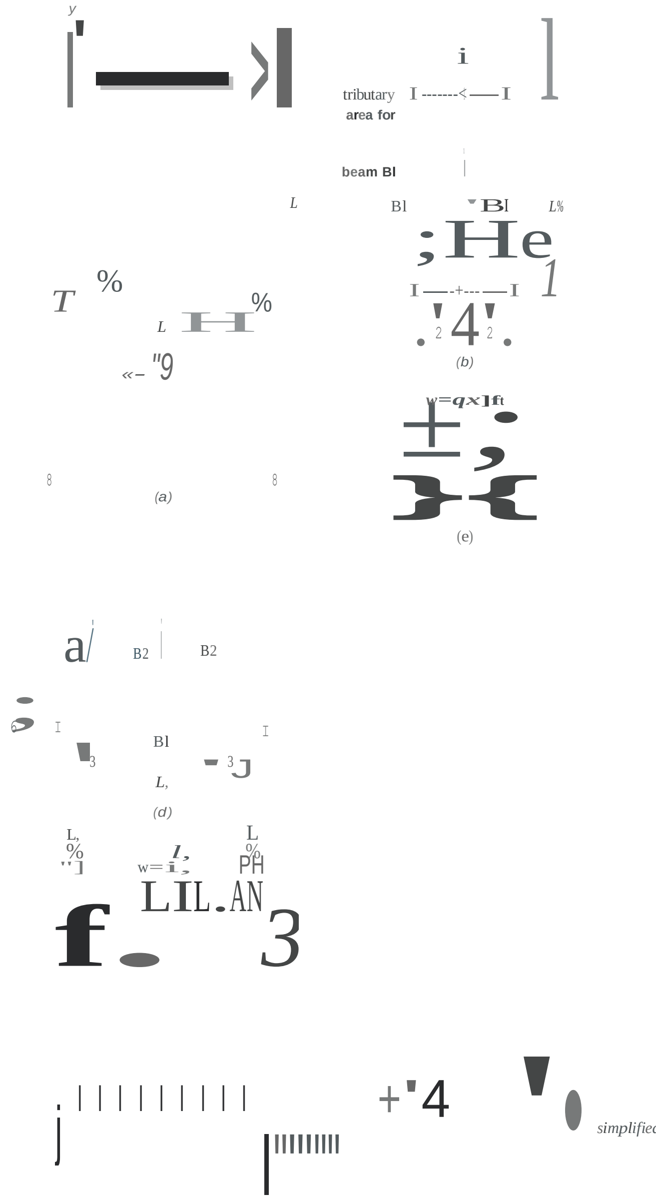
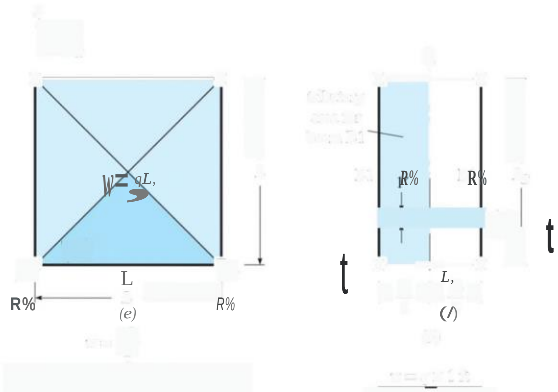
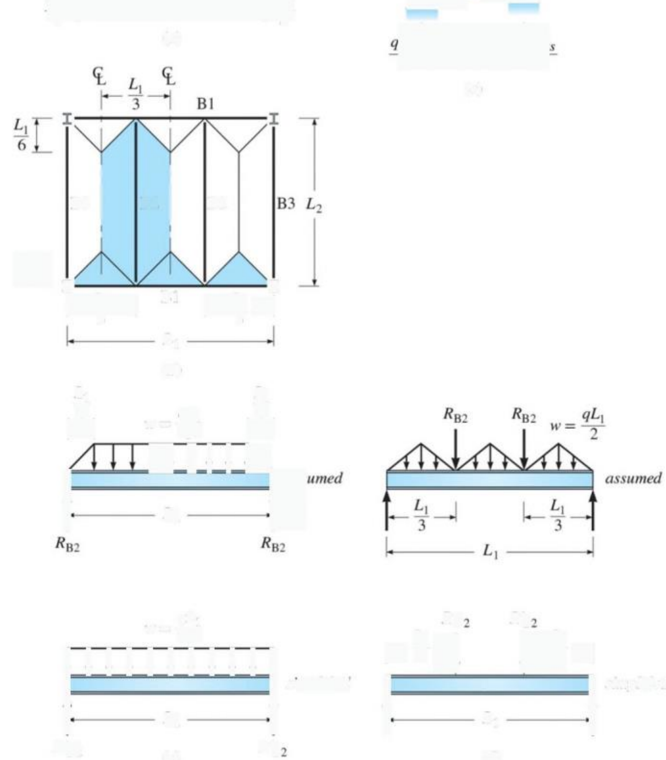
Figure 2.1





L
(e)
L,



B1
B3
umed
assumed

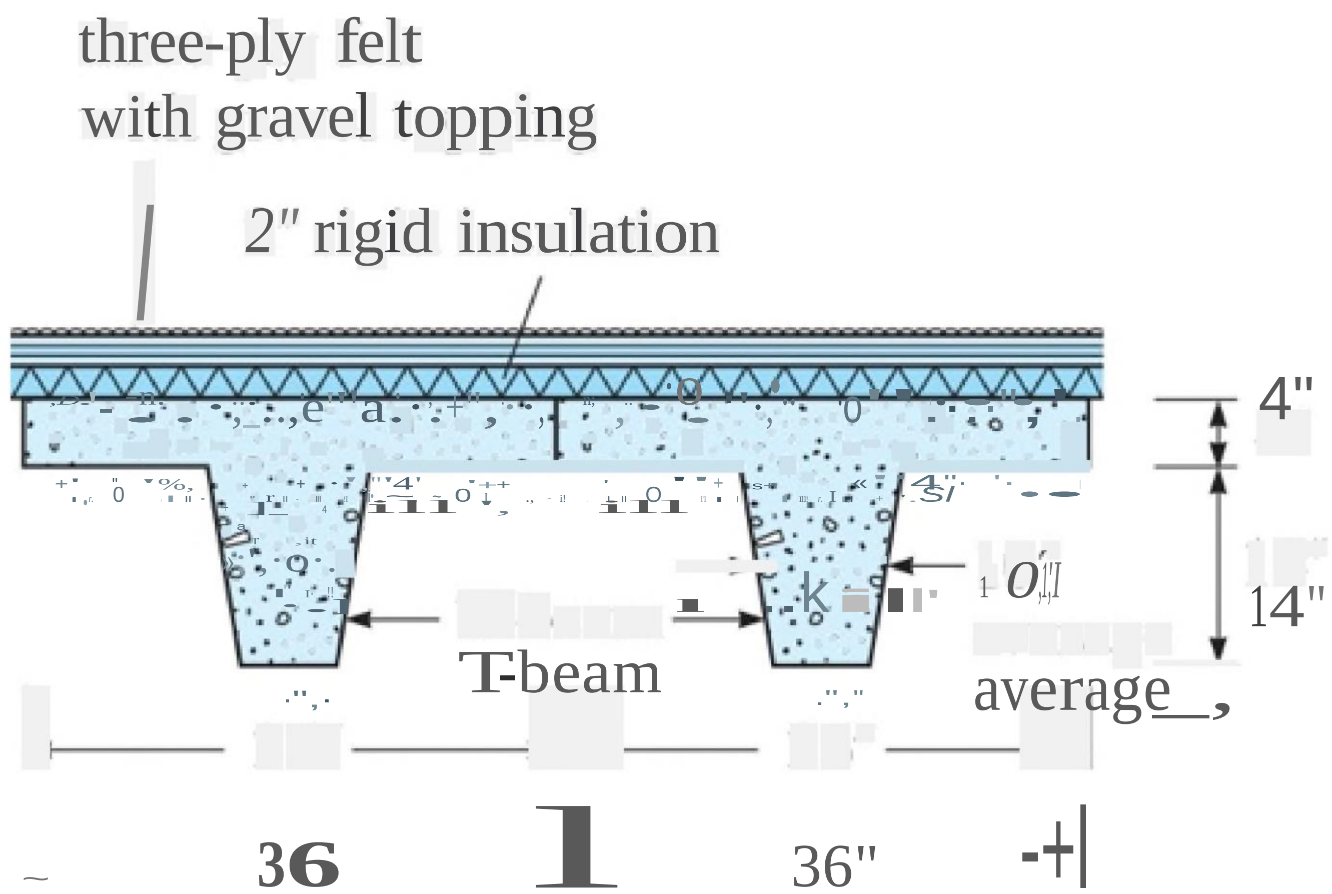
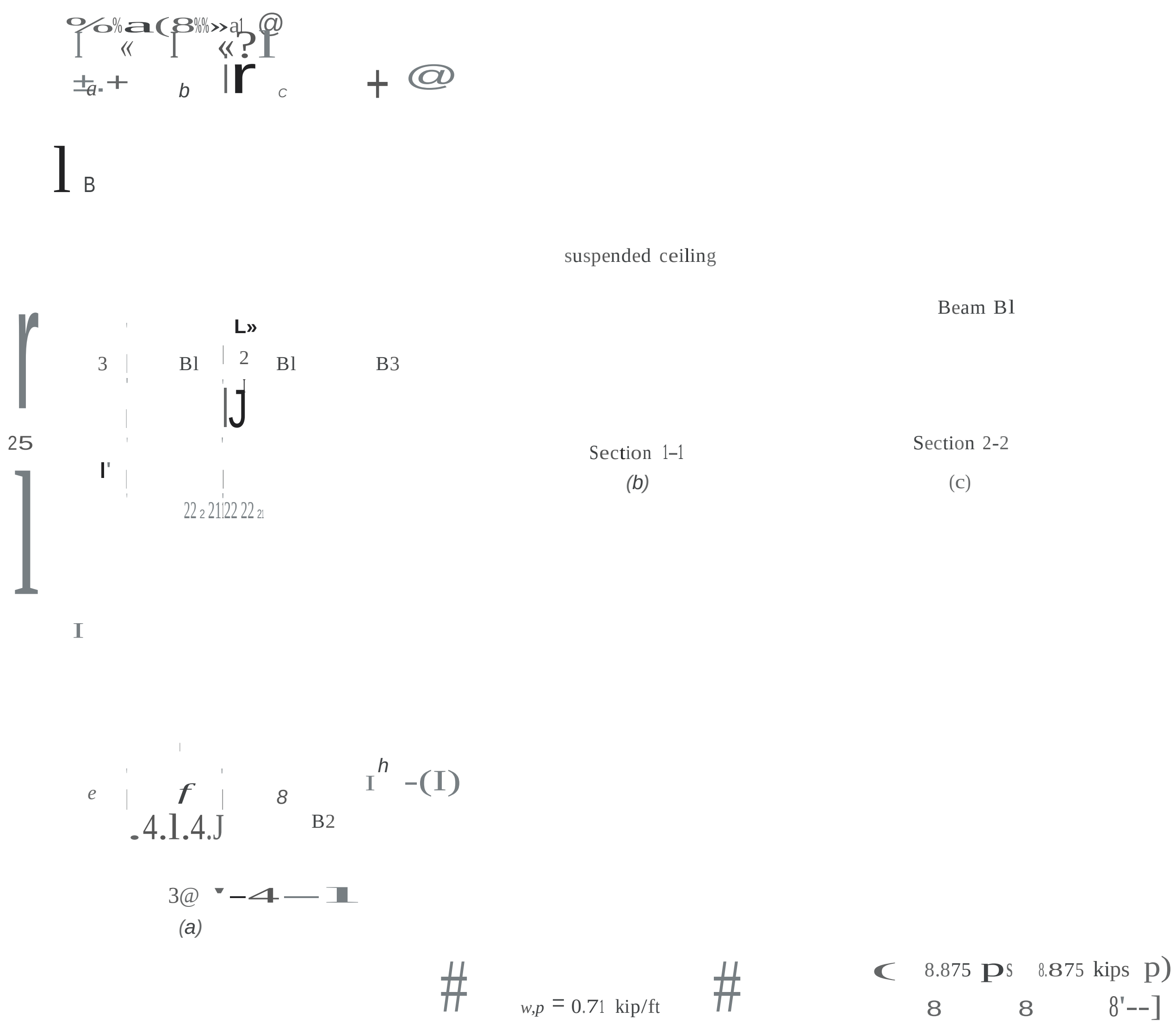

three-ply felt
with gravel topping
2" rigid insulation
4"
14"
T-beam
average
36
36"

Figure 2.3



(a)

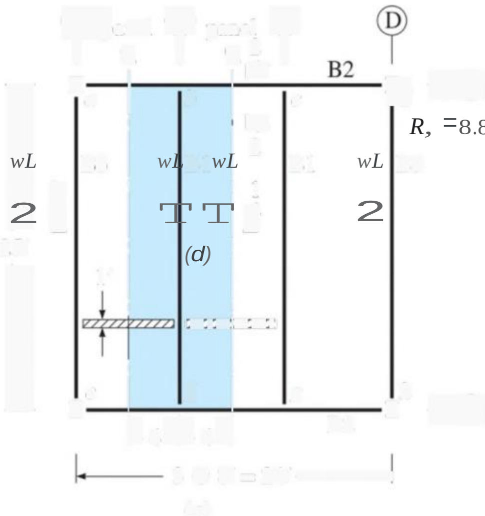
(b)

(c)

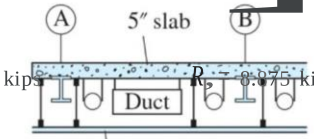
5±/5]

wpz([)()kip/ft

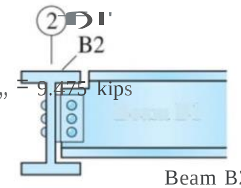
!=== == == == == == == ==!---0)



(d)

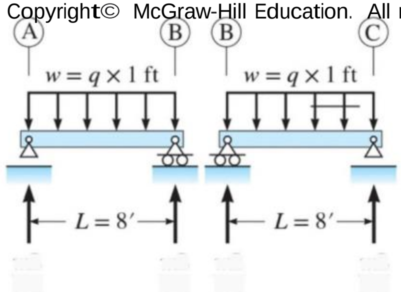


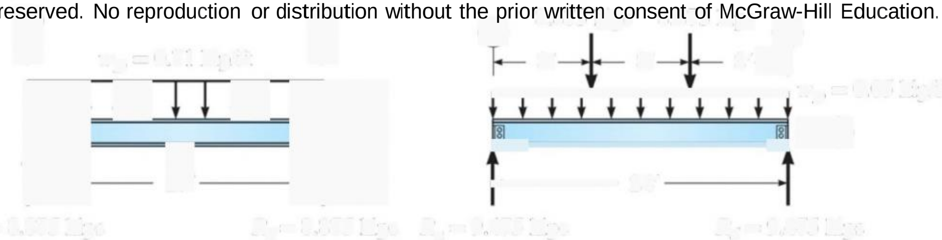
(e)



(f)

$R_,$ =8.875 kips $R_,$ = 8.875 kips $R_{,,}$ = 9.475 kips $R_,$ = 9.475 kips





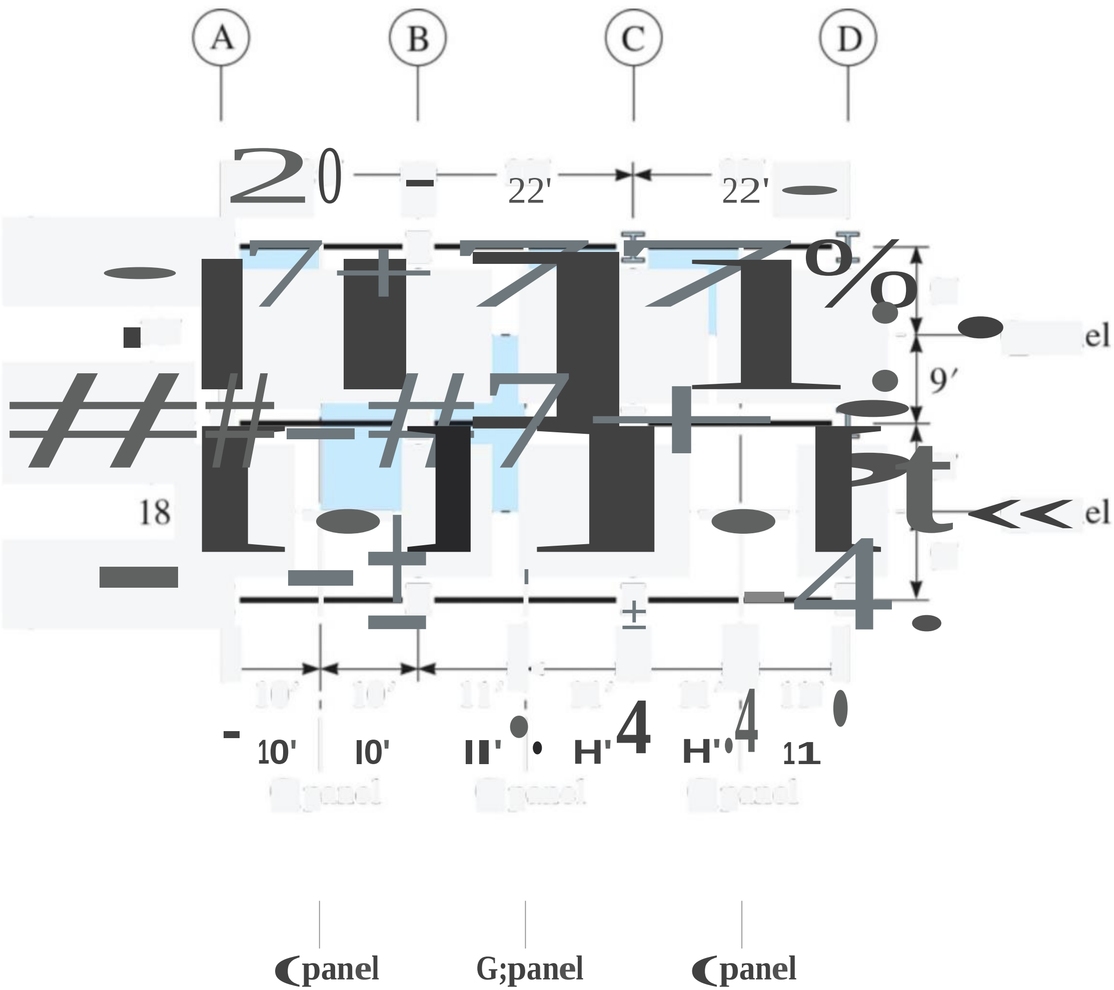
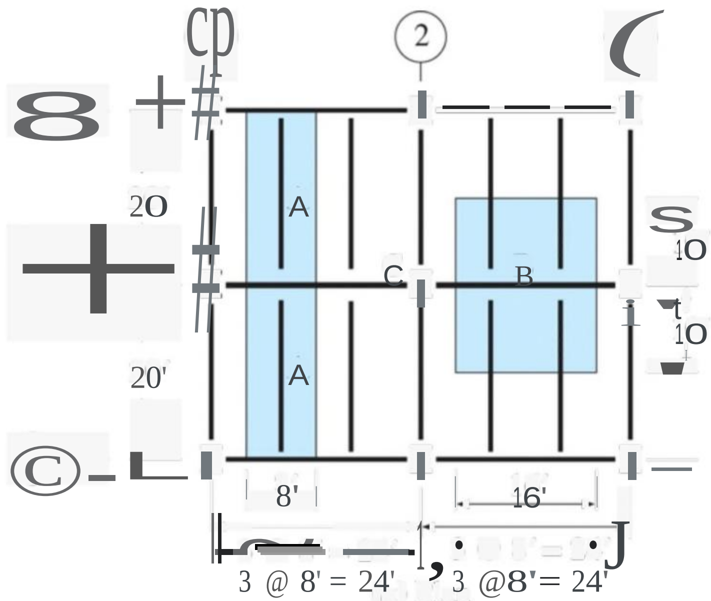
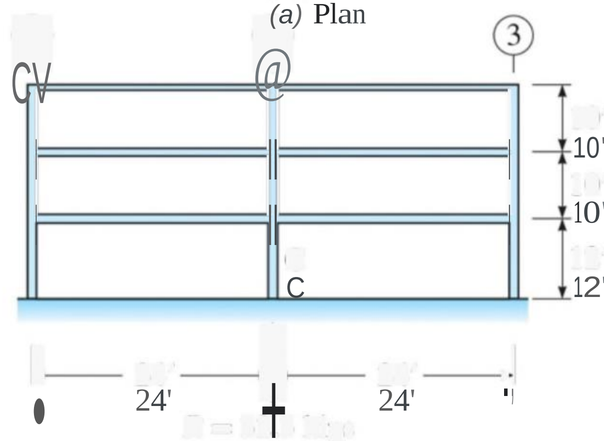

A
B
C
D
22'
22'
9′
18
el
el
10'
panel
panel

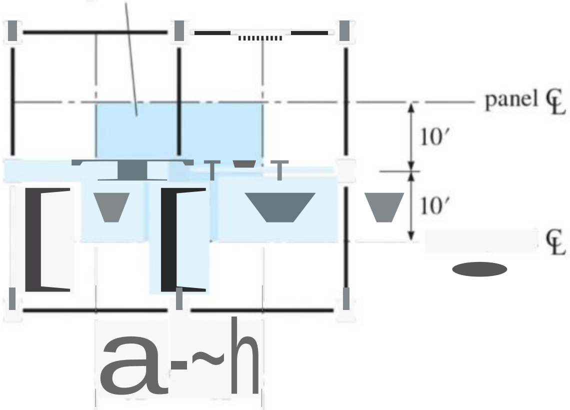
Figure 2.5



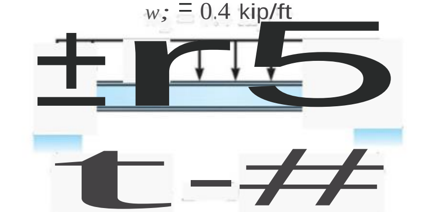
(a) Plan



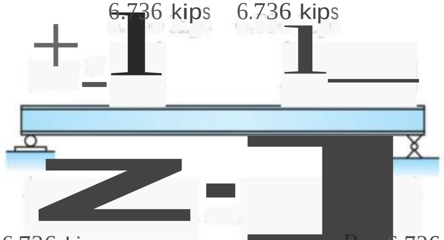
(b) Elevation



(c) Tributary area to column C shown shaded



(d) Beam A



(e) Beam B

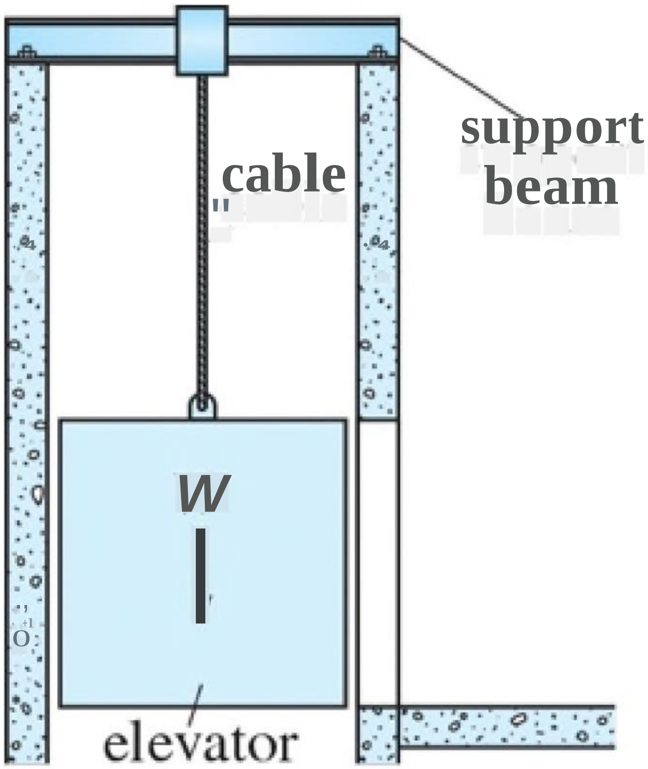
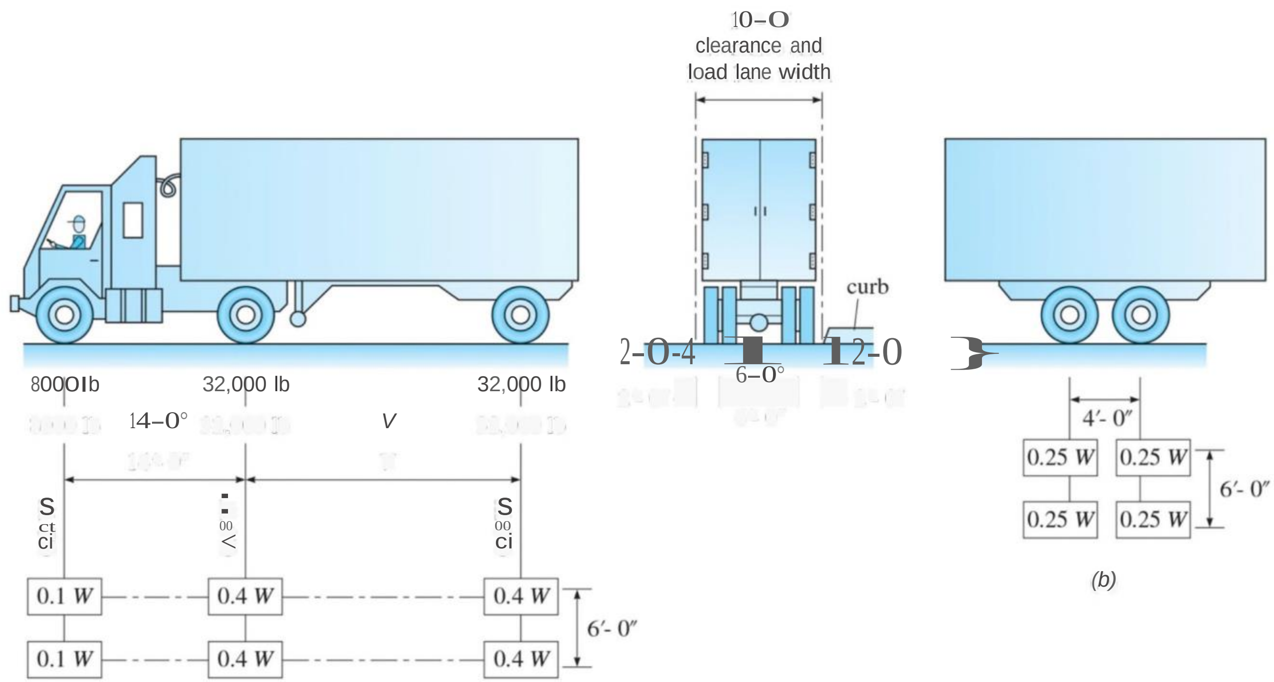
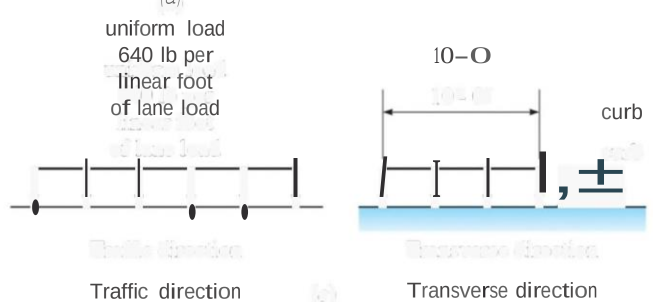

support
beam
cable
W
elevator

Figure 2.7

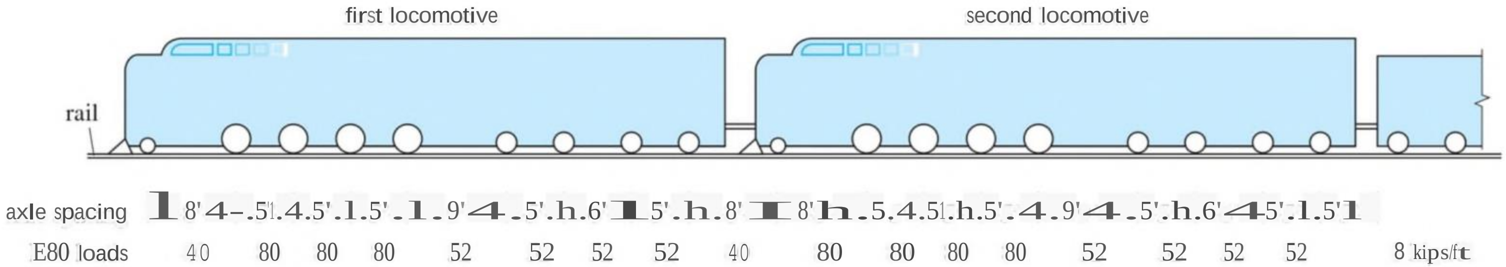


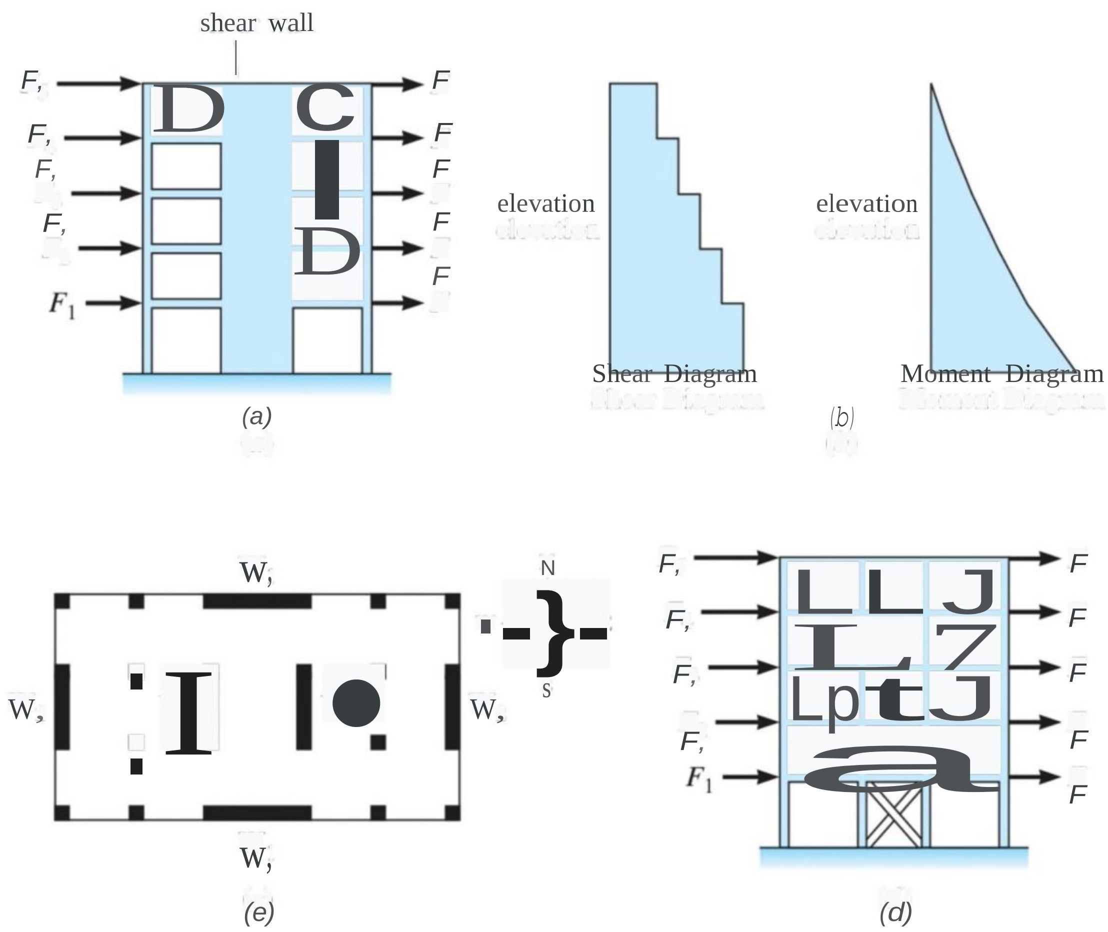
W = Combined weight on the first two axles, which is the same as for the corresponding Design Truck
V = Variable spacing—14 ft to 30 ft inclusive. Spacing to be used is that which produces maximum stresses.

(a)

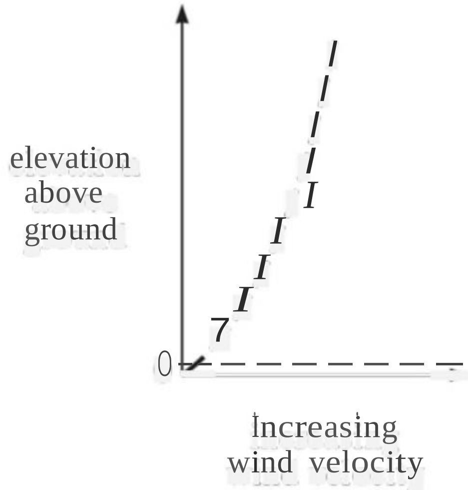


(c)

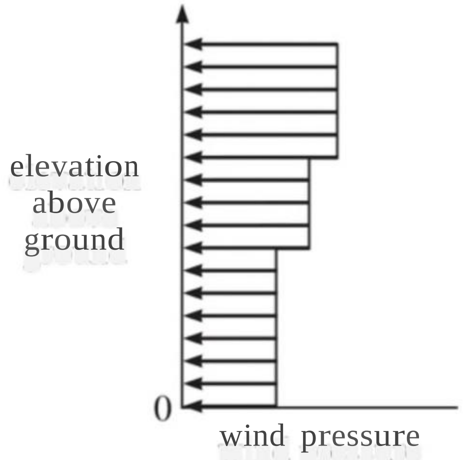

first locomotive
second locomotive
rail
axle spacing 8' 5' 5' 5' 9' 5' 6' 5' 8' 8' 5' 5' 5' 9' 5' 6' 5' 5'
E80 loads 40 80 80 80 52 52 52 52 40 80 80 80 80 52 52 52 52 8 kips/ft


shear wall
F_1
elevation
Shear Diagram
elevation
Moment Diagram
(a)
(b)
N
S
W
F_1
(e)
(d)

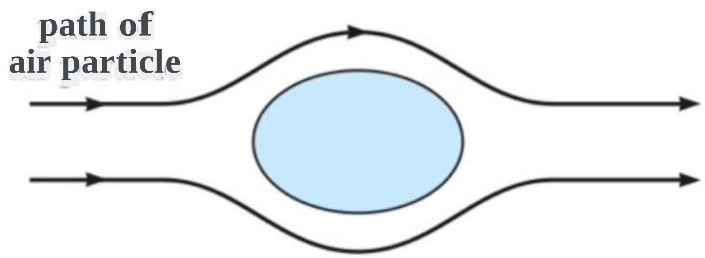
© Chia-Ming Uang


elevation
above
ground
0
increasing
wind velocity

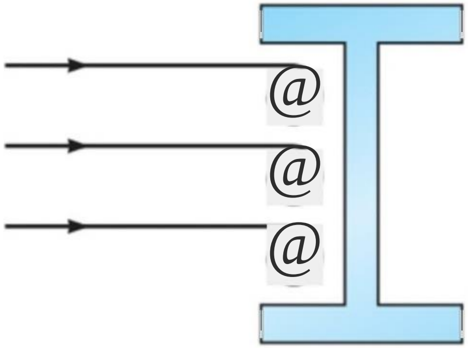
(a)


elevation
above
ground
0
wind pressure

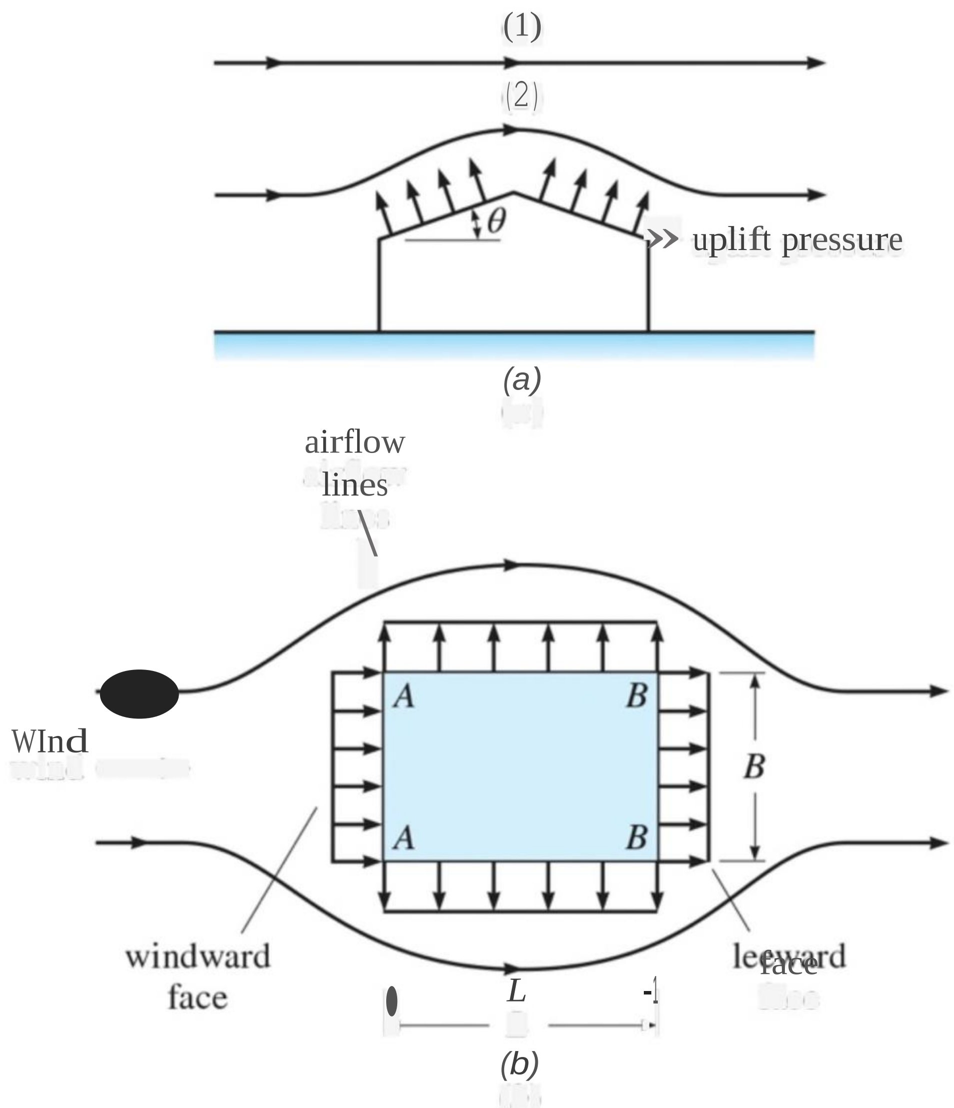
(b)

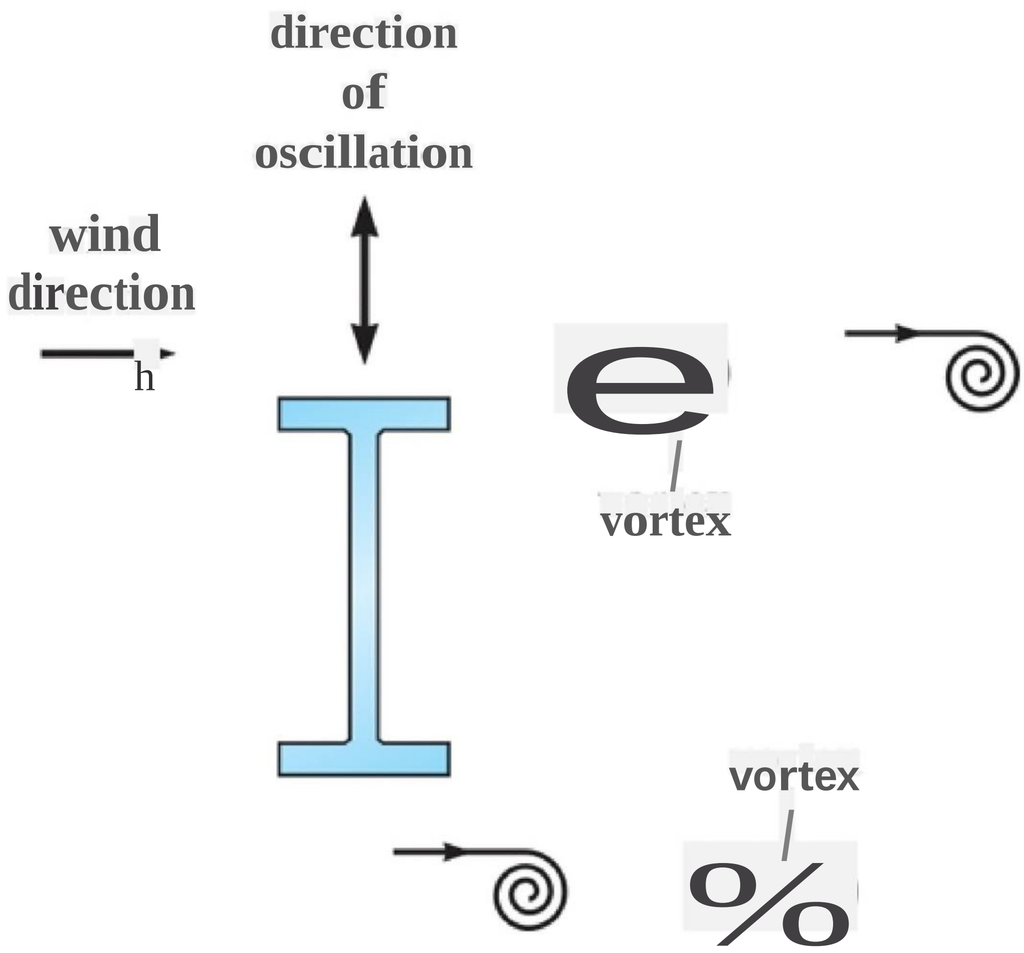


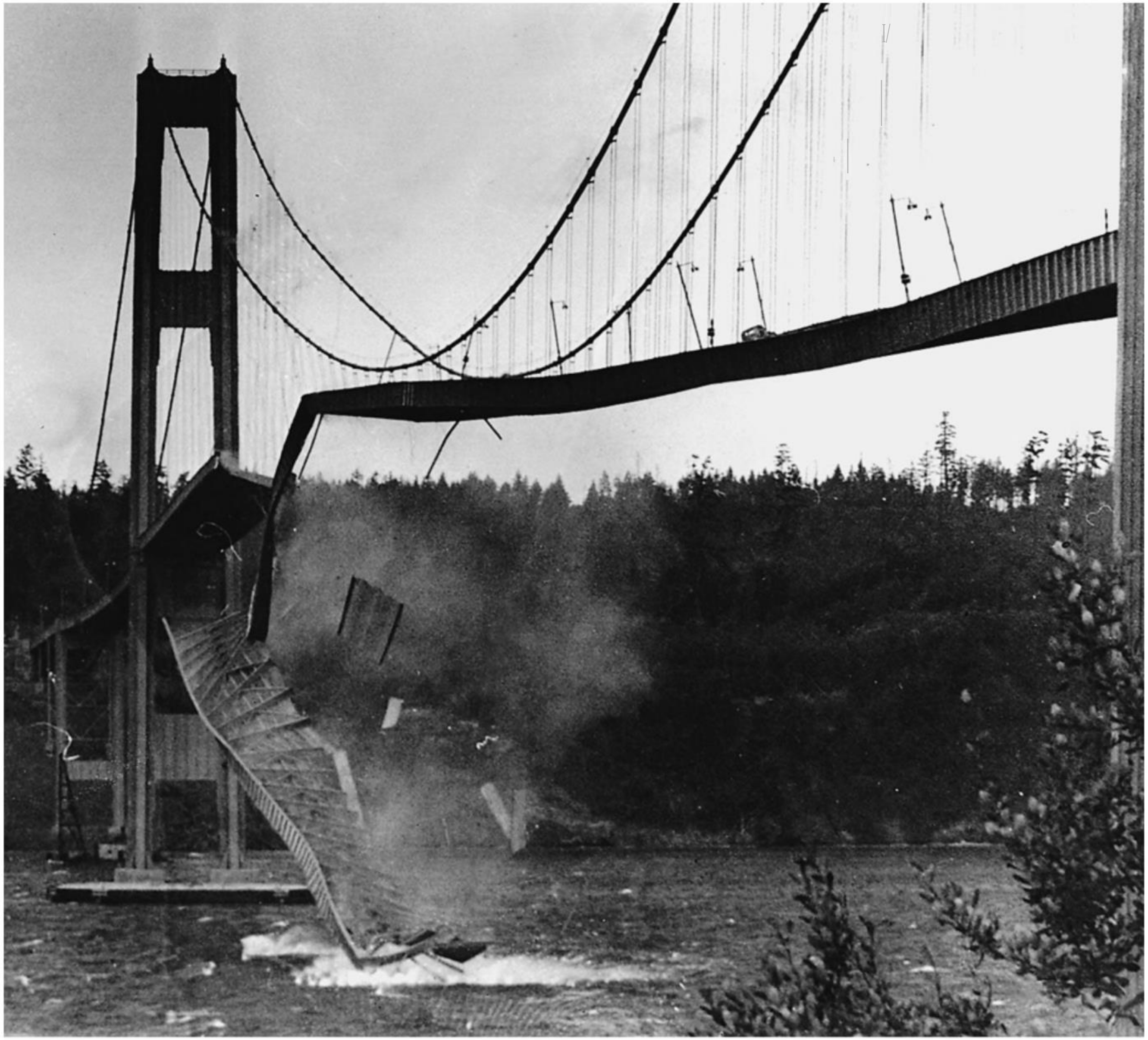
(a)



(b)


(1)
(2)
θ
uplift pressure
(a)
airflow
lines
WInd
A
B
A
B
B
windward
face
leeward
face
L
(b)


direction
of
oscillation
wind
direction
vortex
vortex



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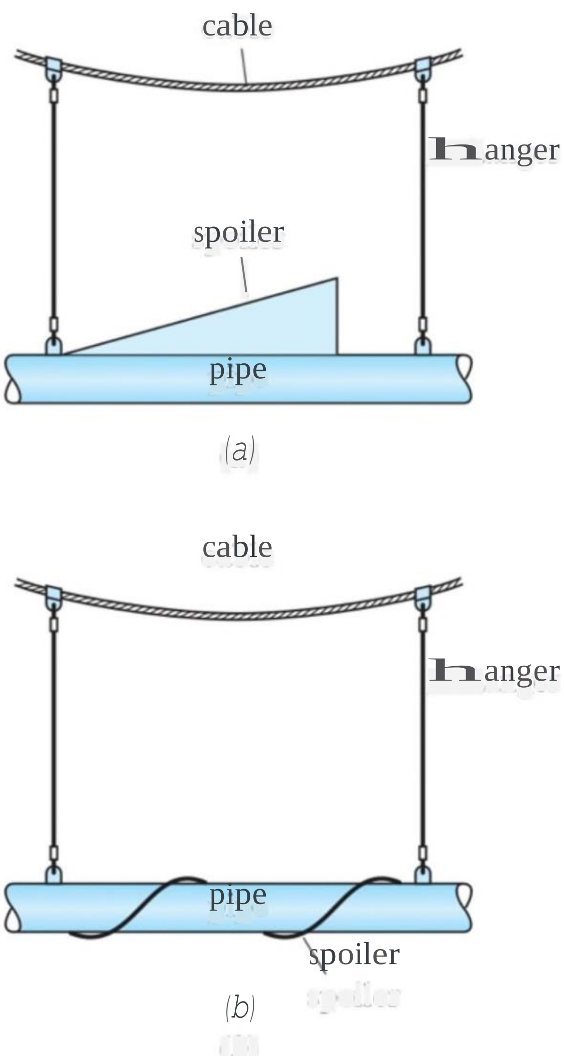
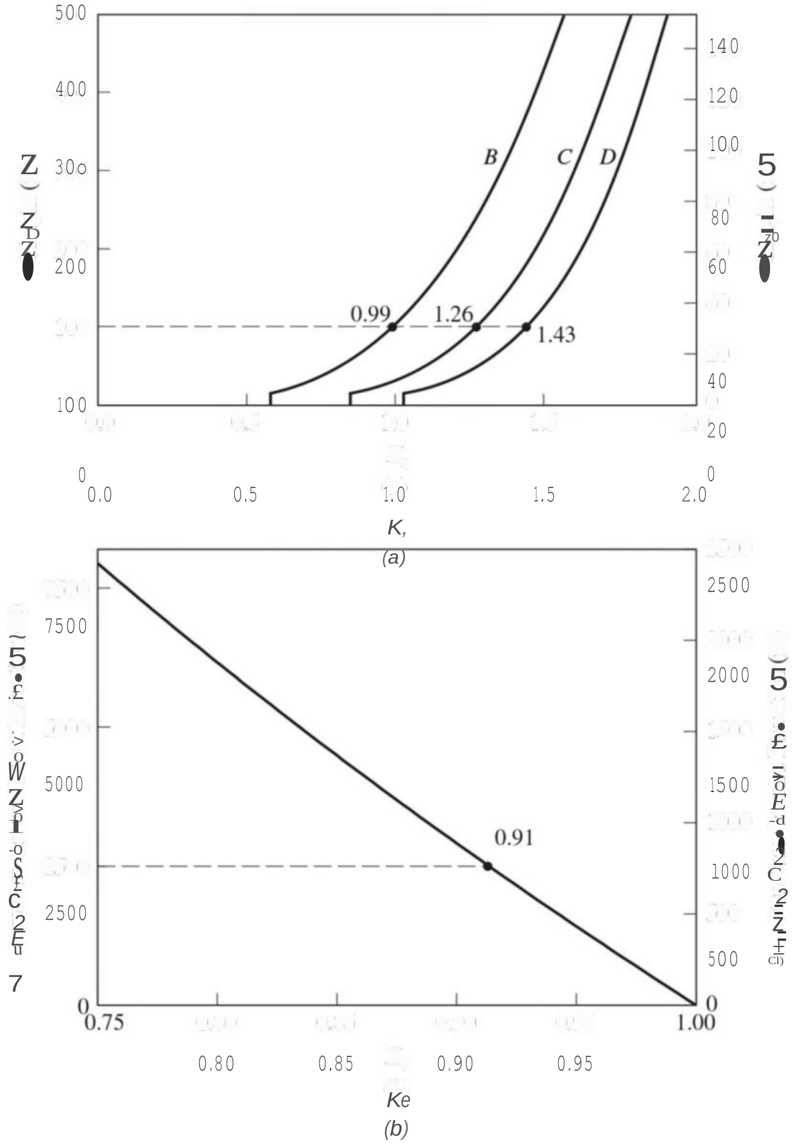
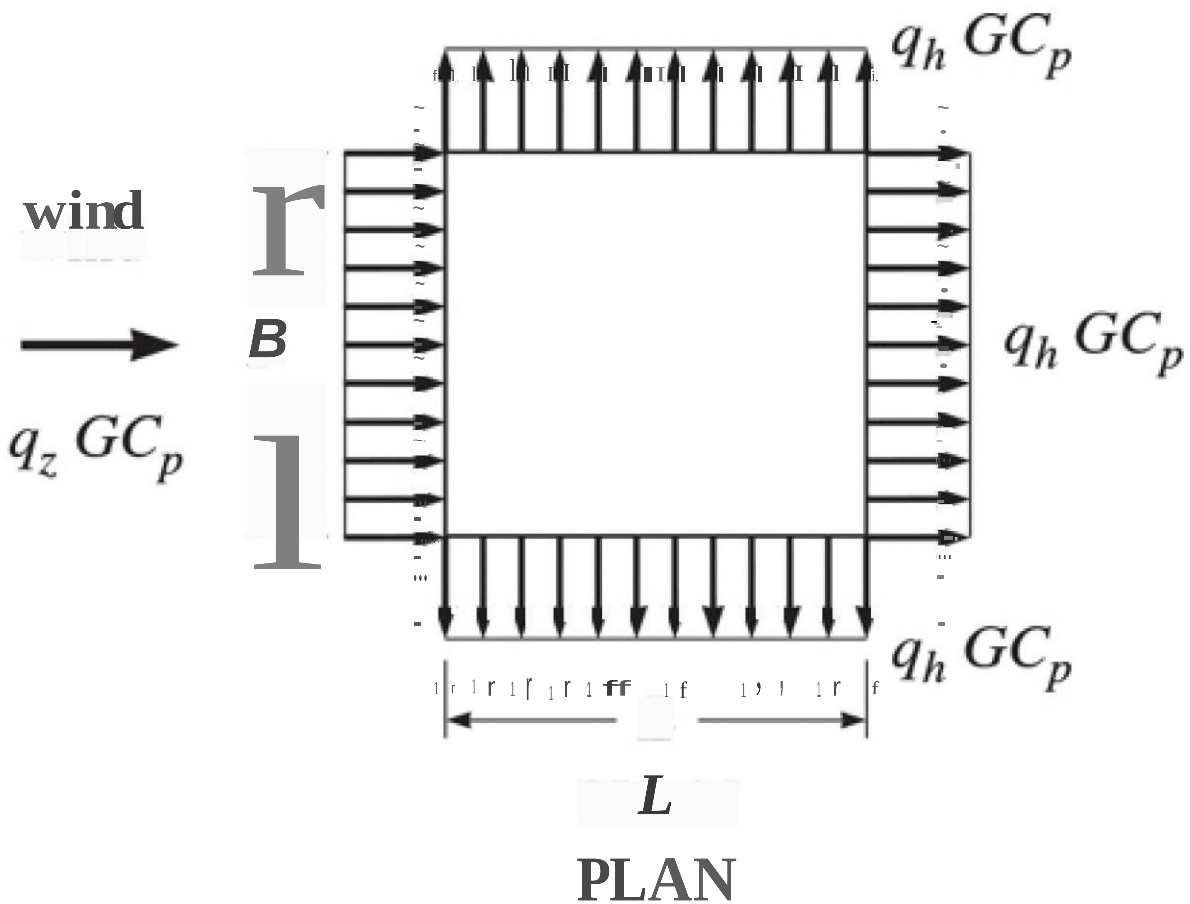
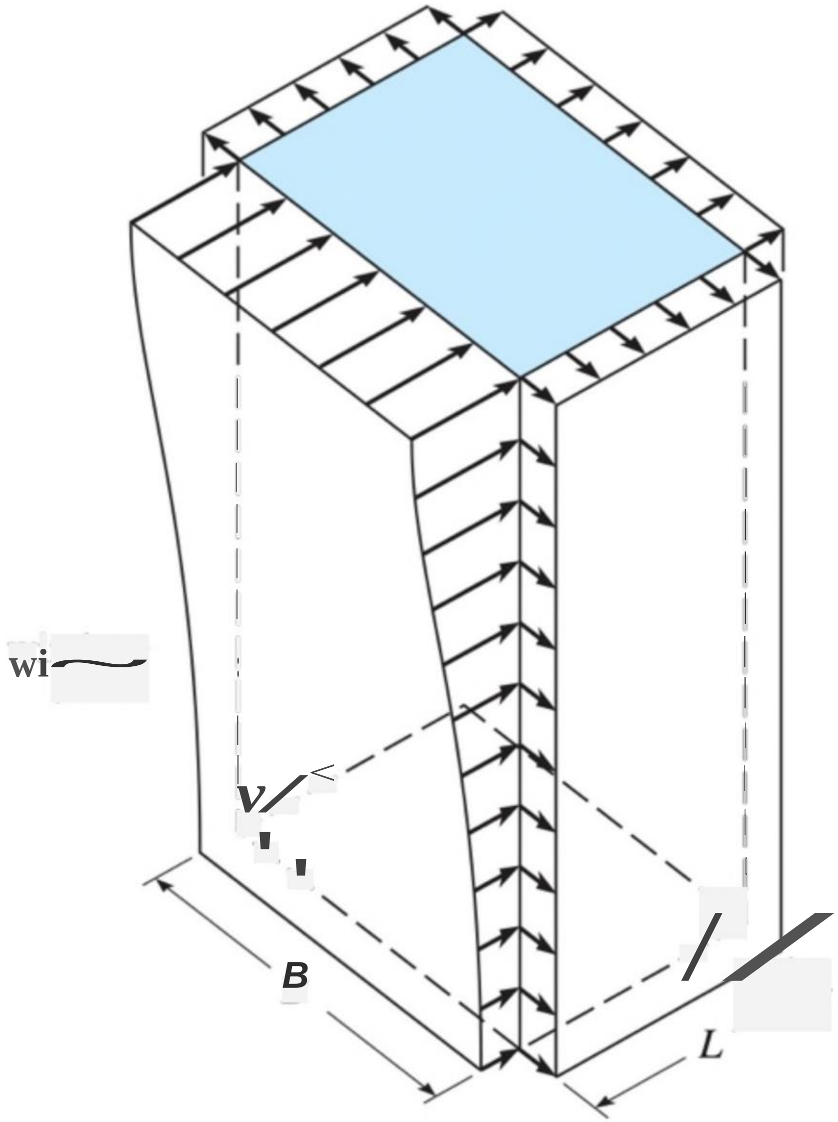

cable
hanger
spoiler
pipe
(a)
cable
hanger
pipe
spoiler
(b)

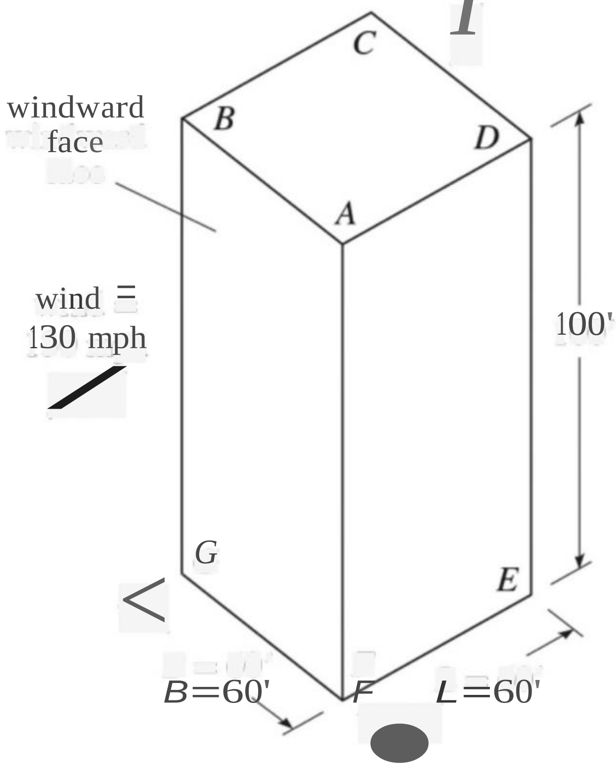
@Aaron Roeth Photography


500
400
300
200
100
0
140
120
100
80
60
40
20
0
B
C
D
0.99
1.26
1.43
0.0
0.5
1.0
1.5
2.0
K,
(a)
7500
5000
2500
0
2500
2000
1500
1000
500
0
0.91
0.75
1.00
0.80
0.85
0.90
0.95
Ke
(b)



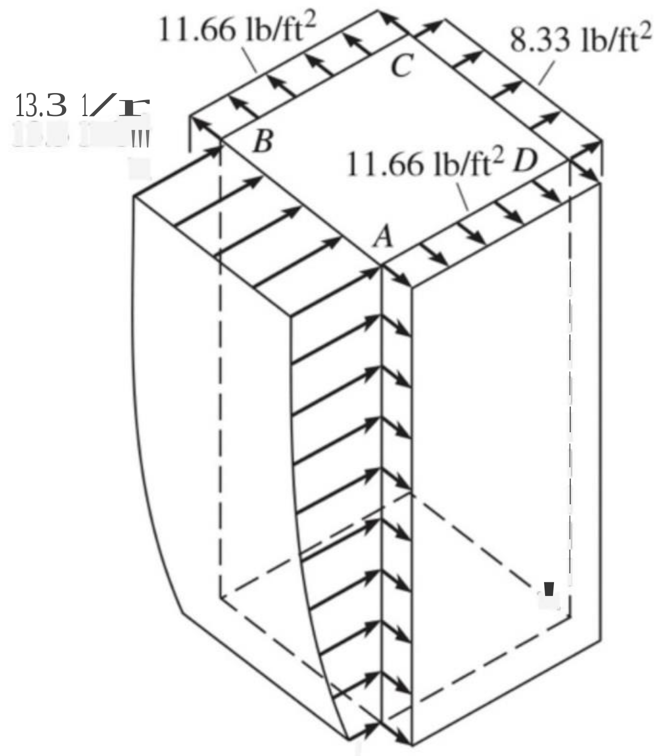
wind
B
$q_z\ GC_p$
$q_h\ GC_p$
$q_h\ GC_p$
$q_h\ GC_p$
L
PLAN


wi
v
B
L



leeward face
windward face
wind = 130 mph
B
C
D
A
G
E
F
100'
B=60'
L=60'

(a)



11.66 lb/ft²
8.33 lb/ft²
11.66 lb/ft² D
B
C
A
11.3 lb/

(b)

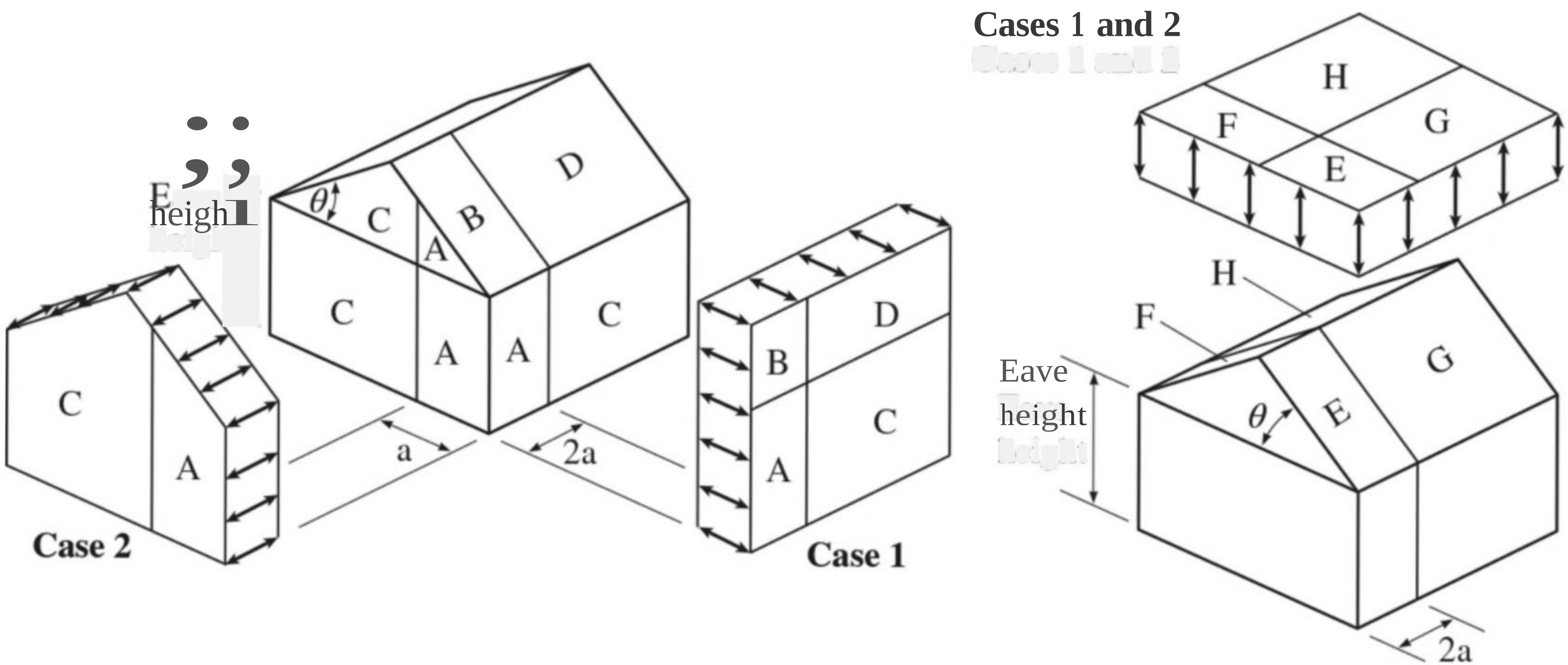
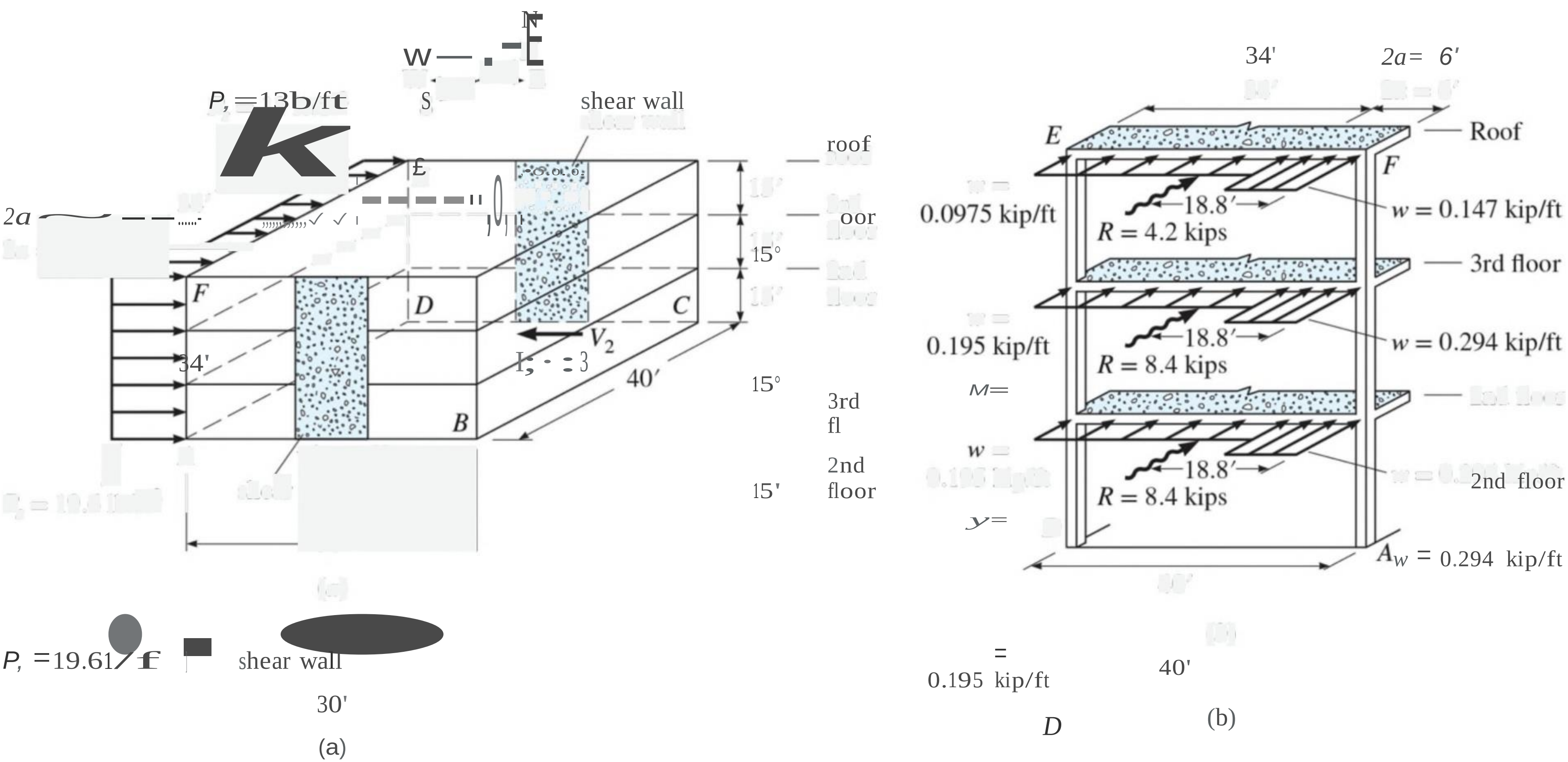
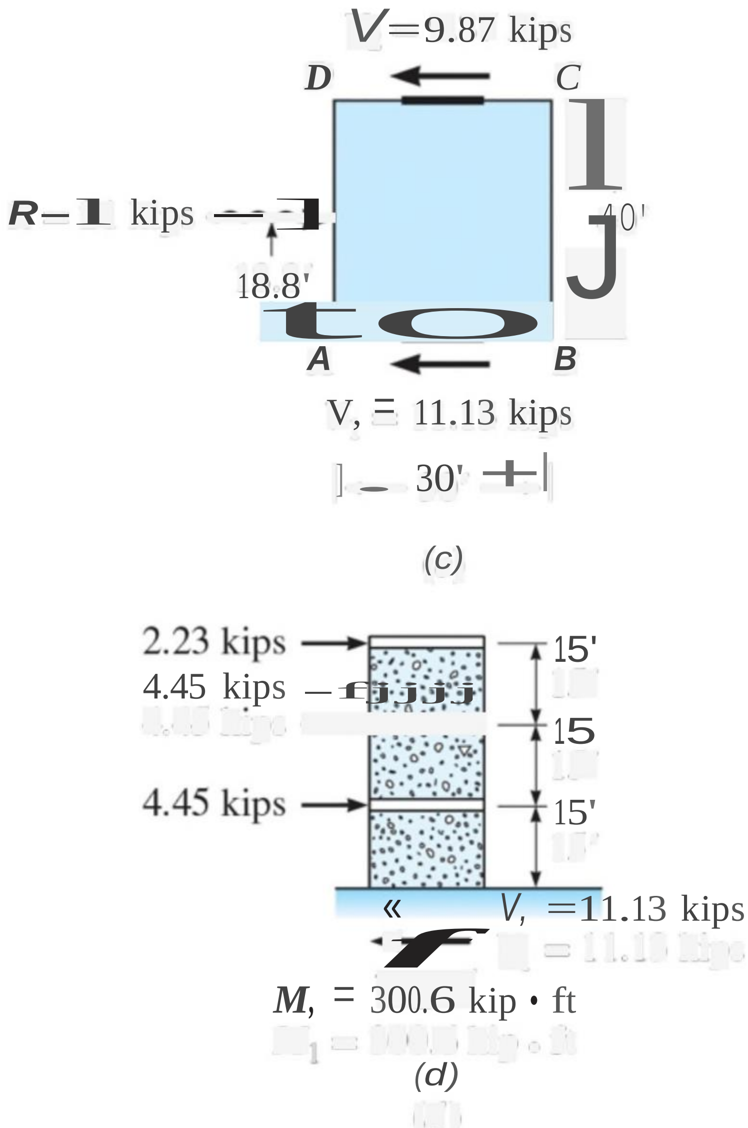

Cases 1 and 2
H
F
G
E
H
F
Eave
height
θ
E
G
2a
E
height
θ
C
A
B
D
C
A
A
C
C
A
Case 2
a
2a
B
D
C
A
Case 1

Figure 2.19



shear wall
roof
15'
15'
15'
34'
40'
F
D
C
B
E
V_2
(a)
34'
2a = 6'
Roof
3rd floor
2nd floor
w = 0.0975 kip/ft
w = 0.195 kip/ft
w = 0.147 kip/ft
w = 0.294 kip/ft
18.8'
R = 4.2 kips
R = 8.4 kips
40'
(b)


V=9.87 kips
D
C
R=1 kips
40'
18.8'
A
B
V, = 11.13 kips
30'
(c)
2.23 kips
4.45 kips
4.45 kips
15'
15
15'
V, =11.13 kips
M, = 300.6 kip • ft
(d)

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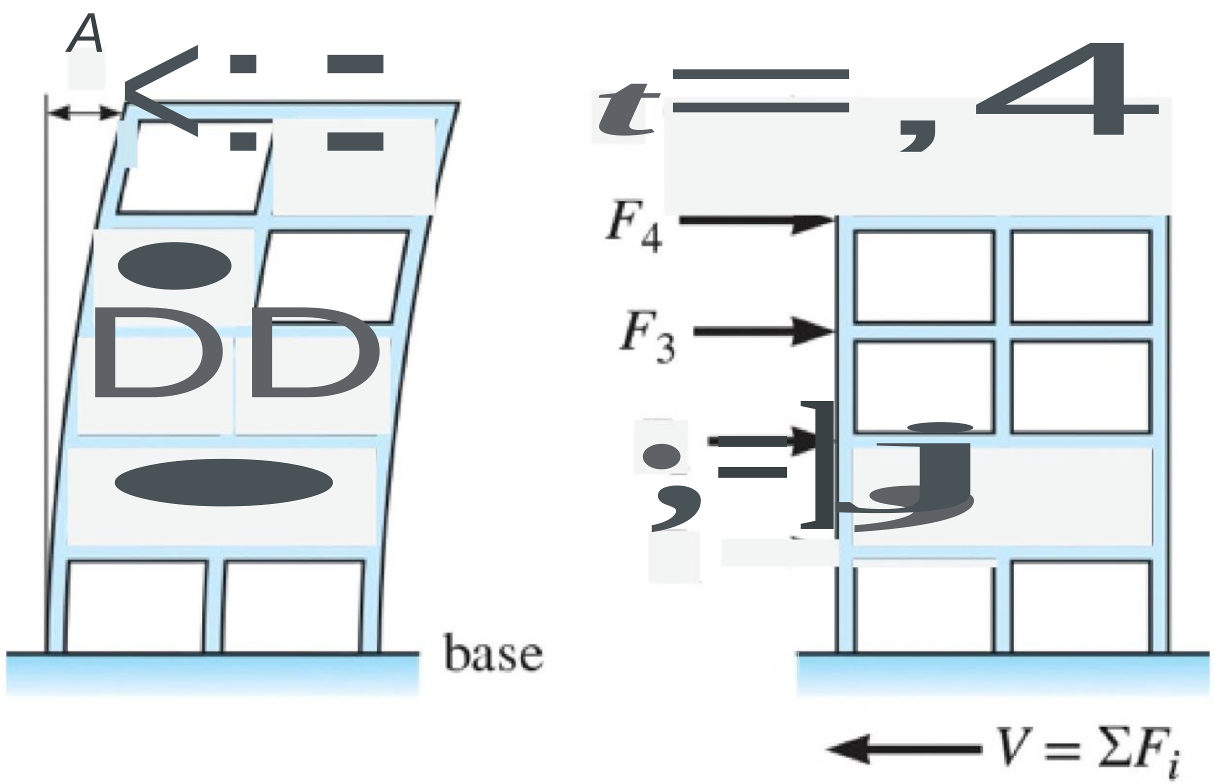
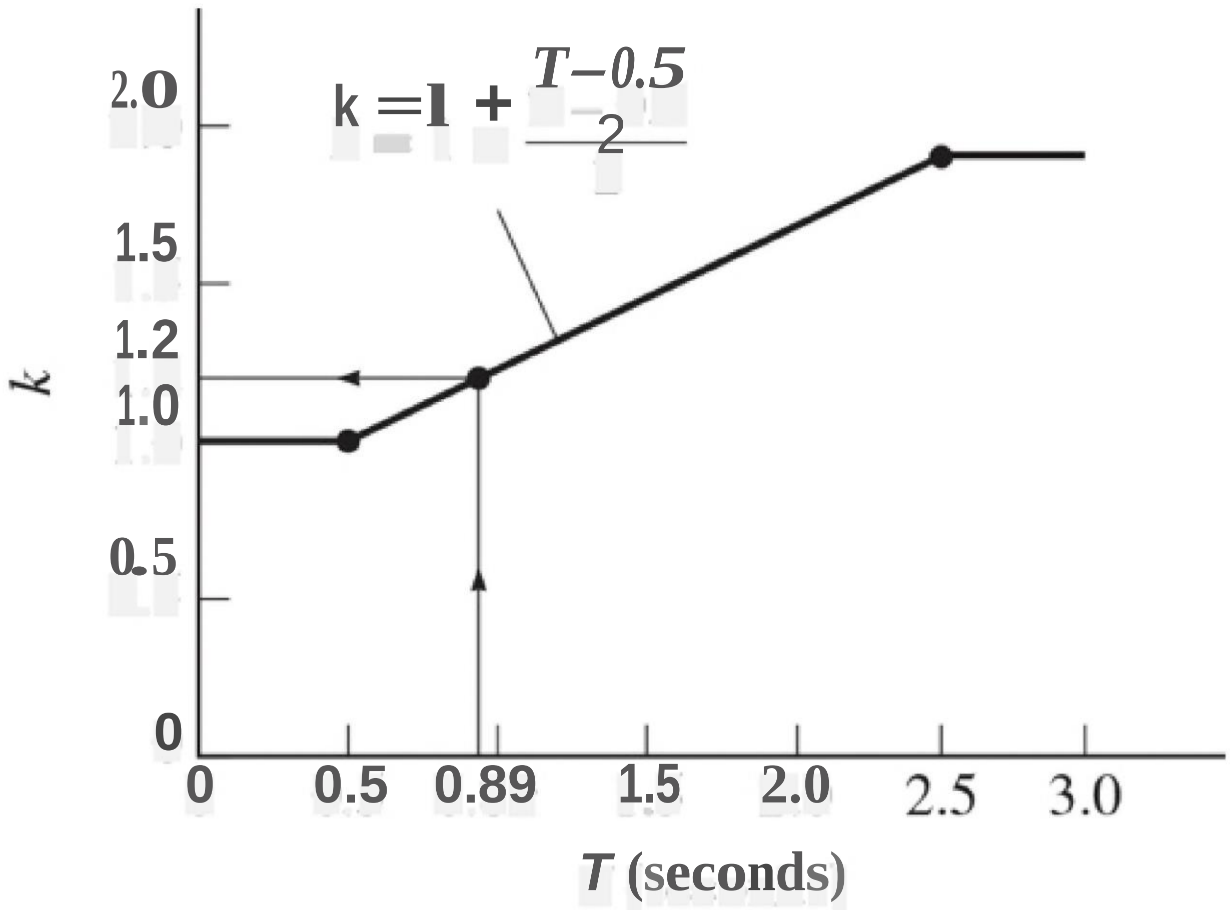
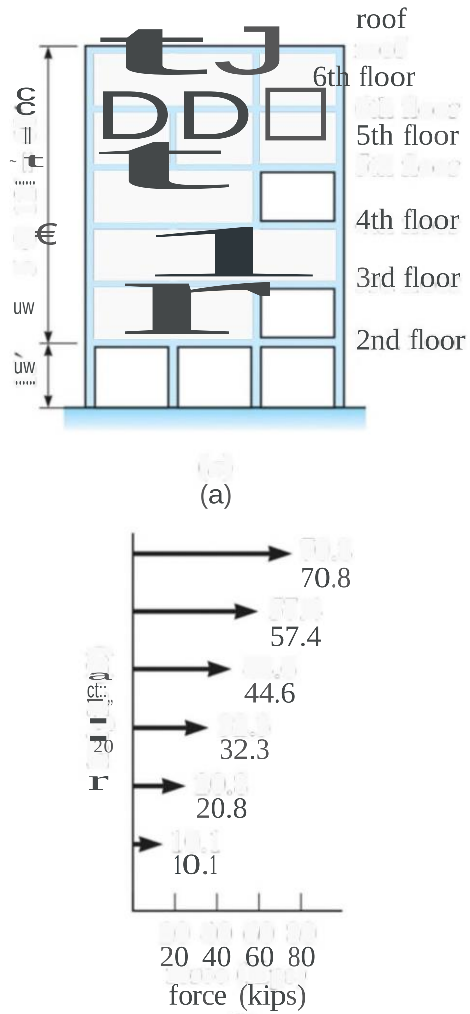


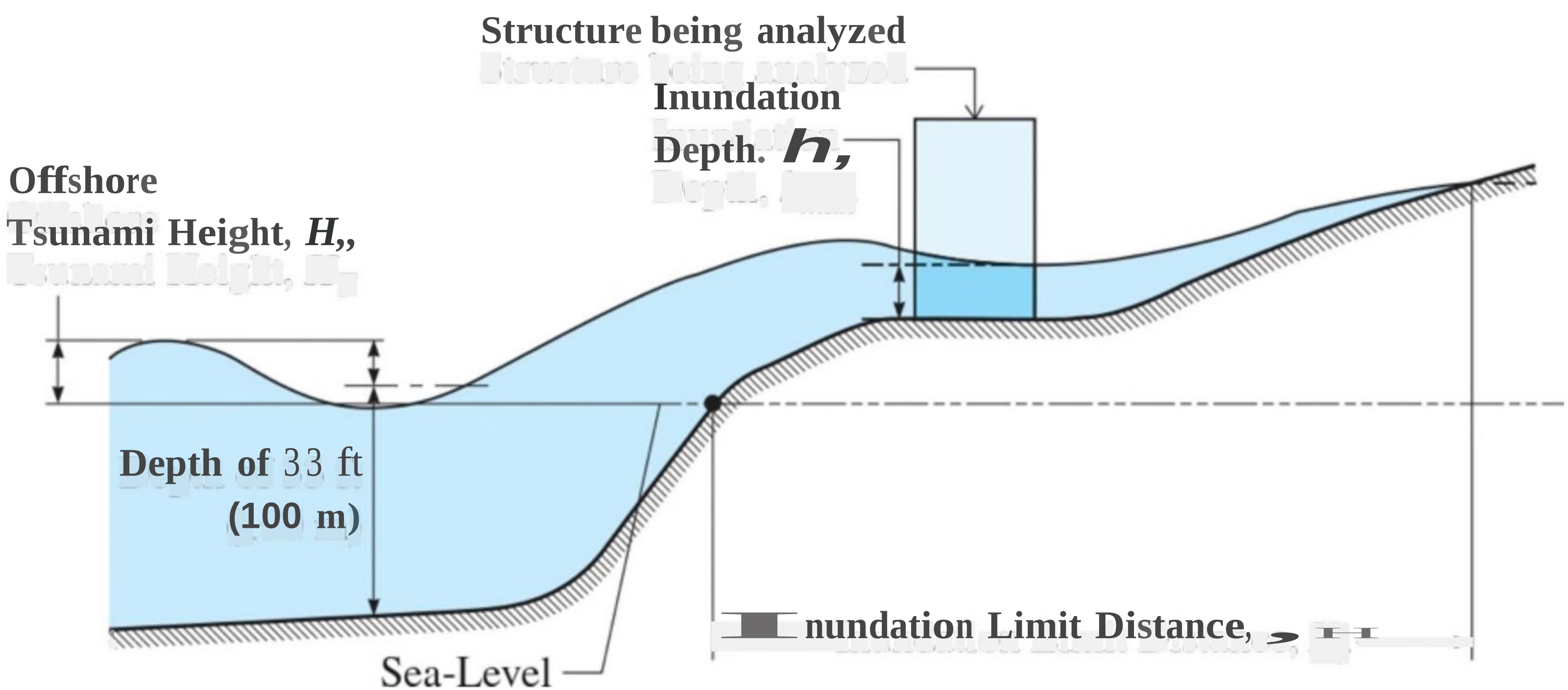
(a) (b)

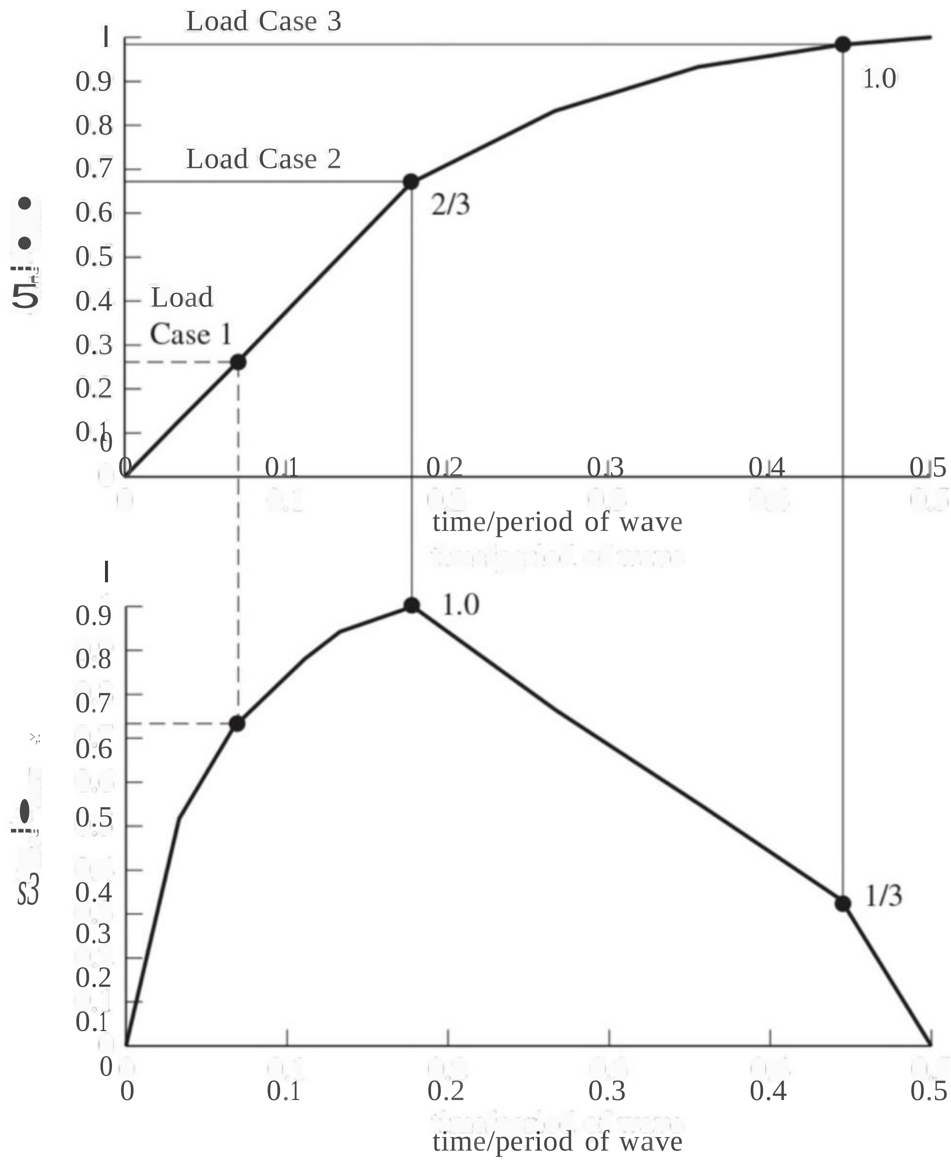

k = 1 + (T−0.5)/2
2.0
1.5
1.2
1.0
0.5
0
k
0
0.5
0.89
1.5
2.0
2.5
3.0
T (seconds)

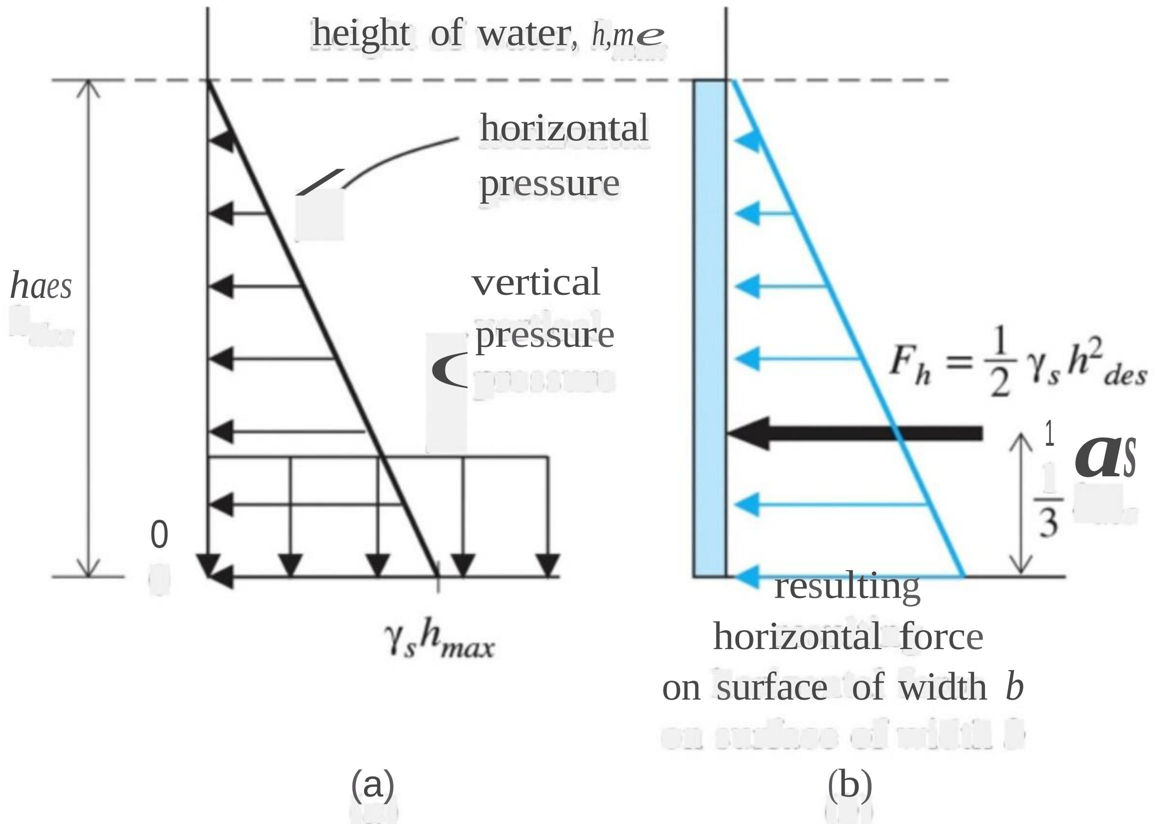

roof
6th floor
5th floor
4th floor
3rd floor
2nd floor
70.8
57.4
44.6
32.3
20.8
10.1
20 40 60 80
force (kips)

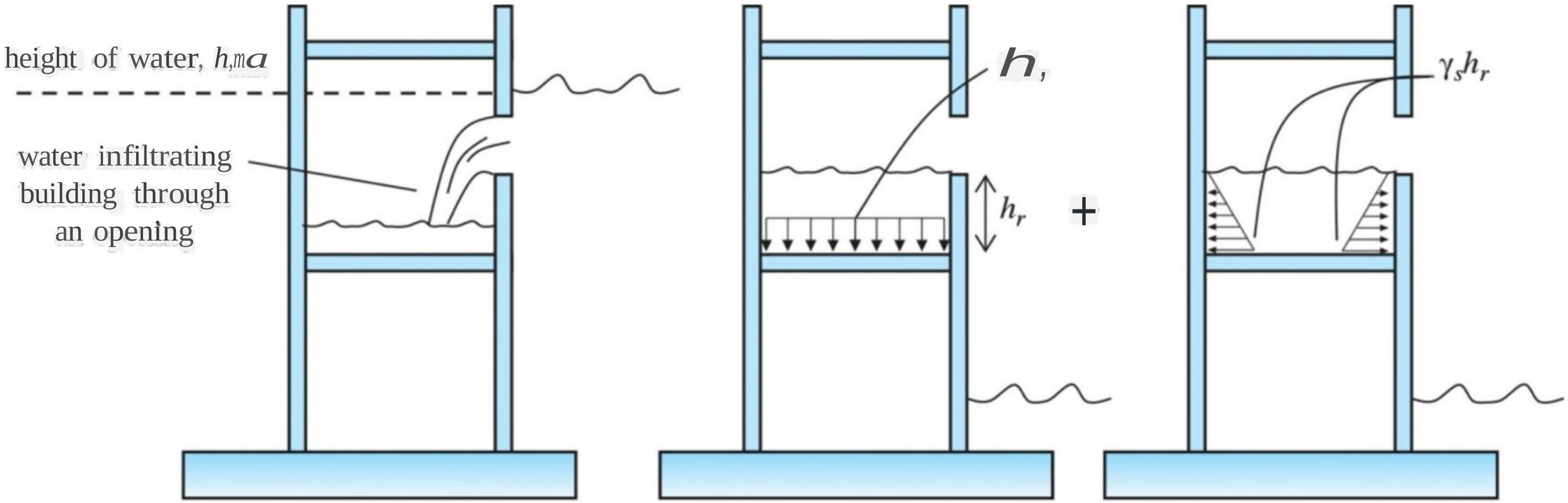
(a)

(b)


Structure being analyzed
Inundation Depth. h,
Offshore Tsunami Height, H,,
Depth of 33 ft (100 m)
Sea-Level
nundation Limit Distance,

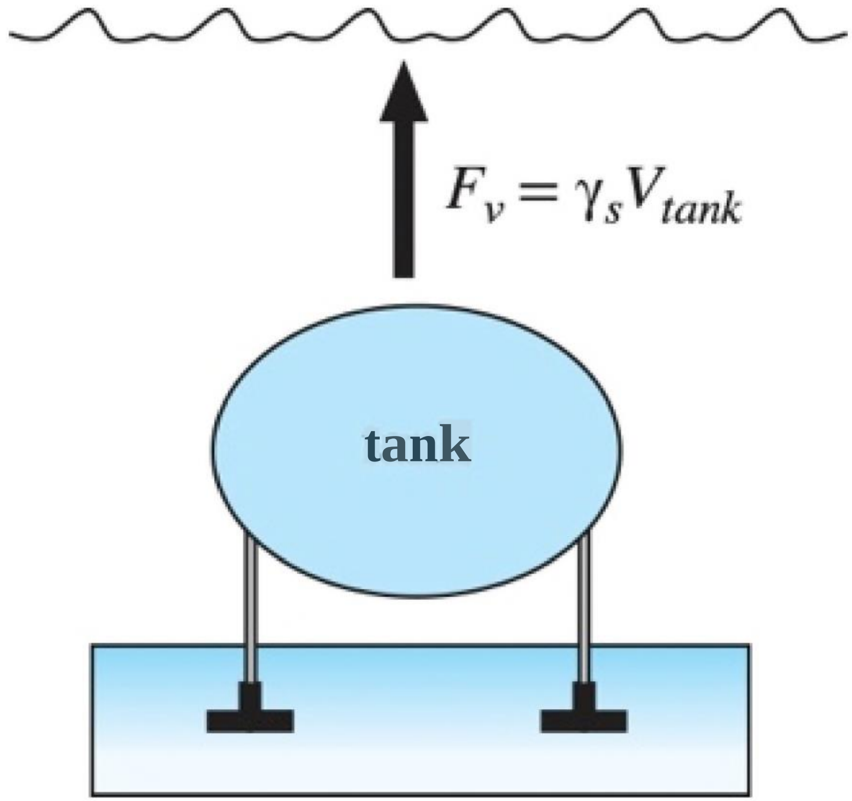

Load Case 3
Load Case 2
Load
Case 1
1.0
2/3
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0.1
0.2
0.3
0.4
0.5
time/period of wave
1.0
1/3
time/period of wave


height of water, h,me
horizontal
pressure
vertical
pressure
haes
0
γshmax
Fh = 1/2 γs h2des
1/3
resulting
horizontal force
on surface of width b
(a)
(b)

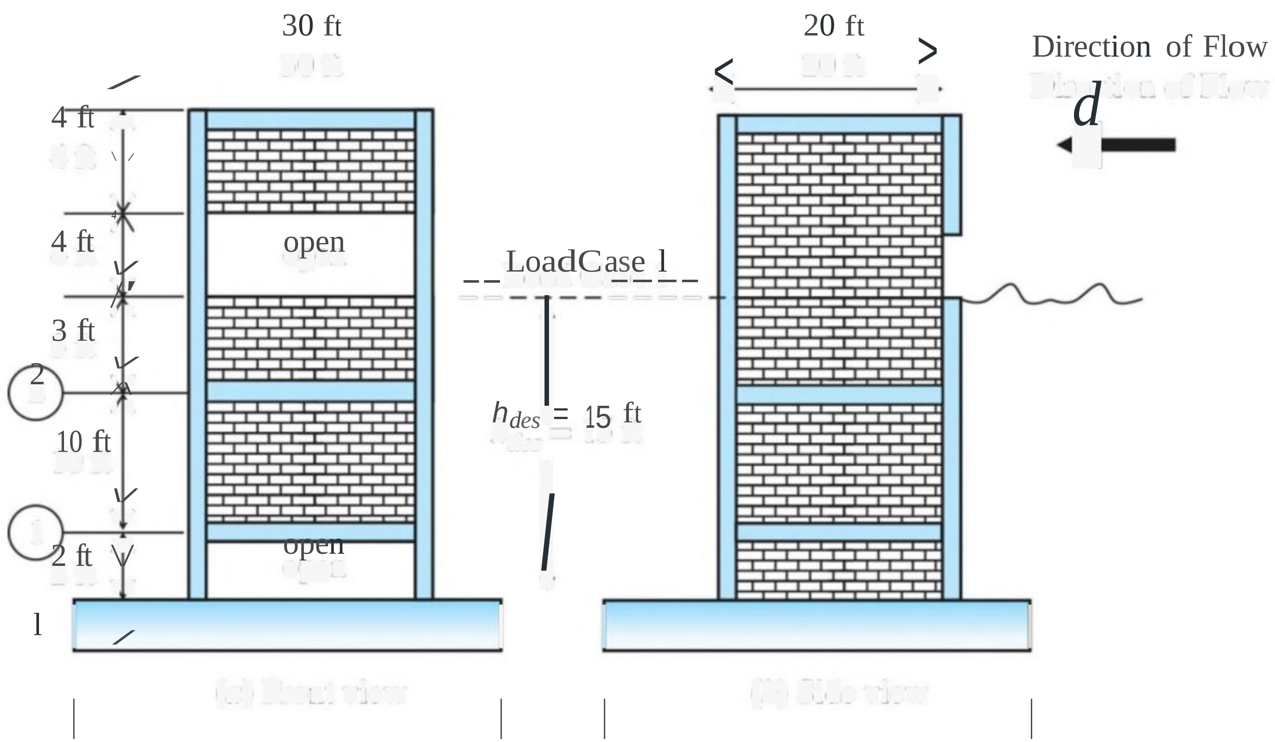


(a) at maximum inundation

(b) after wave recession

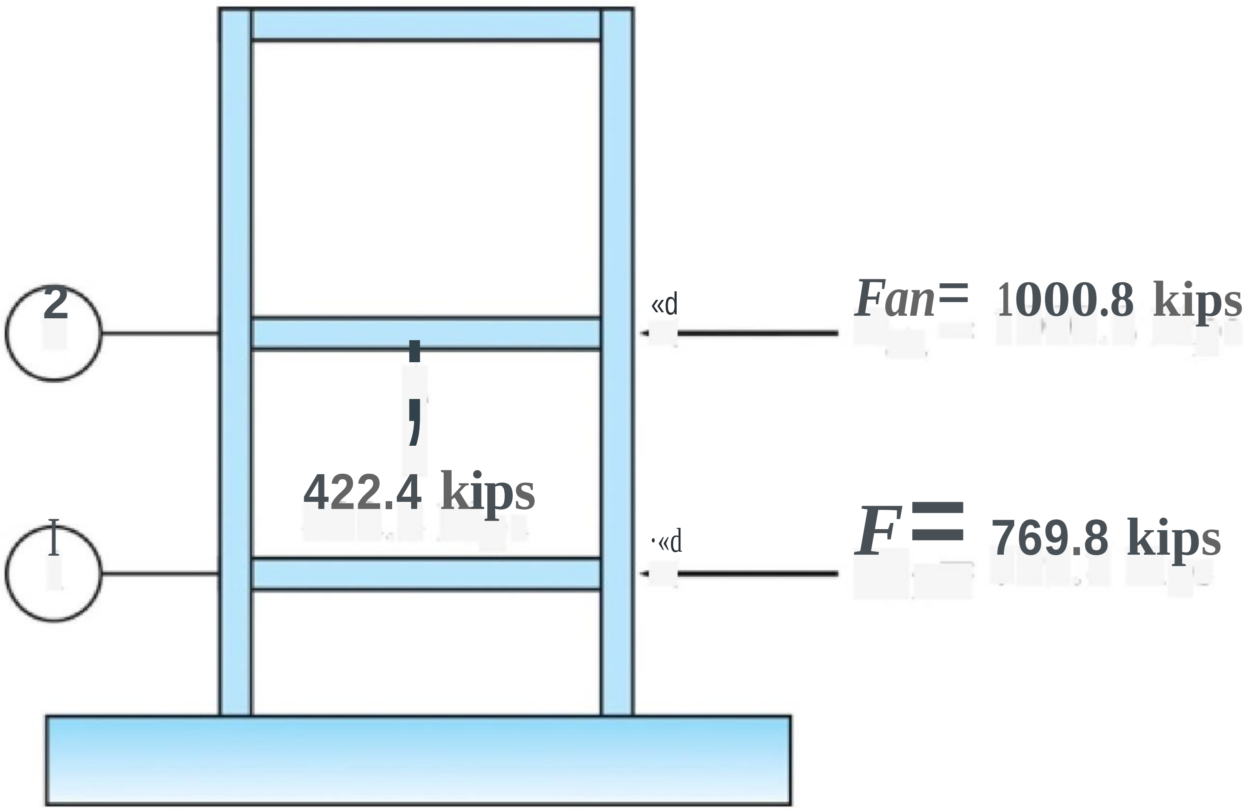

$F_v = \gamma_s V_{tank}$
tank

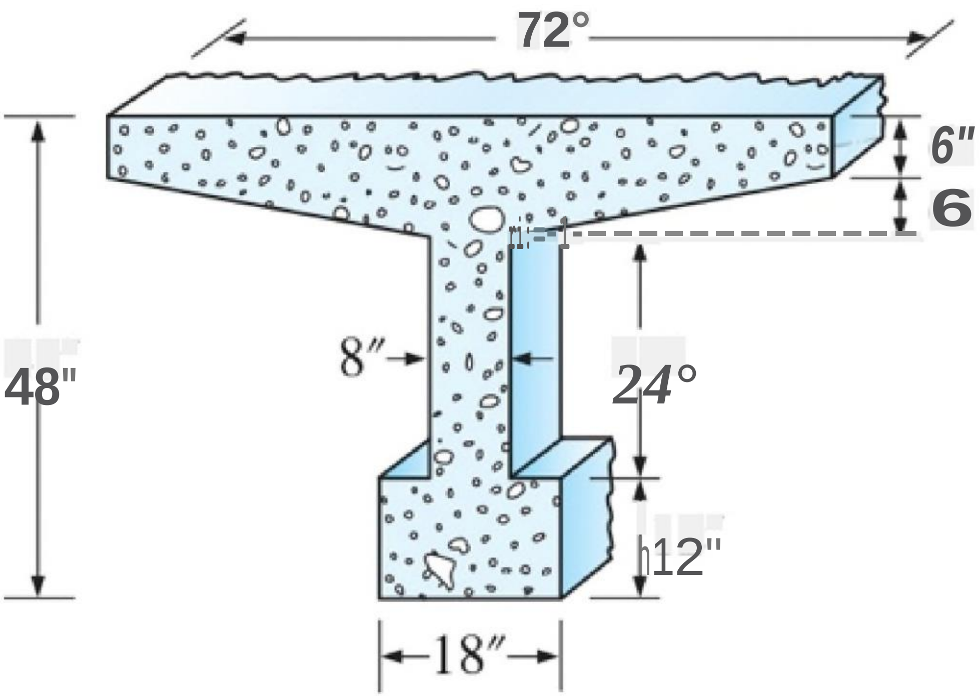
C Taichiro Oazaki



(a) Front view

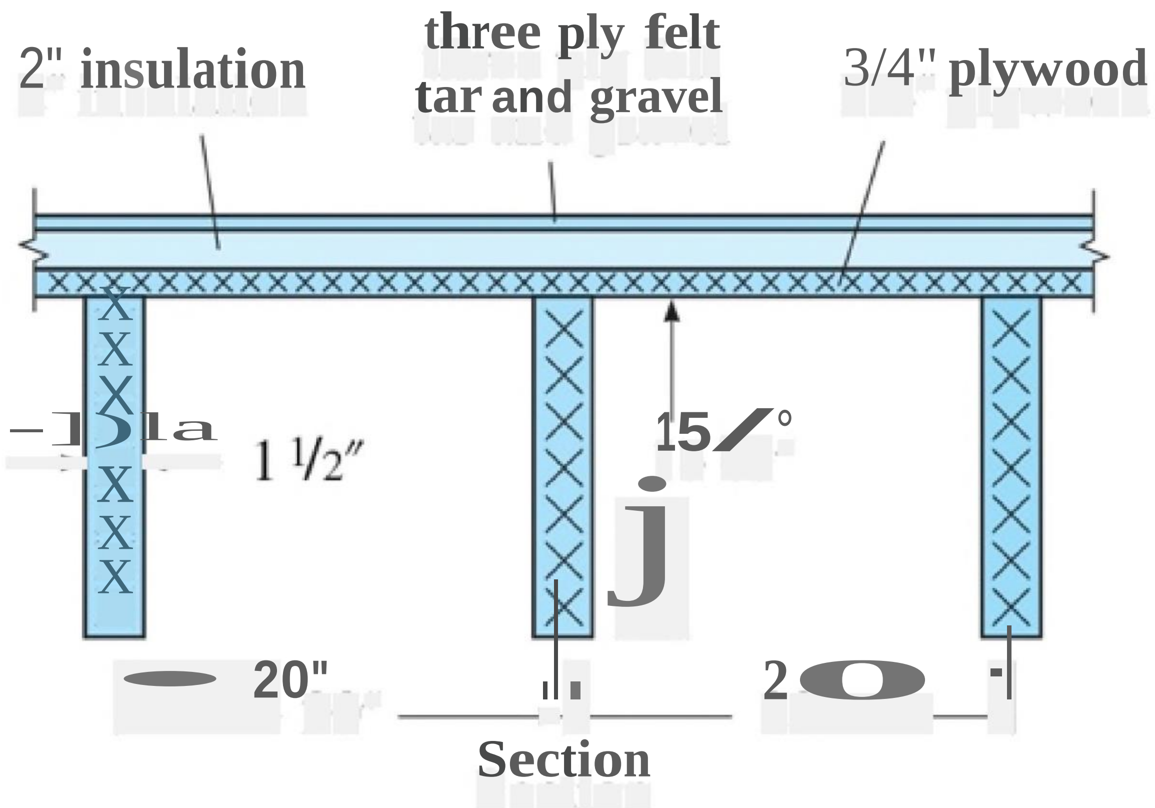
(b) Side view


2
422.4 kips
Fan= 1000.8 kips
F = 769.8 kips

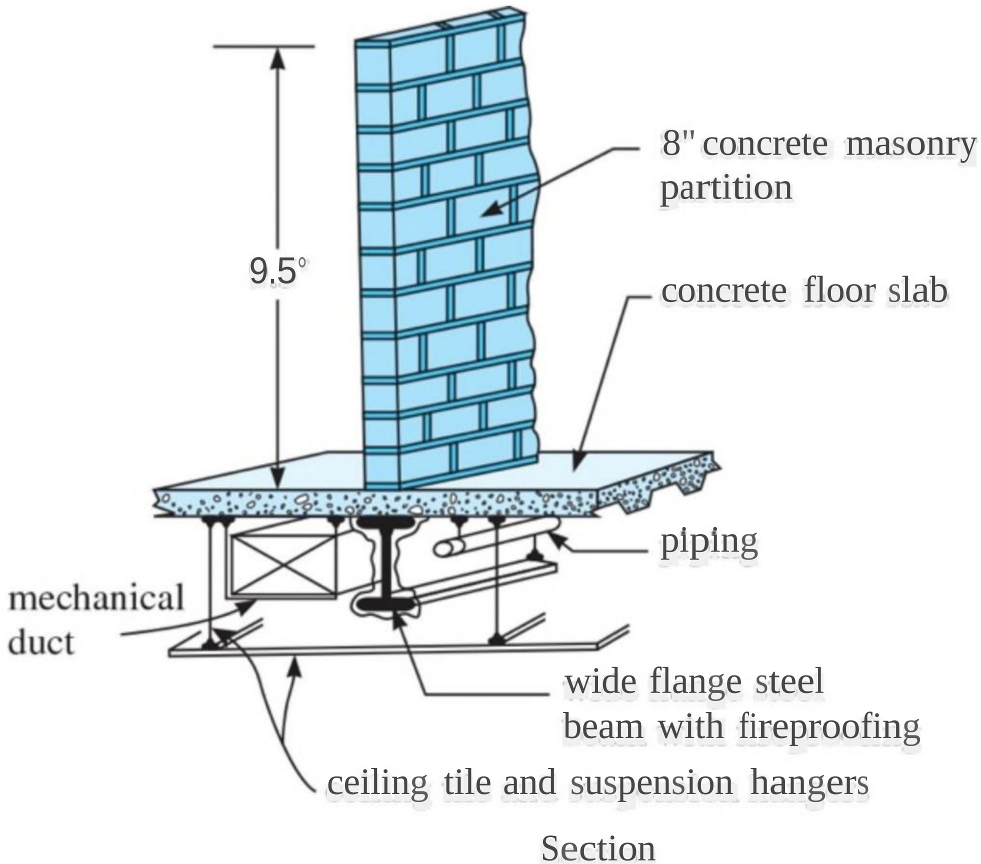


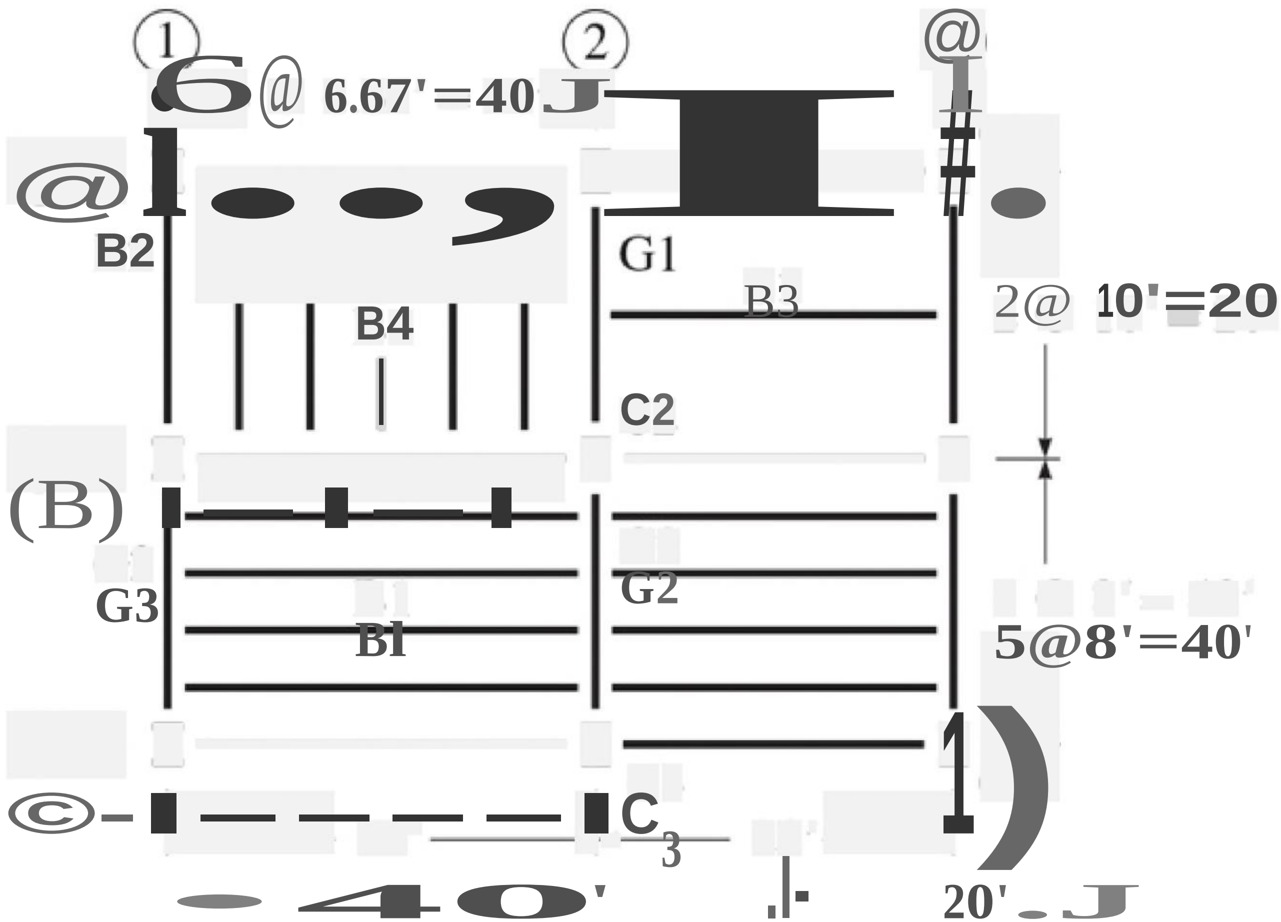
Section

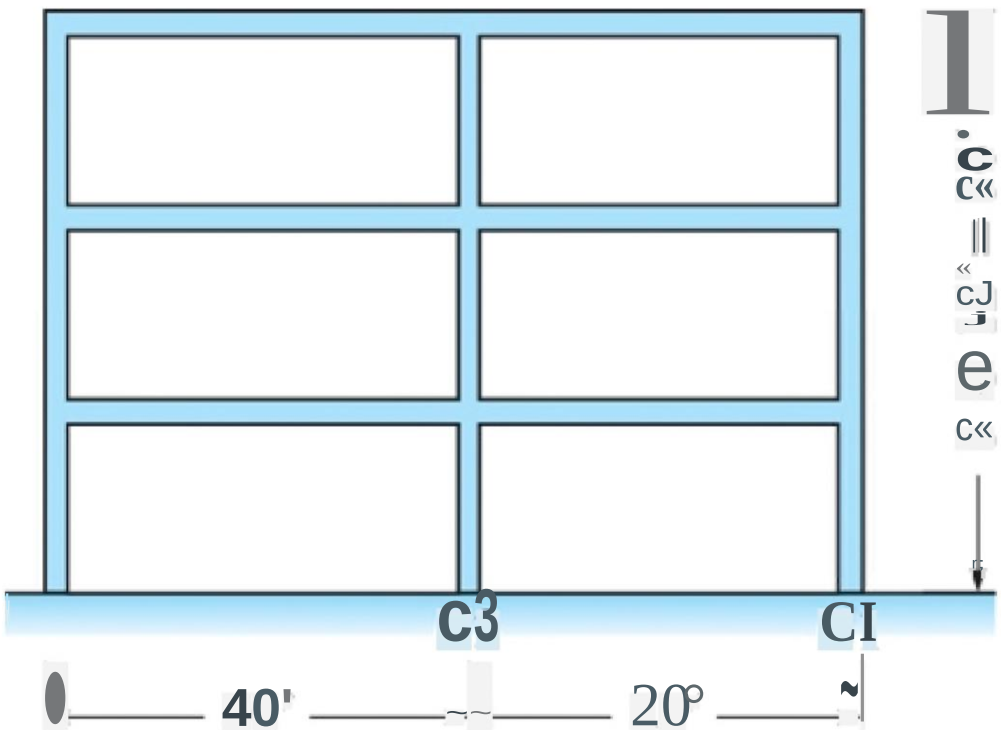
Figure P2.2



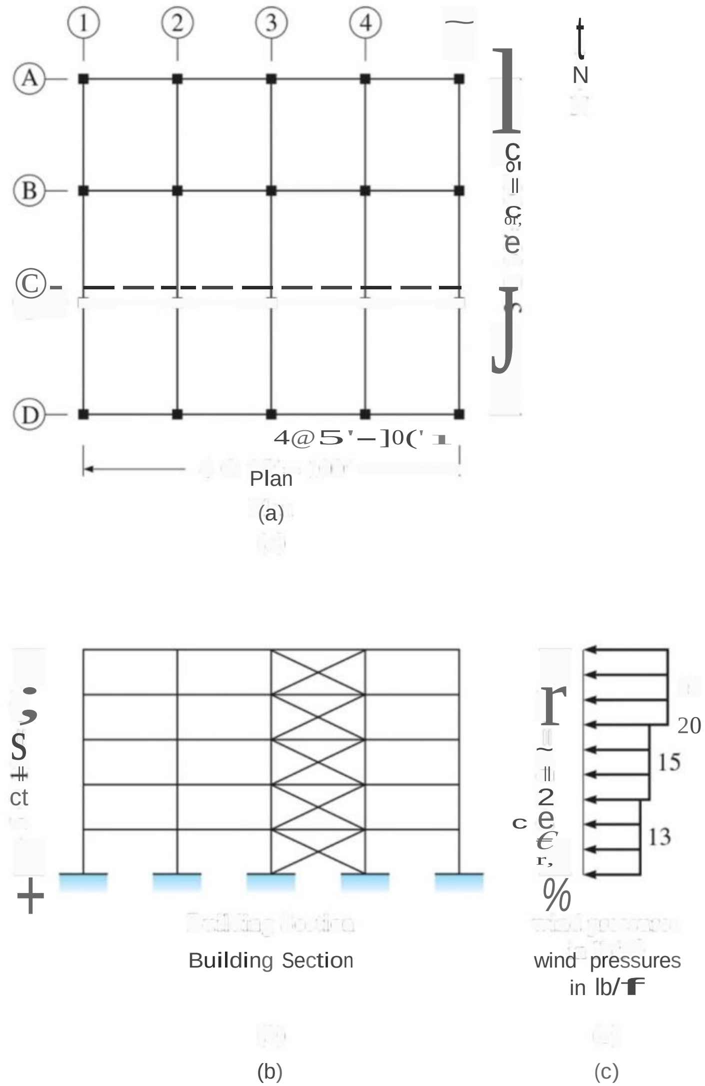
three ply felt
tar and gravel
2" insulation
3/4" plywood
1 1/2"
20"
Section

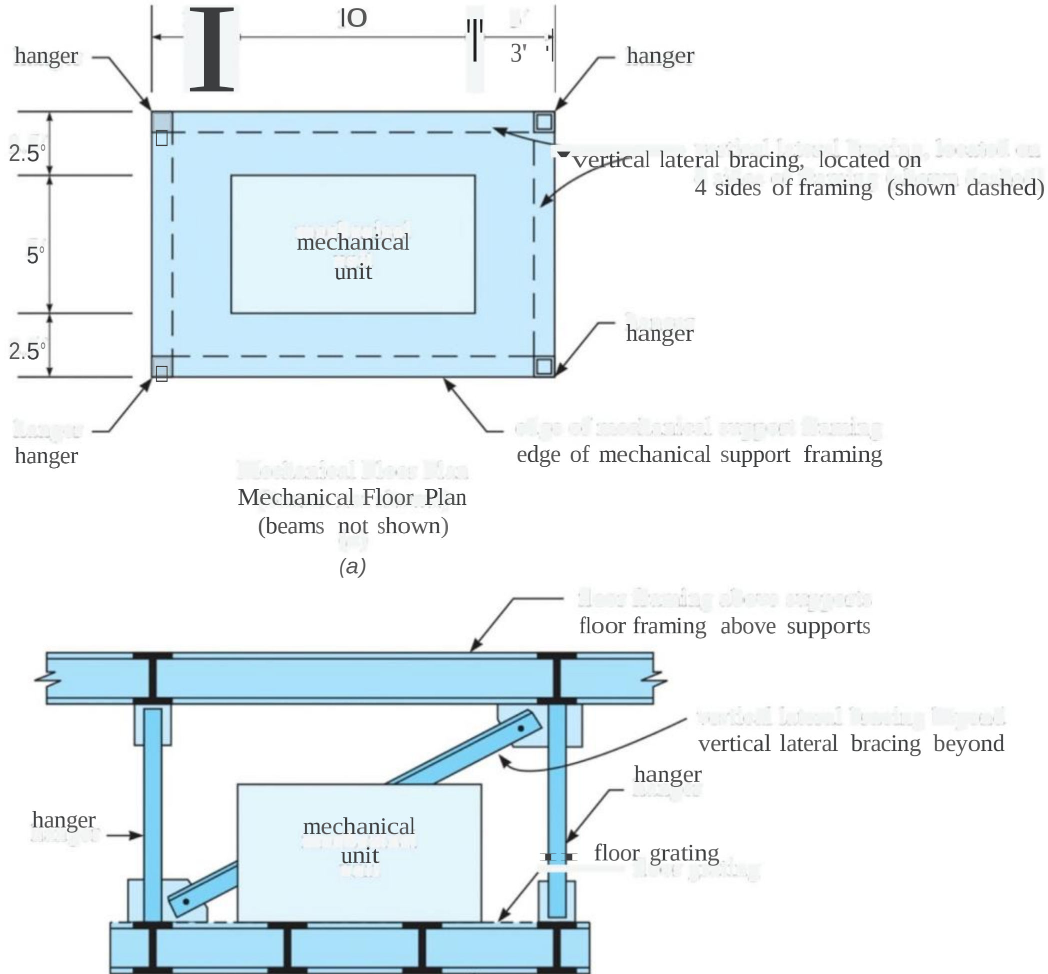

9.5'
8" concrete masonry partition
concrete floor slab
piping
mechanical duct
wide flange steel beam with fireproofing
ceiling tile and suspension hangers
Section


6 @ 6.67'=40
B2
B4
G1
B3
2@ 10'=20
C2
(B)
G3
B1
G2
5@8'=40'
C3
40'
20'



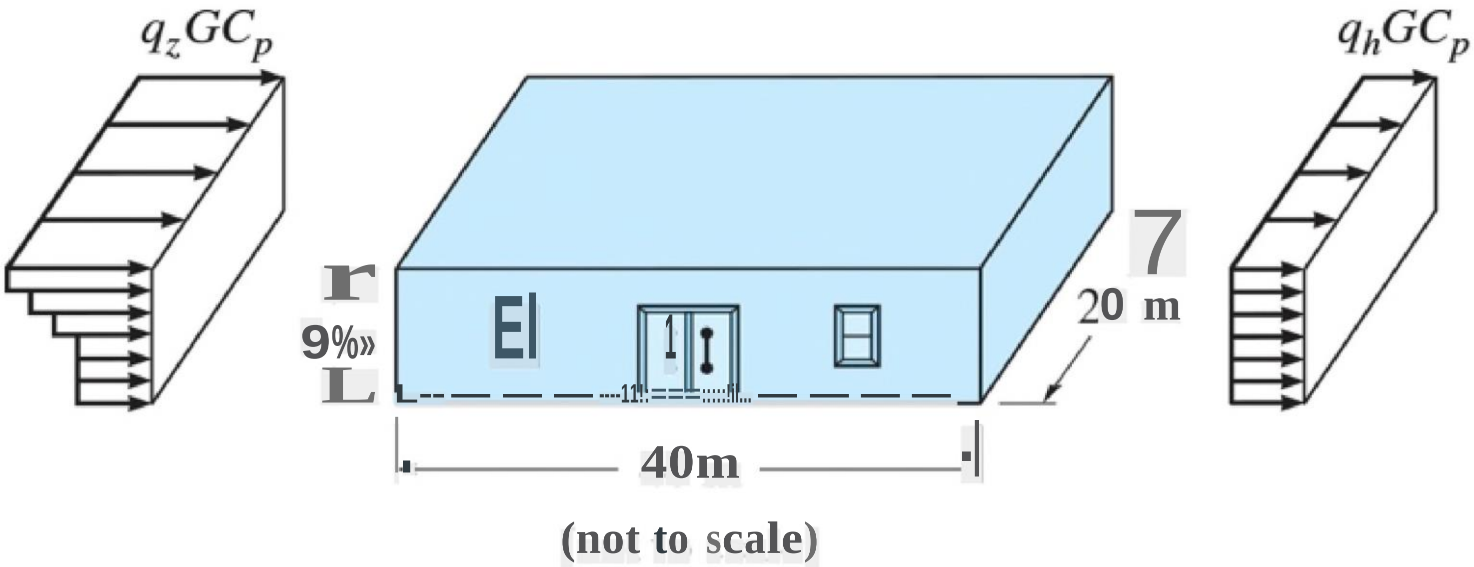
Building Section

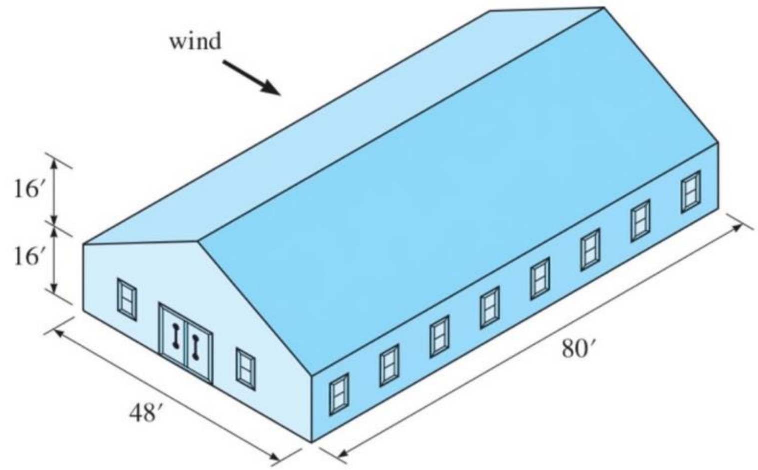

1
2
3
4
A
B
C
D
N
Plan
(a)
20
15
13
Building Section
wind pressures
in lb/ft
(b)
(c)



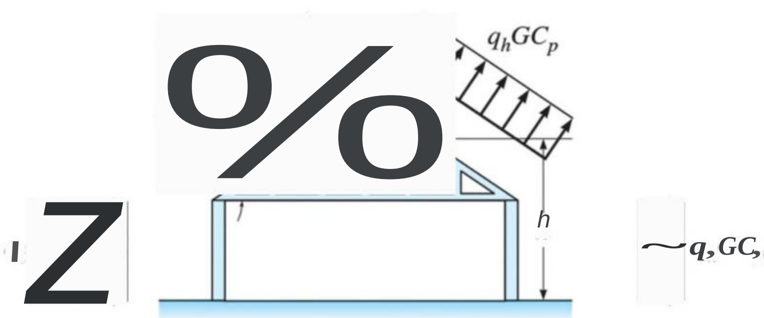
Mechanical Floor Plan
(beams not shown)
(a)

Section
(b)


q_zGC_p
q_hGC_p
20 m
40m
(not to scale)

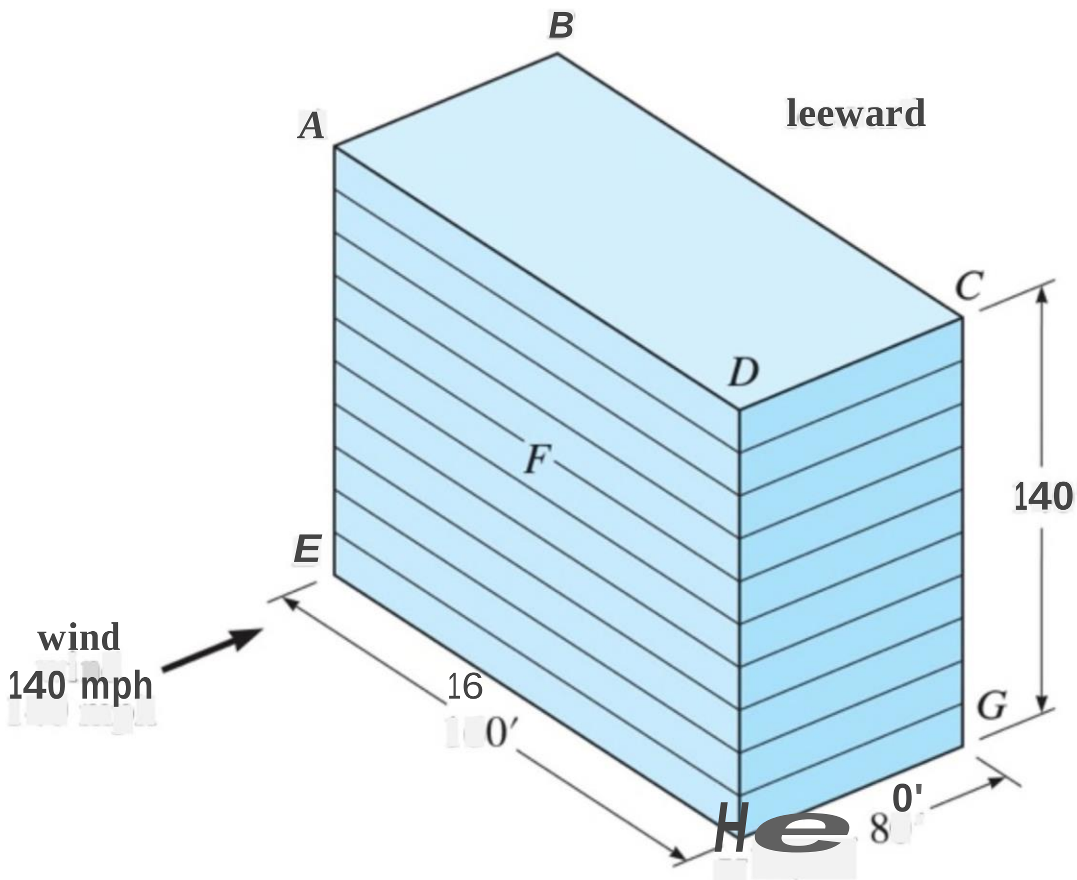


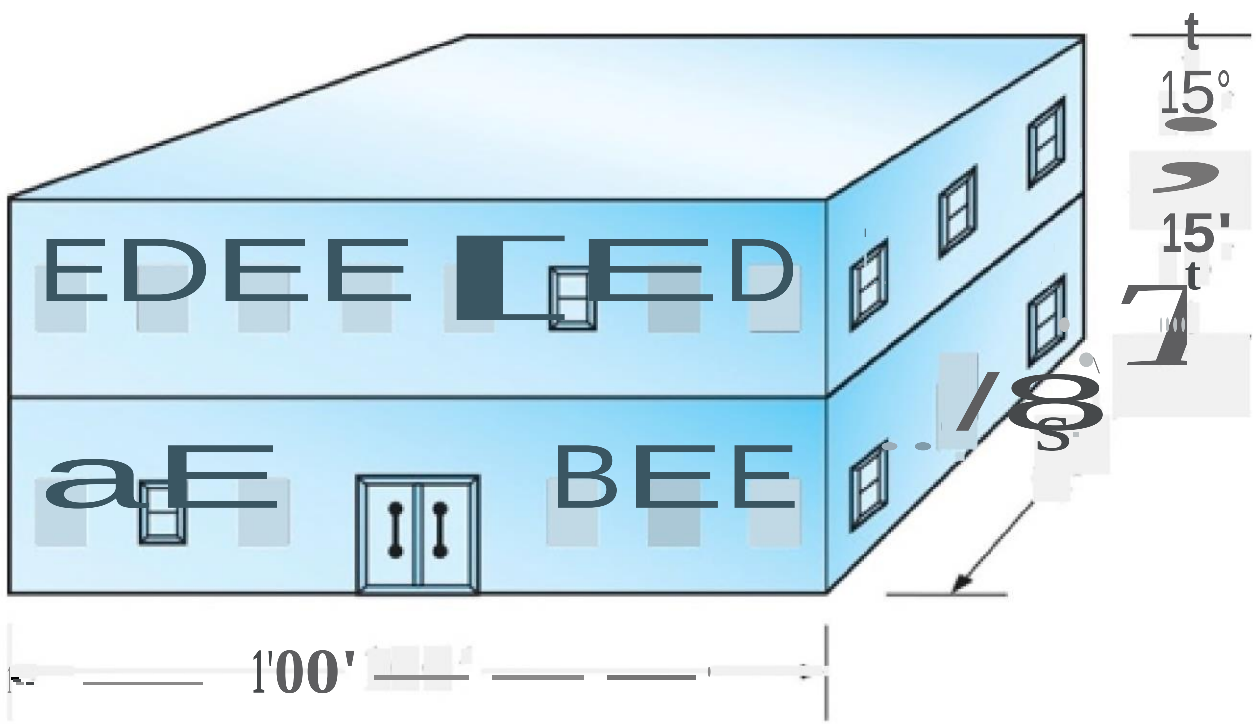
(a)

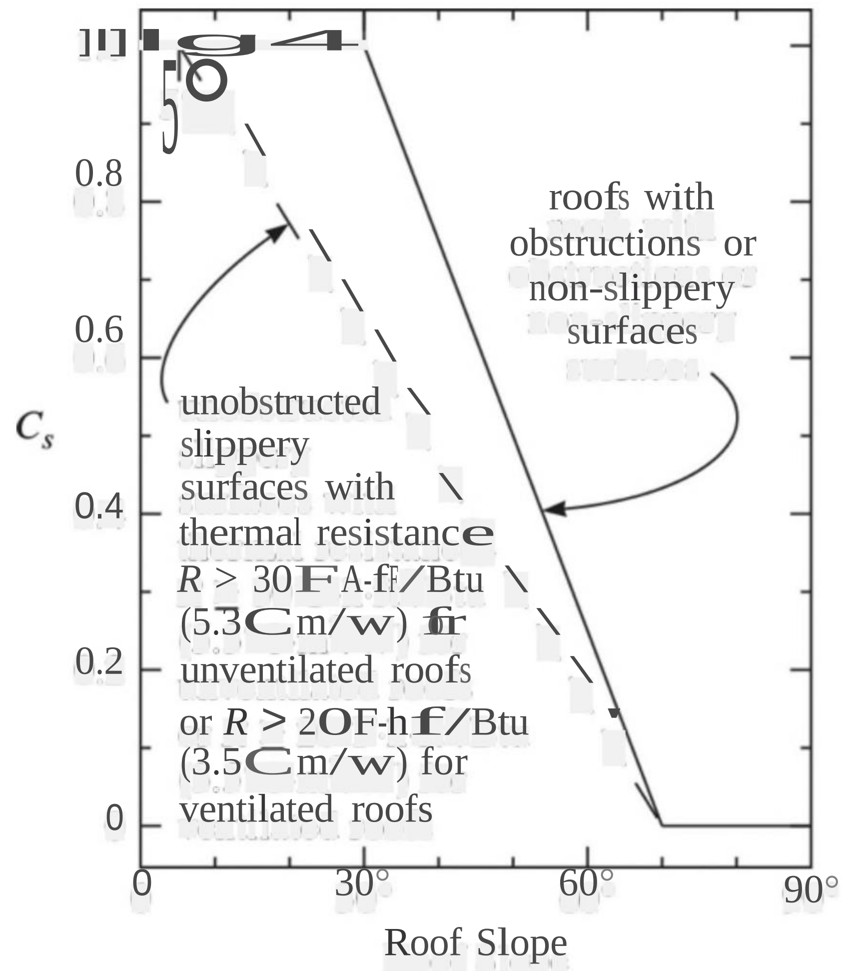


Section

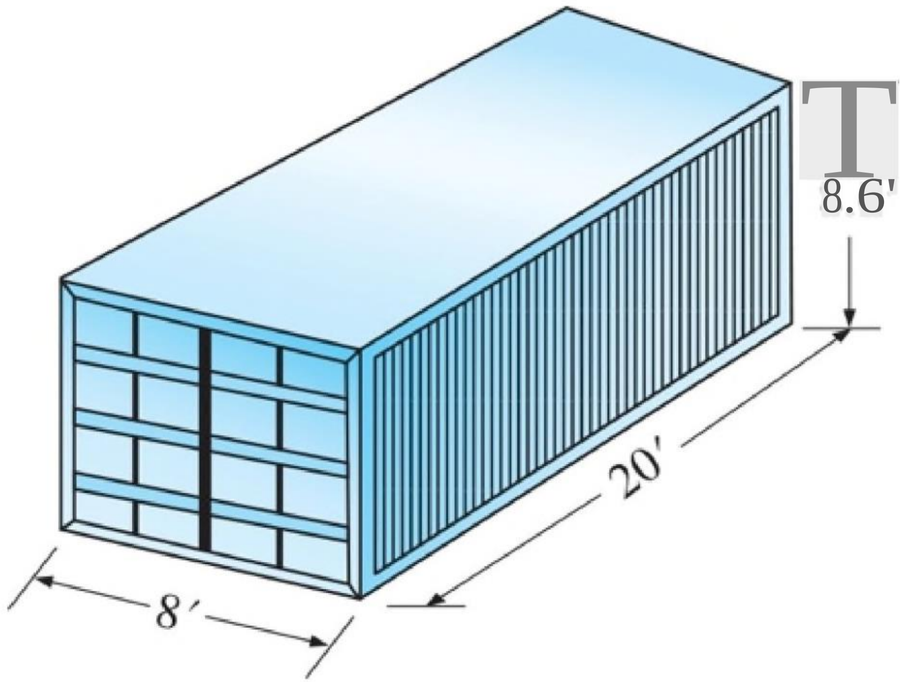
(b)

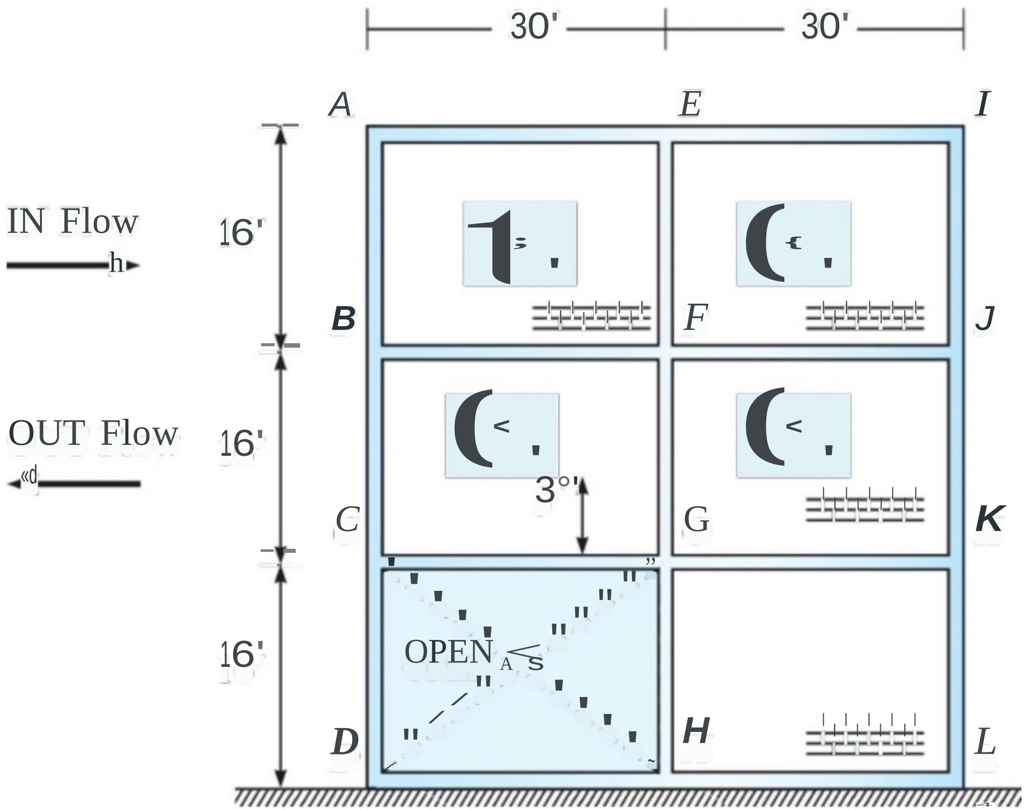

B
A
leeward
C
D
F
140'
E
G
wind
140 mph
16
0′
H
0'
8


15'
15'
100'



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


8.6'
20′
8′



30'
30'
A
E
I
B
F
J
C
G
K
D
H
L
16'
16'
16'
30'
IN Flow
OUT Flow
OPEN