FUNDAMENTALS OF STRUCTURAL ANALYSIS

4th Edition

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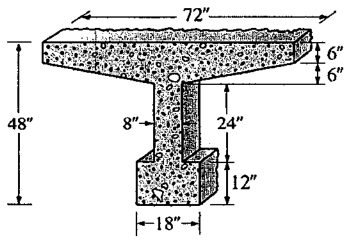
SOLUTIONS MANUAL

CHAPTER 2: DESIGN LOADS

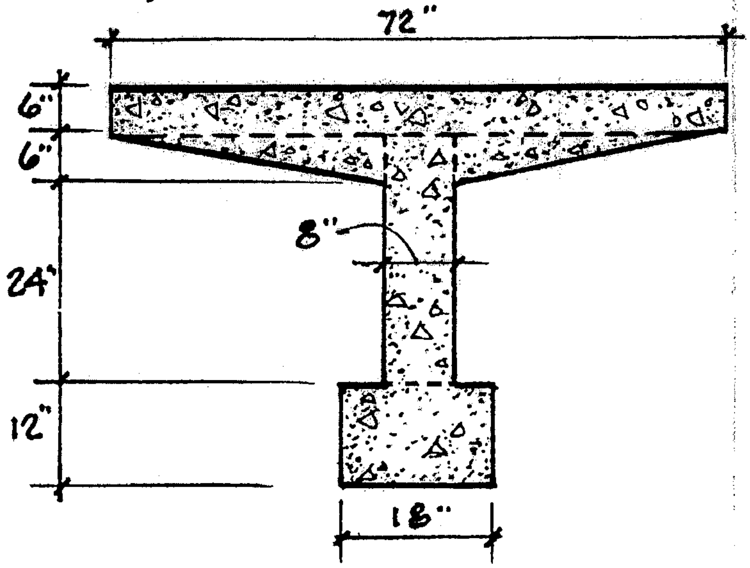
P2.1. Determine the deadweight of a 1-ft-long segment of the prestressed, reinforced concrete tee-beam whose cross section is shown in Figure P2.1. Beam is constructed with lightweight concrete which weighs 120 lbs/ft^3.



Section

P2.1

COMPUTE THE WEIGHT/FT. OF CROSS SECTION
@ 120 LB/FT^3.



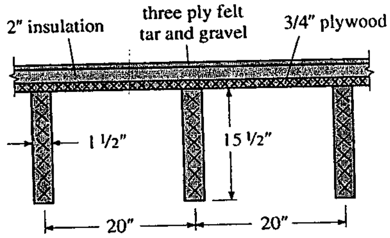
COMPUTE CROSS SECTIONAL AREA:

AREA = $(0.5' \times 6') + 2(\tfrac{1}{2} \times 0.5' \times 2.67') +$
$(0.67' \times 2.5') + (1.5' \times 1')$
$= 7.5\ \text{FT}^2$

WEIGHT OF MEMBER PER FOOT LENGTH:

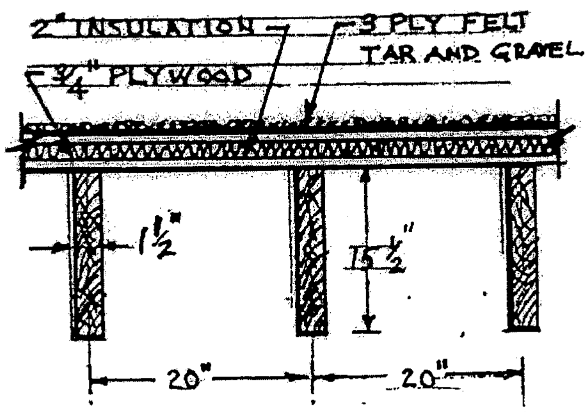
WT/FT = $7.5\ \text{FT}^2 \times 120\ \text{LB/FT}^3 = \underline{900\ \text{LB/FT.}}$

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 in × 16 in southern pine beam (the actual dimensions are $\frac{1}{2}$ in smaller). The $\frac{3}{4}$-in plywood weighs 3 lb/ft^2.



Section

P2.2



SEE TABLE 2.1 FOR WEIGHTS

WT/20" unit

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

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

ROOF & TAR & G: $5.5\text{ psf} \times 20''/12 \times 1' = 9.17\text{ lb}$

19.17 lb

WOOD JOIST $= 37\ \frac{\text{lb}}{\text{ft}^3} \left(\frac{1.5'' \times 15.5''}{144\text{ in}^2/\text{ft}^2}\right) \times 1' = 5.97\text{ lb}$

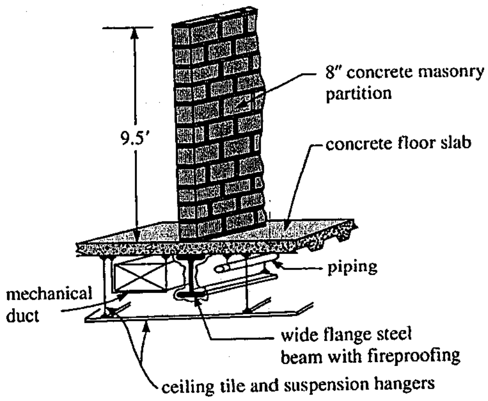
TOTAL WT OF 20" UNIT $= 19.17 + 5.97$

$= 25.14\text{ lb}$ 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 framing (not shown). The wall consists of 8 inch 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.



Section

P2.3

UNIFORM DEAD LOAD W_{DL}
ACTING ON THE WIDE FLANGE BEAM:

WALL LOAD:
9.5'(0.09 ksf) = 0.855 klf

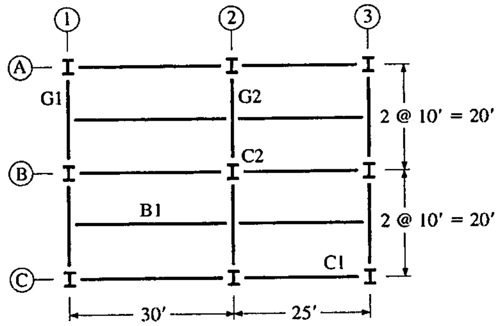
FLOOR SLAB:
10'(0.05 ksf) = 0.50 klf

STEEL FRMG, FIREPROOFG, ARCH'L FEATURES, FLOOR FINISHES, & CEILING:
10'(0.024 ksf) = 0.24 klf

MECH'L, PIPING & ELECTRICAL SYSTEMS:
10'(0.006 ksf) = 0.06 klf

TOTAL W_{DL} = 1.66 klf

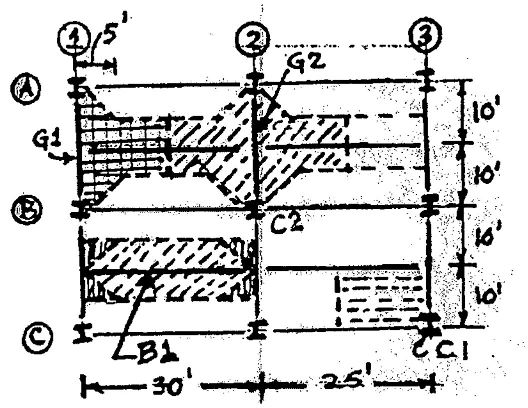
P2.4. Consider the floor plan shown in Figure P2.4. Compute the tributary areas for (*a*) floor beam B1, (*b*) girder G1, (*c*) girder G2, (*d*) corner column C1, and (*e*) interior column C2.



Framing Plan

P2.4

COMPUTE TRIBUTARY AREAS, A_T

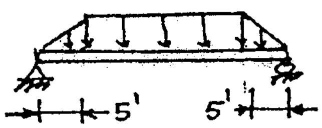


(a) BEAM B1 SPAN 30 ft.

METHOD 1: UNIFORM LOAD OVER 30'

$A_T = 30(5+5) = 300 \text{ FT}^2$

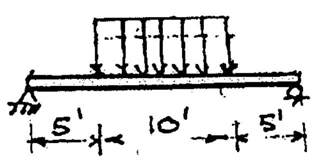
METHOD 2: TAPER LOADS AT ENDS



$A_T = 300 - 4(5 \times 5 \times \tfrac{1}{2}) = 250 \text{ ft}^2$

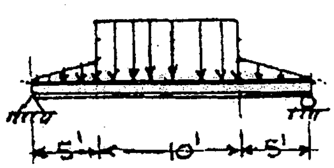
(b) GIRDER G1 SPAN 20 ft.

METHOD 1: UNIFORM LOAD



$A_T = 10 \times 15 = 150 \text{ ft}^2$

METHOD 2: ADD TAPERED LOADS AT ENDS

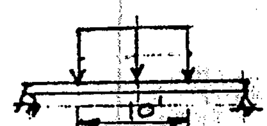


$A_T = 150 + (5 \times 5 \times \tfrac{1}{2})2$

$A_T = 175 \text{ ft}^2$

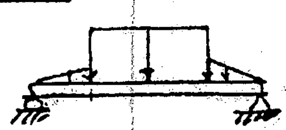
(c) GIRDER G2 SPAN 20 ft

METHOD 1: UNIFORM LOAD



$A_T = (30/2 + 25/2)10 = 275 \text{ ft}^2$

METHOD 2. TAPER LOAD AT ENDS



$A_T = 275 + 4[5 \times 5] \cdot \tfrac{1}{2} = 325 \text{ ft}^2$

(d) COLUMN C1

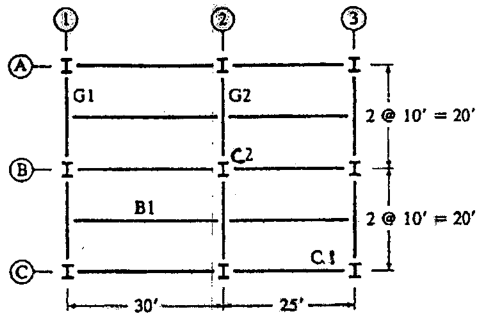
$A_T = 25/2 \times 10 = 125 \text{ ft}^2$

(e) COLUMN C2

$A_T = (15 + 12.5)(10 + 10)$

$A_T = 550 \text{ ft}^2$

P2.5. Refer to Figure P2.4 for the floor plan. Calculate the tributary areas for (*a*) floor beam B1, (*b*) girder G1, (*c*) girder G2, (*d*) corner column C1, and (*e*) interior column C2.



P2.4

Multiply the values of A_T in problem P2.4 by K_{LL}, where K_{LL} = 4 for columns and 2 for beams.

BEAM B1

METHOD 1 $K_{LL}A_T = 2(300\,FT^2) = 600\,FT^2$

METHOD 2 $K_{LL}A_T = 2(500\,FT^2) = 500\,FT^2$

GIRDER G1

METHOD 1 $K_{LL}A_T = 2(150\,FT^2) = 300\,FT^2$

METHOD 2 $K_{LL}A_T = 2(175\,FT^2) = 350\,FT^2$

GIRDER G2

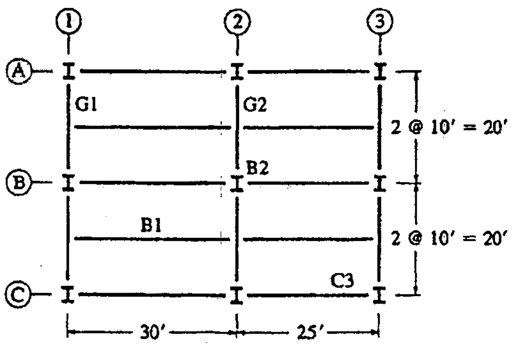
METHOD 1 $K_{LL}A_T = 2(275\,FT^2) = 550\,FT^2$

METHOD 2 $K_{LL}A_T = 2(325\,FT^2) = 650\,FT^2$

COLUMN C1 $K_{LL}A_T = 4(125\,FT^2) = 500\,FT^2$

COLUMN C2 $K_{LL}A_T = 4(550\,FT^2) = 2{,}200\,FT^2$

P2.6. 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 B1, (*b*) girder G1, and (*c*) girder G2. Consider the live load reduction if permitted by the ASCE standard.



P2.4

Values of A_T are evaluated in Prob 2.4. Use simplified loading.

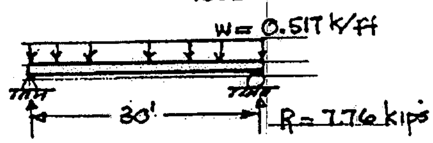
(a) Loading for B1

$K_{LL} A_T = 2 \times 300 = 600 > 400$

$L = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$

$= 60\left(0.25 + \frac{15}{\sqrt{600}}\right) = 51.7\ \text{lb/ft}^2$

Live Load/ft $= \frac{51.7 \times 10}{1000} = 0.517\ \text{k/ft}$

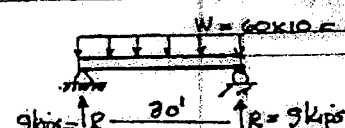


(b) Loading for G1 supports load from a single 30 ft beam (B1).

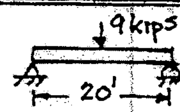
$K_{LL} A_T = 2 \times 150 = 300 < 400$

∴ No reduction allowed.

Compute reaction from beam B1



Load to girder G1 from B1

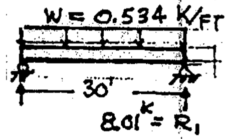


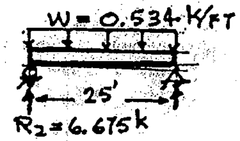
Establish load for G2 which supports beams of 25′ and 30′ at its center.

$K_{LL} A_T = 2(10)\left[30/2 + 25/2\right] = 550\ \text{ft}^2 > 400$

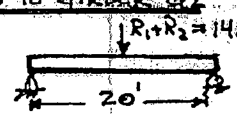
$L = 60\left(0.25 + \frac{15}{\sqrt{550}}\right) = 53.4\ \text{lb/ft}^2$

Live Load/ft $= \frac{53.4 \times 10}{1000} = 0.534\ \text{kips/ft}$

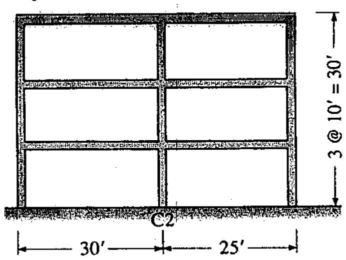




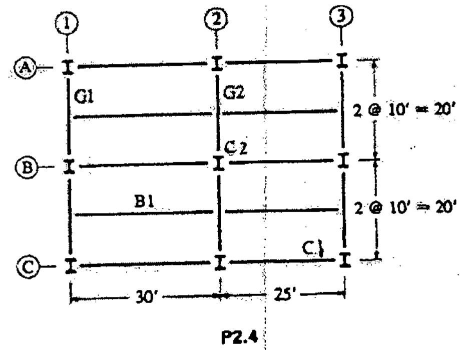
(c) Load to girder G2



P2.7. The building section associated with the floor plan in Figure P2.4 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.



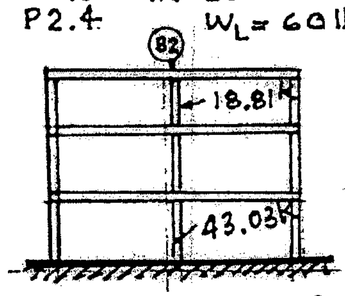
Building Section



P2.4

P2.7

COMPUTE THE LIVE LOAD FORCES IN COLUMN C2 IN P2.4 $\quad W_L = 60\ lb/FT^2$



$A_T = (\frac{30}{2}+\frac{25}{2})20 = 550\ ft^2$

$K_{LL} A_T = 4 \times 550 = 2{,}200\ ft^2 > 400\ ft^2$

$\therefore$ Reduce W_L

3RD Story

$W = W_L\left[0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right]$

$= 60\left[0.25 + \frac{15}{\sqrt{2200}}\right] = 34.19\ psf$

SINCE $34.19 > 0.5 W_L = 30$, USE $34.19\ psf$

$P = WA_T = \frac{34.19(550)}{1000} = \underline{18.81\ kips}$

1ST Story

COLUMN supports 3 FLOORS

$K_{LL} A_T = 4(550 \times 3) = 6600\ ft^2$

$\therefore$ Reduce W_L

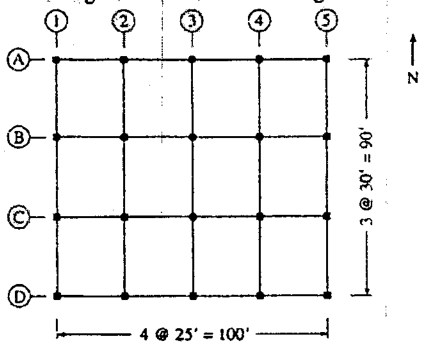
$W = 60\left[0.25 + \underbrace{\frac{15}{\sqrt{6600}}}_{}^{(.4396)}\right] = \underline{26.08\ psf}$

SINCE $26.08 > .4(60)$, Use $26.08\ psf$

$P = WA_T = 26.08(3 \times 550)$

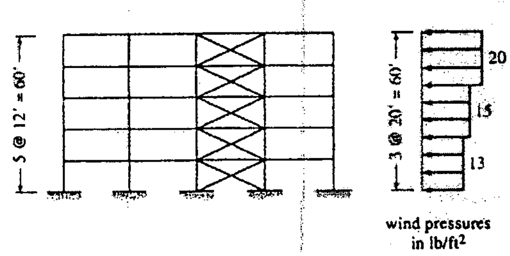
$= \underline{43.03\ kips}$

P2.8. A five-story building is shown in Figure P2.8. Following the ASCE standard, the wind pressure along the height on the windward side has been established as shown in Figure P2.8(*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.



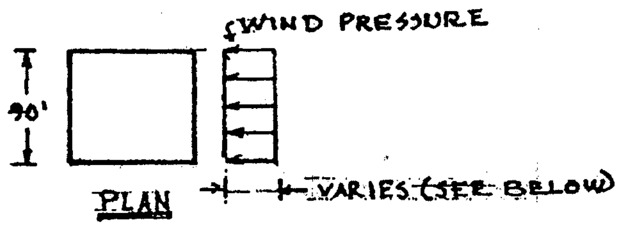
(*a*)

P2.8

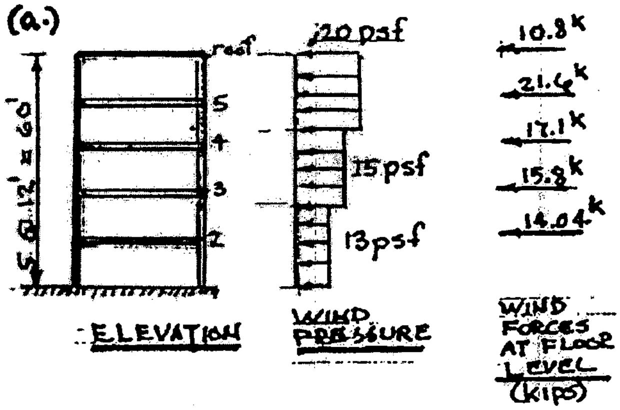


(*b*) (*c*)

P2.8



(a.)



a.) Resulant Wind Forces

Roof	20 psf (6 × 90) = 10,800 lb
5th floor	20 psf (12 × 90) = 21,600 lb
4th floor	20 psf (2 × 90) + 15 (10 × 90) = 17,100 lb
3rd floor	15 psf (10 × 90) + 13 (2 × 90) = 15,800 lb
2nd floor	13 psf (12 × 90) = 14,040 lb

b) HORIZONTAL BASE SHEAR V_{BASE} = Σ FORCES AT EACH LEVEL =

$10.8^k + 21.6^k + 17.1^k + 15.8^k + 14.04^k =$

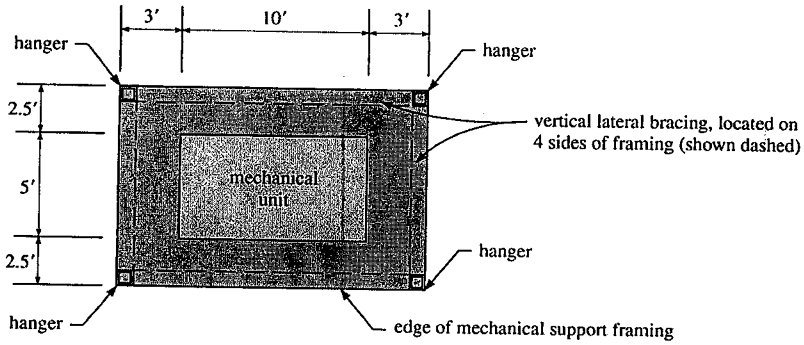
$V_{BASE} = \underline{79.34^k}$

OVERTURNING MOMENT OF THE BUILDING = Σ(FORCE @ EA. LEVEL × HEIGHT ABOVE BASE)

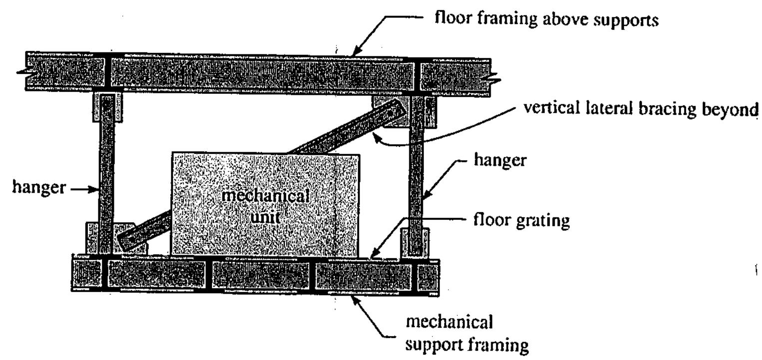
$10.8^k(60') + 21.6(48') + 17.1(36') + 15.8^k(24') + 14.04^k(12') =$

$M_{OVERTURNING} = \underline{2,848^{ft\cdot k}}$

P2.9. A mechanical support framing system is shown in Figure P2.9. 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 4,000 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 4 edges of the mechanical floor framing for stability and transfer of lateral loads.



Mechanical Floor Plan
(beams not shown)
(*a*)



Section
(*b*)

P2.9

a) LIVE LOAD IMPACT FACTOR = 20%

b.) TOTAL LL

MACHINERY = 1.20(4 KIPS) = 4.8^k

UNIFORM LL = ((10'x16')-(5'x10'))(0.04KSF) = 4.4^k

TOTAL LL = 9.2^k

∴ TOTAL LL ACTING ON ONE HANGER = 9.2^k/4 HANGERS = 2.3 KIPS

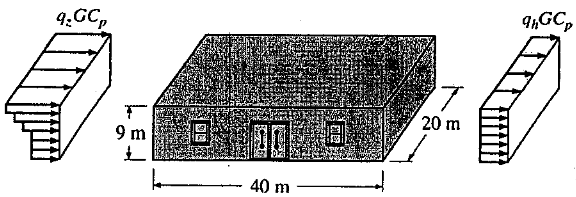
c.) TOTAL DL

FLOOR FRAMING = 10'x16' (0.025 KSF) = 4^k

∴ TOTAL DL ACTING ON ONE HANGER = 4^k/4 HANGERS = 1 KIP

∴ TOTAL DL+LL ON ONE HANGER = $2.3^k + 1^k$ = 3.3 KIPS

P2.10. The dimensions of a 9-m-high warehouse are shown in Figure P2.10. 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?



(not to scale)

P2.10

USE I = 1

$q_s = 0.613V^2$ (EQ 2.4b)

$= 0.613(40)^2 = 980.8$ N/m²

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

$q_z = 980.8(1)(K_z)(1)(0.85) = 833.7K_z$

0 – 4.6 m: $q_z = 833.7(0.85) = 708.6$ N/m²

4.6 – 6.1 m: $q_z = 833.7(0.90) = 750.3$ N/m²

6.1 – 7.6 m: $q_z = 833.7(0.94) = 783.7$ N/m²

7.6 – 9 m: $q_z = 833.7(0.98) = 817.1$ N/m²

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 – 4.6 m p = 481.8 N/m²

4.6 – 6.1 m p = 510.2 N/m²

6.1 – 7.6 m p = 532.9 N/m²

7.6 – 9 m p = 555.6 N/m²

TOTAL WIND FORCE, F_W, WINDWARD WALL

$F_W = 481.8[4.6 \times 20] + 510.2[1.5 \times 20]$
$+ 532.9[1.5 \times 20] + 555.6[1.4 \times 20]$

$F_W = 91{,}180$ N

FOR LEEWARD WALL

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

$q_h = q_z$ at 9 m = 817.1 N/m² (above)

$p = 817.1(0.85)(-0.2) = -138.9$ N/m²

TOTAL WIND FORCE, F_L, ON LEEWARD WALL

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

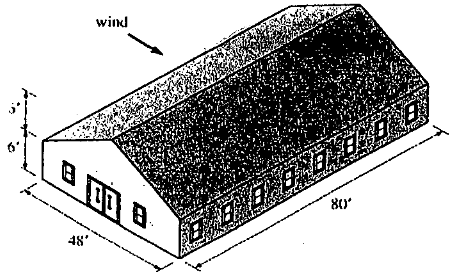
TOTAL FORCE = $F_W + F_L$

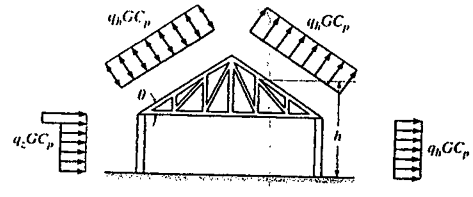
= 91,180 N + 25,003

= 116,183.3 N

*BOTH F_L AND F_W ACT IN SAME DIRECTION.

P2.11. The dimensions of an enclosed gabled building are shown in Figure P2.11*a*. The external pressures for the wind load perpendicular to the ridge of the building are shown in Figure P2.11*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 P2.11, 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.





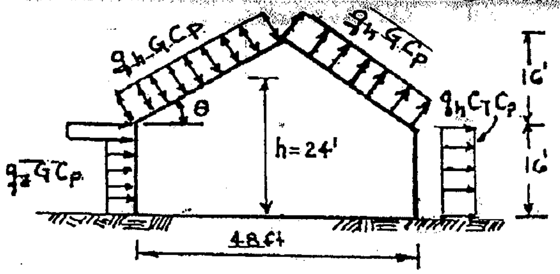
Section
(*b*)

P2.11

TABLE P2.11
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				



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

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

$C_p = 0.2738$ (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'

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

$q_s = 0.00256V^2$ (EQ 2.4a)

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

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

0−15'; $q_z = 25.6(1)(0.57)(1)(0.85)$
$= \underline{12.40 \text{ lb/ft}^2}$

15−16'; $q_z = 13.49 \text{ lb/ft}^2$

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

WIND PRESSURE ON WINDWARD WALL & ROOF

$p = q_z G C_p$

WALL, 0−15' $p = 12.40 \times 0.85 \times 0.80$
$p = \underline{8.43 \text{ psf}}$

WALL, 15'−16' $p = 13.49 \times 0.85 \times 0.8 = \underline{9.17 \text{ psf}}$

ROOF, $p = 14.36 \times 0.85 \times 0.2738$
$p = \underline{3.34 \text{ psf}}$

WIND PRESSURE ON LEEWARD SIDE

FOR WALL $p = q_h G C_p$

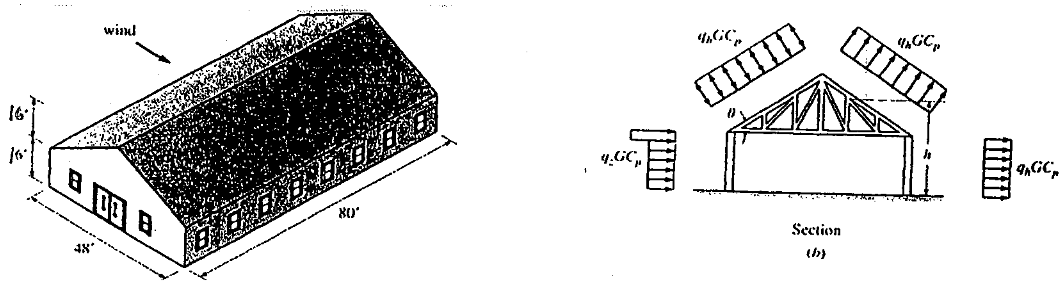
FOR $h = 24'$; $q_h = q_z = 14.36 \text{ lb/ft}^2$

$C_p = -0.2$ FOR WALL $= 0.6$ FOR ROOF

FOR WALL $p = 14.36(0.85)(-0.2)$
$p = \underline{-2.44 \text{ lb/ft}^2}$

FOR ROOF $p = 14.36(0.85)(-0.6)$
$= \underline{-7.32 \text{ lb/ft}^2}$ (uplift)

P2.12. Establish the wind pressures on the building in Problem P2.11 when the windward roof is subjected to an uplift wind force.



P2.11

TABLE P2.11
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~~ SEE P2.11 SOLUTION

WINDWARD ROOF (NEGATIVE PRESSURE)

$\theta = 33.7°$

Interpolate between 30° and 35° for negative C_p value in Table P2.10

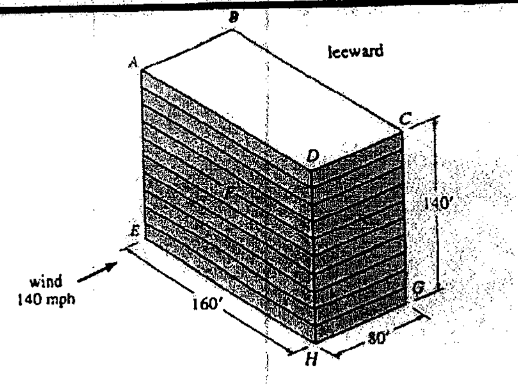
$$C_p = -0.274$$

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

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

NOTE: Wind pressures on other 3 surfaces are the same as in P2.11

P2.13. (*a*) Determine the wind pressure distribution on the four sides of the 10-story hospital shown in Figure P2.13. The building is located near the Georgia coast where the wind velocity contour map in Figure 2.15 of the text 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.



P2.13

(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

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

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

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

$q_z = 49.05 K_z$

COMPUTE WIND PRESSURE "p" ON WINDWARD FACE

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

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

$C_p = 0.8$ on windward side

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

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

$p = q_z G C_p$; Use VALUE OF q_z AT 140 FT. i.e. $K_z = 1.52$

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

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

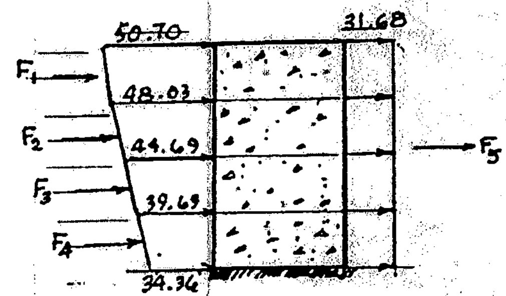
$p = \underline{-31.68 \text{ psf}}$ ANS.

WIND PRESSURE ON SIDE WALLS

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

$p = -44.36$ psf

(b.) VARIATION OF WIND PRESSURE ON WINDWARD AND LEEWARD SIDES



WIND PRESSURES (psf)

COMPUTE TOTAL WIND FORCE (kips)

$F_1 = \frac{50.7 + 48.03}{2}\left[\frac{35 \times 160}{1000}\right] = 276.42$ kips

$F_2 = \frac{48.03 + 44.69}{2}\left[\frac{35 \times 160}{1000}\right] = 259.62$ k

$F_3 = \frac{44.69 + 39.69}{2}\left[\frac{35 \times 160}{1000}\right] = 236.26$ k

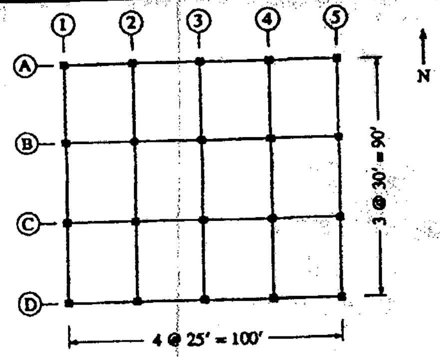
$F_4 = \frac{39.69 + 34.36}{2}\left[\frac{35 \times 160}{1000}\right] = 207.34$ k

$F_5 = \frac{31.68(140 \times 160)}{1000} = 709.63$ k

TOTAL WIND FORCE $= \Sigma\, F_1 + F_2 + F_3 + F_4 + F_5$

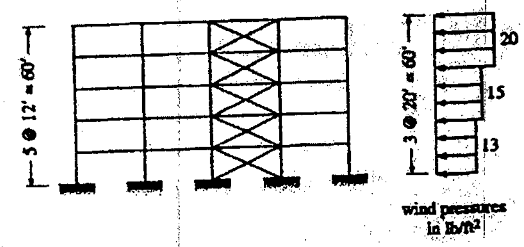
$= \underline{1689.27 \text{ Kips}}$

P2.14. Consider the five-story building shown in Figure P2.8. 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 $0.9g$ 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.



(a)

P2.8



(b) (c)

P2.8

FUNDAMENTAL PERIOD

$T = C_t h_n^{3/4}$ $\boxed{C_t = 0.035 \text{ for steel moment frames}}$

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

$T = 0.75$ SEC.

$W = 4(100 \times 90)\,90\ \text{lb/ft}^2 + (100 \times 90)\,70\ \text{lb/ft}^2$
$= 3{,}870{,}000\ \text{lbs} = 3{,}870\ \text{kips}$

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

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

$V_{max} = \dfrac{S_{DS} W}{R/I} = \dfrac{0.9(3870)}{8/1}$
$= 435\ \text{kips}$

$V_{min} = 0.0441\, I\, S_{DS} W$
$= 0.0441(1)(0.9)(3870)$
$= 153.6\ \text{kips}$

THEREFORE, USE $V = 258$ kips

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

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

FORCES AT EACH FLOOR LEVEL

FLOOR	WEIGHT W_i, (kips)	FLOOR HEIGHT h_i (ft)	$w_i h_i^k$	$\dfrac{W_x h_x^k}{\sum W_i h_i^k}$	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
	$\Sigma = 3{,}870$		$\Sigma = 213{,}961$		$\Sigma = 258$

P2.15. 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 P2.14. Which method produces a larger seismic base shear?

ASCE APPROXIMATE FUNDAMENTAL PERIOD:

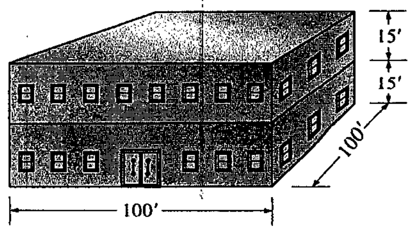
$T = 0.1N$

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

P2.16. (*a*) A two-story hospital facility shown in Figure P2.16 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 $K_z = 1.0$. 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/ft² for both the floor and roof, is to be designed for the following seismic factors: $S_{DS} = 0.27g$ and $S_{D1} = 0.06g$; 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?



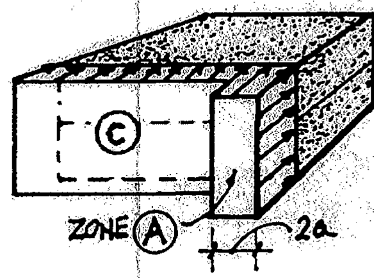
P2.16

(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}	$P_s = 1.66(1)1.15 P_{s30} = 1.909 P_{s30}$
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 : $\frac{15'}{2}(24.44\,\text{PSF})\,20'/1000 = 3.67^k$

ZONE C : $\frac{15'}{2}(16.3\,\text{PSF})\,80'/1000 = 9.78^k$

F_{ROOF} RESULTANT $= 13.45^k$

F_{2ND}: ZONE A : $15'(24.44\,\text{PSF})\,20'/1000 = 7.33^k$

ZONE C : $15'(16.3\,\text{PSF})\,80'/1000 = 19.56^k$

F_{2ND} RESULTANT $= 26.89^k$

BASE SHEAR $V_{BASE} = F_{ROOF} + F_{2ND} = \underline{40.34^k}$

OVERTURNING MOMENT $M_{O.T.} = \Sigma F_i h_i$

$M_{O.T.} = 13.45^k(30') + 26.89^k(15') = \underline{806.9^{\text{FT}\cdot\text{K}}}$

P2.16 CONTINUED

P2.16 CONTINUED

(b) SEISMIC LOADS BY EQUIVALENT LATERAL FORCE PROCEDURE

GIVEN: $w = 90$ PSF FLOOR & ROOF; $S_{DS} = 0.27g$, $S_{D1} = 0.06g$; $R=8$, $I=1.5$

BASE SHEAR $V_{BASE} = \dfrac{S_{D1} W}{T(R/I)}$

WHERE W TOTAL BUILDING DEAD LOAD:

$W_{ROOF} = 90$ PSF $(100')^2 = 900^k$

$W_{2ND} = 90$ PSF $(100')^2 = 900^k$

$W_{TOTAL} = 1800^k$

AND $T = C_T h_n^x = 0.342$ SEC.

$C_T = 0.016$ REINF. CONCRETE FRAME

$x = 0.9$ " " "

$h = 30'$ BUILDING HEIGHT

$V_{BASE} = \dfrac{0.06(1800^k)}{(0.342\text{sec})(8/1.5)} = 0.033W = \underline{59.2^k}$ CONTROLS

$V_{MAX.} = \dfrac{S_{DS} W}{R/I} = \dfrac{0.27(1800^k)}{(8/1.5)} = 0.051W = 91.1^k$

$V_{MIN.} = 0.044 S_{DS} I W = 0.044(0.27)(1.5)(1800^k)$
$= 0.0178W = 32.1^k$

FORCE @ EACH LEVEL $F_x = \dfrac{W_x h_x^k}{\Sigma W_i h_i^k} V_{BASE}$, WHERE $V_{BASE} = 59.2^k$

$T < 0.5$ SEC. THUS $k = 1.0$

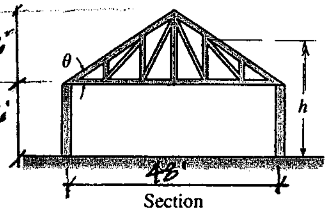
LEVEL	W_i	h_i	$W_i h_i^k$	$W_x h_x^k / \Sigma W_i h_i^k$	FORCE @ EA. LEVEL:
ROOF	900^k	$30'$	27000	0.667	$F_{ROOF} = 39.5^k$
2ND	900^k	$15'$	13500	0.333	$F_{2ND} = 19.76^k$
			$\Sigma W_i h_i^k = 40500$		$\Sigma F_x = V_{BASE} = 59.2^k$

OVERTURNING MOMENT $M_{O.T.} = \Sigma F_x h_i$

$M_{O.T.} = 39.5^k(30') + 19.76^k(15') = \underline{1,481.4 \text{ FT·K}}$

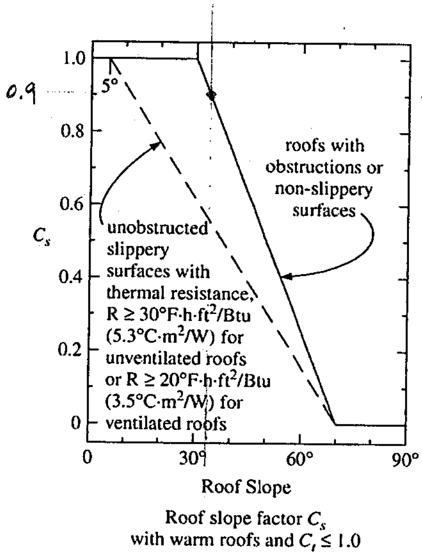
(c) SEISMIC FORCES GOVERN THE LATERAL STRENGTH DESIGN.

P2.17. In the gabled roof structure shown in Figure P2.11, 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 P2.17. If roof trusses are spaced at 16 ft on center, what is the uniform snow load along a truss?



Section
(b)

P2.11



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

P2.17

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)

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

$P_s = C_s P_s = 0.9(19.6\text{ PSF}) = \underline{17.64\text{ PSF}}$

UNIFORM LOAD ACTING ON TRUSSES SPACED @ 16' O.C.

$W_{SNOW} = 17.64\text{ PSF}(16') = \underline{282.2\text{ PLF}}$

P2.18. 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

LOAD COMBINATIONS - FACTORED STRENGTH

END MOMENTS

$1.4DL = 1.4(-180\ FT\cdot K)$ $= -252\ FT\cdot K$

$1.2\ DL + 1.6LL + 0.5(\cancel{L_R\ or\ S})^{0} = 1.2(-180) + 1.6(-150)$ $= -456\ FT\cdot K$ *

$1.2\ DL \pm 1.0E + LL + 0.2(\cancel{S})^{0} = 1.2(-180) + (-80) + (-150)$ $= -446\ FT\cdot K$

MID-SPAN MOMENTS

$1.4DL = 1.4(90\ FT\cdot K)$ $= +126\ FT\cdot K$

$1.2\ DL + 1.6LL + 0.5(\cancel{L_R\ or\ S})^{0} = 1.2(90) + 1.6(+150)$ $= +348\ FT\cdot K$ *

$1.2\ DL \pm 1.0E + LL + 0.2(\cancel{S})^{0} = 1.2(90) + 0 + (150)$ $= +258\ FT\cdot K$

BEAM NEEDS TO BE DESIGNED FOR MAX. END MOMENT $= -456\ FT\cdot K$

MAX. MID-SPAN MOMENT $= +348\ FT\cdot K$