

For the loads specified, compute the maximum combined load using the ASCE 7-16 load combinations for allowable stress design.

D = 50 psf, Lr = 75 psf, R = 8 psf, S = 20 psf

Since Lr is larger than R and S it will control.

No seismic effects exist so load cases 8, 9, & 10 are not required.

1. D	50 psf
2. D + L	$50psf + 0psf = \underline{50psf}$
3. D + Lr	$50psf + 75psf = \underline{125psf}$
4. D + 0.75L + 0.75Lr	$50psf + 0.75(0psf) + 0.75(75psf) = \underline{106.3psf}$
5. D + 0.6W	$50psf + 0.7(0psf) = \underline{50psf}$
6. D + 0.75L + 0.75(0.6W) + 0.75Lr	$50psf + 0.75(0psf) + 0.75(0.6)(0psf) + 0.75(75psf) = \underline{106.3psf}$
7. 0.6D + 0.6W	$0.6(50psf) + 0.6(0psf) = \underline{30psf}$

Load combination 3 controls with a total load of 125 psf.

P2.2

For the loads specified, compute the maximum combined load using the ASCE 7-16 load combinations for allowable stress design.

D = 45 psf, L = 60 psf

No seismic effects exist so load cases 8, 9, & 10 are not required.

1. D	45 psf
2. D + L	$45psf + 60psf = \underline{100psf}$
3. D + (Lr or S or R)	$45psf + 0psf = \underline{45psf}$
4. D + 0.75L + 0.75(Lr or S or R)	$45psf + 0.75(60psf) + 0.75(0psf) = \underline{90psf}$
5. D + 0.6W	$45psf + 0.6(0psf) = \underline{45psf}$
6. D + 0.75L + 0.75(0.6W) + 0.75(Lr or S or R)	$45psf + 0.75(60psf) + 0.75(0.6)(0psf) + 0.75(0psf) = \underline{90psf}$
7. 0.6D + 0.6W	$0.6(45psf) + 0.6(0psf) = \underline{27psf}$

Load combination 2 controls with a total load of 100 psf.

P2.3

For the loads specified, compute the maximum combined load using the ASCE 7-16 load combinations for allowable stress design.

$D = 2750 \text{ lb}$, $L = 4500 \text{ lb}$, $L_r = 1500 \text{ lb}$, $R = 1250 \text{ lb}$, $S = 1000 \text{ lb}$

L_r is larger than R and S so it will control

No seismic effects exist so load cases 8, 9, & 10 are not required.

1. D	2750 lb
2. $D + L$	$2750 \text{ lb} + 4500 \text{ lb} = \underline{7250 \text{ lb}}$
3. $D + L_r$	$2750 \text{ lb} + 1500 \text{ lb} = \underline{4250 \text{ lb}}$
4. $D + 0.75L + 0.75L_r$	$2750 \text{ lb} + 0.75(4500 \text{ lb}) + 0.75(1500 \text{ lb}) = \underline{7250 \text{ lb}}$
5. $D + 0.6W$	$2750 \text{ lb} + 0.6(0 \text{ lb}) = \underline{2750 \text{ lb}}$
6. $D + 0.75L + 0.75(0.6W) + 0.75L_r$	$2750 \text{ lb} + 0.75(4500 \text{ lb}) + 0.75(0.6)(0 \text{ lb}) + 0.75(1500 \text{ lb}) = \underline{7250 \text{ lb}}$
7. $0.6D + 0.6W$	$0.6(2750 \text{ lb}) + 0.6(0 \text{ lb}) = \underline{1650 \text{ lb}}$

Load combinations 2, 4 and 6 all produce a maximum load of 7250 lb.

P2.4

For the loads specified, compute the maximum combined load using the ASCE 7-16 load combinations for allowable stress design.

D = 87 psf, L = 150 psf

No seismic effects exist so load cases 8, 9, & 10 are not required.

1. D	$87psf$
2. D + L	$87psf + 150psf = \underline{237psf}$
3. D + (Lr or S or R)	$87psf + 0psf = \underline{87psf}$
4. D + 0.75L + 0.75(Lr or S or R)	$87psf + 0.75(150psf) + 0.75(0psf) = \underline{199.5psf}$
5. D + 0.6W	$87psf + 0.6(0psf) = \underline{87psf}$
6. D + 0.75L + 0.75(0.6W) + 0.75(Lr or S or R)	$87psf + 0.75(150psf) + 0.75(0.6)(0psf) + 0.75(0psf) = \underline{199.5psf}$
7. 0.6D + 0.6W	$0.6(87psf) + 0.6(0psf) = \underline{52.2psf}$

Load combination 2 controls with a total load of 237 psf.

P2.5

For the loads specified, compute the maximum combined load using the ASCE 7-16 load combinations for allowable stress design.

D = 75 psf, Lr = 35 psf, R = 12 psf

Since Lr is larger than R it will control.

No seismic effects exist so load cases 8, 9, & 10 are not required.

1. D	$75psf$
2. D + L	$75psf + 0psf = \underline{75psf}$
3. D + Lr	$75psf + 35psf = \underline{110psf}$
4. D + 0.75L + 0.75Lr	$75psf + 0.75(0psf) + 0.75(35psf) = \underline{101.3psf}$
5. D + 0.6W	$75psf + 0.7(0psf) = \underline{75psf}$
6. D + 0.75L + 0.75(0.6W) + 0.75Lr	$75psf + 0.75(0psf) + 0.75(0.6)(0psf) + 0.75(35psf) = \underline{101.3psf}$
7. 0.6D + 0.6W	$0.6(75psf) + 0.6(0psf) = \underline{45psf}$

Load combination 3 controls with a total load of 110 psf.

P2.6

For the loads specified, compute the maximum combined load using the ASCE 7-16 load combinations for allowable stress design.

$D = 13 \text{ k}$, $L = 32 \text{ k}$, $W = -22 \text{ k}$, $(E_v + E_h) = 16 \text{ k}$, $(-E_v + E_h) = -16 \text{ k}$ Because wind and seismic have different factors and reversible loads, it is not possible to say which one controls in all cases. Calculate for both

1. D	$13k$
2. D + L	$13k + 32k = \underline{45k}$
3. D + (Lr or S or R)	$13k + 0k = \underline{13k}$
4. D + 0.75L + 0.75(Lr or S or R)	$13k + 0.75(32k) + 0.75(0k) = \underline{24k}$
5. D + 0.6W	$13k + 0.6(-22k) = \underline{-0.2k}$
6. D + 0.75L + 0.75(0.6W) + 0.75(Lr or S or R)	$13k + 0.75(32k) + 0.75(0.6)(-22k) + 0.75(0k) = \underline{27.1k}$
7. 0.6D + 0.6W	$0.6(13k) + 0.6(-22k) = \underline{-5.4k}$
8. D + 0.7($E_v + E_h$)	$13k + 0.7(16k) = \underline{24.2k}$
9. D + 0.75L + 0.525($E_v + E_h$) + 0.75(Lr)	$13k + 0.75(32k) + (0.525)(16k) + 0.75(0k) = \underline{45.4k}$
10. 0.6D + 0.7($-E_v + E_h$)	$0.6(13k) + 0.7(-16k) = \underline{-3.4k}$

Load combination 9 provides the maximum positive value of 45.4 k while load combination 7 produces the maximum negative value of -5.4 k.

P2.7

For the loads specified, compute the maximum combined load using the ASCE 7-16 load combinations for strength design.

D = 50 psf, L_r = 75 psf, R = 8 psf, S = 20 psf

Since L_r is larger than R and S it will control.

No seismic effects exist so load cases 6 & 7 are not required.

1. 1.4D	$1.4(50psf) = \underline{70psf}$
2. 1.2D + 1.6L + 0.5L _r	$1.2(50psf) + 1.6(0psf) + 0.5(75psf) = \underline{97.5psf}$
3. 1.2D + 1.6L _r + 0.5L	$1.2(50psf) + 1.6(75psf) + 0.5(0psf) = \underline{180psf}$
4. 1.2D + 1.0W + 0.5L + 0.75L _r	$1.2(50psf) + 1.0(0psf) + 0.5(0psf) + 0.5(75psf) = \underline{97.5psf}$
5. 0.9D + 1.0W	$0.9(50psf) + 1.0(0psf) = \underline{45psf}$

Load combination 3 controls with a total load of 180 psf.

P2.8

For the loads specified, compute the maximum combined load using the ASCE 7-16 load combinations for strength design.

D = 45 psf, L = 60 psf

equations 3 and 4 should be 0.5 and not 1.0.

Since $L < 100$ psf then the factors in front of the L term in

No seismic effects exist so load cases 6 & 7 are not required.

1. $1.4D$	$1.4(45psf) = \underline{63psf}$
2. $1.2D + 1.6L + 0.5Lr$	$1.2(45psf) + 1.6(60psf) + 0.5(0psf) = \underline{150psf}$
3. $1.2D + 1.6Lr + 0.5L$	$1.2(45psf) + 1.6(0psf) + 0.5(60psf) = \underline{84psf}$
4. $1.2D + 1.0W + 0.5L + 0.75Lr$	$1.2(45psf) + 1.0(0psf) + 0.5(60psf) + 0.5(0psf) = \underline{84psf}$
5. $0.9D + 1.0W$	$0.9(45psf) + 1.0(0psf) = \underline{40.5psf}$

Load combination 2 controls with a total load of 150 psf.

P2.9

For the loads specified, compute the maximum combined load using the ASCE 7-16 load combinations for strength design.

$D = 2750 \text{ lb}$, $L = 4500 \text{ lb}$, $L_r = 1500 \text{ lb}$, $R = 1250 \text{ lb}$, $S = 1000 \text{ lb}$

L_r is larger than R and S so it will control

Since $L < 100 \text{ psf}$ then the factors in front of the L term in equations 3 and 4 should be 0.5 and not 1.0.

No seismic effects exist so load cases 6 & 7 are not required.

1. $1.4D$	$1.4(2750\text{lb}) = \underline{3850\text{lb}}$
2. $1.2D + 1.6L + 0.5L_r$	$1.2(2750\text{lb}) + 1.6(4500\text{lb}) + 0.5(1500\text{lb}) = \underline{11,250\text{lb}}$
3. $1.2D + 1.6L_r + 0.5L$	$1.2(2750\text{lb}) + 1.6(1500\text{lb}) + 0.5(4500\text{lb}) = \underline{7950\text{lb}}$
4. $1.2D + 1.0W + 0.5L + 0.75L_r$	$1.2(2750\text{lb}) + 1.0(0\text{lb}) + 0.5(4500\text{lb}) + 0.5(1500\text{lb}) = \underline{6300\text{lb}}$
5. $0.9D + 1.0W$	$0.9(2750\text{lb}) + 1.0(0\text{lb}) = \underline{2475\text{lb}}$

Load combination 2 controls with a total load of 11,250lb.

P2.10

For the loads specified, compute the maximum combined load using the ASCE 7-16 load combinations for strength design.

D = 87 psf, L = 150 psf

equations 3 and 4 should be 1.0 and not 0.5.

Since $L > 100$ psf then the factors in front of the L term in

No seismic effects exist so load cases 6 & 7 are not required.

1. 1.4D	$1.4(87psf) = \underline{121.8psf}$
2. 1.2D + 1.6L + 0.5Lr	$1.2(87psf) + 1.6(150psf) + 0.5(0psf) = \underline{344.4psf}$
3. 1.2D + 1.6Lr + 0.5L	$1.2(87psf) + 1.6(0psf) + 1.0(150psf) = \underline{254.4psf}$
4. 1.2D + 1.0W + 0.5L + 0.75Lr	$1.2(87psf) + 1.0(0psf) + 1.0(150psf) + 0.5(0psf) = \underline{254.4psf}$
5. 0.9D + 1.0W	$0.9(87psf) + 1.0(0psf) = \underline{78.3psf}$

Load combination 2 controls with a total load of 344.4 psf.

P2.11

For the loads specified, compute the maximum combined load using the ASCE 7-16 load combinations for strength design.

D = 75 psf, Lr = 35 psf, R = 12 psf

Since Lr is larger than R, it will control.

No seismic effects exist so load cases 6 & 7 are not required.

1. 1.4D	$1.4(75psf) = \underline{105psf}$
2. 1.2D + 1.6L + 0.5Lr	$1.2(75psf) + 1.6(0psf) + 0.5(35psf) = \underline{107.5psf}$
3. 1.2D + 1.6Lr + 0.5L	$1.2(75psf) + 1.6(35psf) + 0.5(0psf) = \underline{146psf}$
4. 1.2D + 1.0W + 0.5L + 0.75Lr	$1.2(75psf) + 1.0(0psf) + 0.5(0psf) + 0.5(35psf) = \underline{107.5psf}$
5. 0.9D + 1.0W	$0.9(75psf) + 1.0(0psf) = \underline{67.5psf}$

Load combination 3 controls with a total load of 146 psf.

P2.12

For the loads specified, compute the maximum combined load using the ASCE 7-16 load combinations for strength design.

$D = 13 \text{ k}$, $L = 32 \text{ k}$, $W = -22 \text{ k}$, $(E_v + E_h) = 16 \text{ k}$, $(-E_v + E_h) = -16 \text{ k}$ Because wind and seismic have different factors and reversible loads, it is not possible to say which one controls in all cases. Calculate for both. Also since L resulted from a load less than 100 psf, use a factor of 0.5 instead of 1.0 on load combinations 3, 4 and 5.

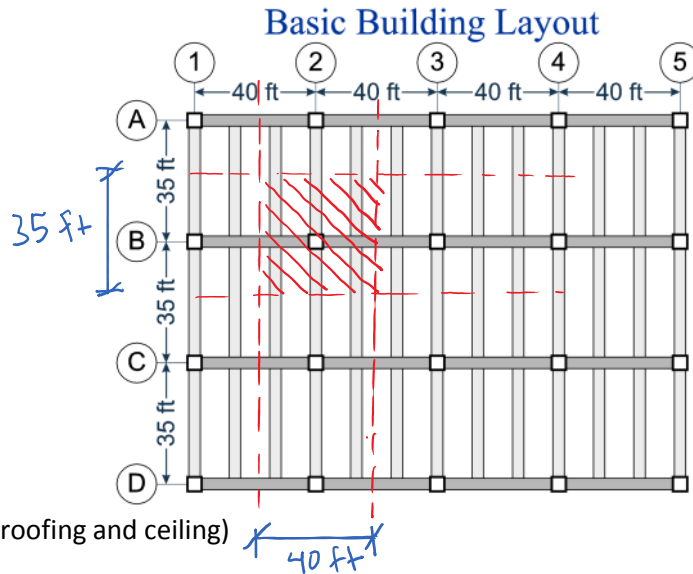
1. $1.4D$	$1.4(13k) = \underline{18.2k}$
2. $1.2D + 1.6L + 0.5L_r$	$1.2(13k) + 1.6(32k) + 0.5(0k) = \underline{66.8k}$
3. $1.2D + 1.6L_r + 0.5L$	$1.2(13k) + 1.6(0k) + 0.5(32k) = \underline{31.6k}$
4. $1.2D + 1.0W + 0.5L + 0.75L_r$	$1.2(13k) + 1.0(-22k) + 0.5(32k) + 0.5(0k) = \underline{9.6k}$
5. $0.9D + 1.0W$	$0.9(13k) + 1.0(-22k) = \underline{-10.3k}$
6. $1.2D + 1.0(E_v + E_h) + 0.5L + 0.2S$	$1.2(13k) + 1.0(+16k) + 0.5(32k) + 0.2(0k) = \underline{47.6k}$
7. $0.9D + 1.0(-E_v + E_h)$	$0.9(13k) + 1.0(-16k) = \underline{-4.3k}$

Load combination 2 controls with a total load positive load 66.8k and load combination 5 controls with a total negative load of -10.3 k.

P2.13

Use the building layout provided. Determine the roof dead load and also the roof live load (in psf) to be applied to Column B2.

The roof of the building is flat. It is composed of 3-ply ready roofing on 3 inches of reinforced concrete. The ceiling beneath the roof is a suspended ceiling.



Dead Load: (contribution from slab, 3-ply roofing and ceiling)
Values found in Table 2.1 of the text.

Source	Dead load calculation
Concrete slab	$(3in) \left(\frac{1ft}{12in} \right) (150 lb/ft^3) = \underline{37.5psf}$
3-ply ready roofing	<u>1psf</u>
Suspended ceiling	<u>2psf</u>
Total Dead Load (D)	$37.5psf + 1psf + 2psf = \underline{\underline{40.5psf}}$

Roof Live Load: $L_r = R_1 R_2 (20psf)$

We need to calculate the reduction factors. R_2 is a function of roof slope. Since the roof is flat then $R_2 = 1.0$. R_1 is a function of the tributary area of the column. The tributary area for column B2 is defined by drawing dividing lines half-way between the column lines. Thus, A_{TB2} is found as:

$$A_{TB2} = (35ft)(40ft) = 1400ft^2$$

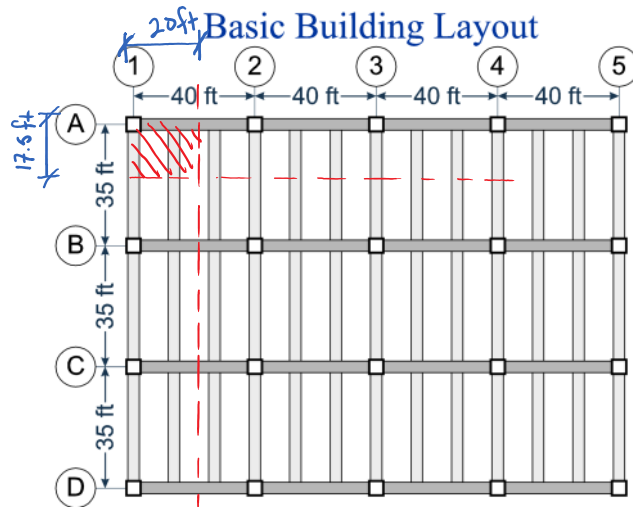
Since the tributary area is greater than $600ft^2$ then $R_1 = 0.6$.

$$L_r = (0.6)(1.0)(20psf) = \underline{\underline{12psf}}$$

P2.14

Use the building layout provided. Determine the roof dead load and also the roof live load (in psf) to be applied to Column A1.

The roof of the building is flat. It is composed of 2 inches of reinforced concrete on 18-gauge metal decking that weighs 3 psf. A single-ply waterproof sheet (0.7psf) will be used. The ceiling beneath the roof will not be finished but an allowance for mechanical ducts should be provided.



Dead Load: (contribution from slab, decking, waterproof sheet, and mechanical ducts.)
Values found in Table 2.1 of the text.

Source	Dead load calculation
Concrete slab	$(2in) \left(\frac{1ft}{12in} \right) (150 lb/ft^3) = \underline{25psf}$
18-gauge decking	<u>3psf</u>
Waterproof sheet	<u>0.7psf</u>
Mechanical ducts	<u>4psf</u>
Total Dead Load (D)	$25psf + 3psf + 0.7psf + 4psf = \underline{\underline{32.7psf}}$

Roof Live Load: $L_r = R_1 R_2 (20psf)$

We need to calculate the reduction factors. R_2 is a function of roof slope. Since the roof is flat then $R_2 = 1.0$. R_1 is a function of the tributary area of the column. The tributary area for column A1 is defined by drawing dividing lines half-way between the column lines. Thus, $A_{T_{A1}}$ is found as:

$$A_{T_{A1}} = (17.5ft)(20ft) = 350ft^2$$

Since the tributary area is greater than $200ft^2$ but less than $600ft^2$ then

$$R_1 = 1.2 - 0.001(350ft^2) = 0.85.$$

$$L_r = (0.85)(1.0)(20psf) = \underline{\underline{17psf}}$$

P2.15

Determine the dead load and live load for the second floor of a library in which any area can be used for stacks. Assume that there will be a steel channel ceiling system (2 psf) and asphalt tile on the floors. The floors are 6-in. reinforced concrete. Allowance should be provided for mechanical ducts.

Dead Load (D): (Values taken from Table 2.1 of the text.)

<u>Source</u>	<u>Dead load calculation</u>
Concrete slab	$(6in) \left(\frac{1ft}{12in} \right) (150 lb/ft^3) = \underline{75psf}$
Steel channel ceiling system	<u>2psf</u>
Asphalt tile	<u>1psf</u>
Mechanical ducts	<u>4psf</u>
Total Dead Load (D)	$75psf + 2psf + 1psf + 4psf = \underline{\underline{82psf}}$

Floor Live Load (L): (Value found in Table 2.2 of the text)

Library stacks $L = \underline{\underline{150psf}}$

P2.16

Determine the dead load and live load for a floor in a light manufacturing warehouse/office complex in which any area can be used for storage. Assume that there will be no ceiling or floor finish and that the floors are 4-in. reinforced concrete. Allowance for mechanical ducts should be provided.

Dead Load (D): (Values taken from Table 2.1 of the text.)

<u>Source</u>	<u>Dead load calculation</u>
Concrete slab	$(4in) \left(\frac{1ft}{12in} \right) (150 lb/ft^3) = \underline{50psf}$
Mechanical ducts	$\underline{4psf}$
Total Dead Load (D)	$50psf + 4psf = \underline{\underline{54psf}}$

Floor Live Load (L): (Value found in Table 2.2 of the text)

Light manufacturing $L = \underline{\underline{125psf}}$

P2.17

Determine the dead load and live load for a typical upper floor in an office building with movable steel partition walls. The ceiling below is a suspended steel channel system (2 psf) and the floors have a linoleum finish. The floors are 3-in. reinforced concrete. Allowance should be provided for mechanical ducts.

Dead Load (D): (Values taken from Table 2.1 of the text.)

<u>Source</u>	<u>Dead load calculation</u>
Concrete slab	$(3in) \left(\frac{1ft}{12in} \right) (150 lb/ft^3) = \underline{37.5psf}$
Movable steel partitions	$\underline{4psf}$
Linoleum	$\underline{1psf}$
Suspended Steel Channel	$\underline{2psf}$
Mechanical ducts	$\underline{4psf}$
Total Dead Load (D)	$37.5psf + 4psf + 1psf + 2psf + 4psf = \underline{\underline{48.5psf}}$

Floor Live Load (L): (Value found in Table 2.2 of the text)

Offices $L = \underline{\underline{50psf}}$

P2.18

Determine the loads on an upper floor in a school, a steel channel ceiling system, and a 3-in. reinforced concrete floor with asphalt tile covering. Allowance should be made for mechanical ducts.

Dead Load (D): (Values taken from Table 2.1 of the text.)

<u>Source</u>	<u>Dead load calculation</u>
Concrete slab	$(3in) \left(\frac{1ft}{12in} \right) (150 lb/ft^3) = \underline{37.5psf}$
Asphalt tile	$\underline{1psf}$
Suspended Steel Channel	$\underline{2psf}$
Mechanical ducts	$\underline{4psf}$
Total Dead Load (D)	$37.5psf + 1psf + 2psf + 4psf = \underline{\underline{44.5psf}}$

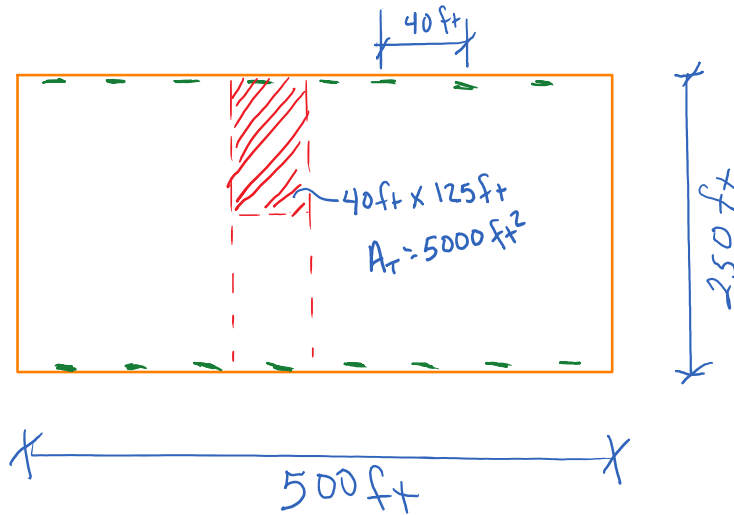
Floor Live Load (L): (Value found in Table 2.2 of the text)

School Corridor $L = \underline{\underline{80psf}}$

School classrooms $L = \underline{\underline{40psf}}$

P2.19

Determine the design rain load on the roof of a building that is 250 ft wide and 500 ft long. The architect has decided to use 6-in. diameter drains spaced uniformly on the long sides of the building at 40-ft intervals for the secondary drainage system. The secondary drains are located 1.5 in. above the roof surface (primary drain). The rainfall intensity at the location of this building is 2.5 in. per hour.



The tributary for each drain is 5000ft^2 as shown in the sketch.

The peak flow is calculated as: $Q = 0.0104(5000\text{ft}^2) \left(\frac{2.5\text{in}}{\text{hr}} \right) = 130\text{gpm}$

Refer to Table 2.5 of the text and for a 6 in. diameter drain pipe the hydraulic head is $d_h = 1\text{in}$ for 100gpm and $d_h = 2\text{in}$ for 190gpm . This means interpolation must take place.

$$\frac{190\text{gpm} - 130\text{gpm}}{190\text{gpm} - 100\text{gpm}} = \frac{2\text{in} - d_h}{2\text{in} - 1\text{in}} \quad \Rightarrow \quad d_h = \underline{1.33\text{in}}$$

The static depth is given as $d_s = \underline{1.5\text{in}}$

$$R = 5.2(1.33\text{in} + 1.5\text{in}) = \underline{\underline{14.7\text{psf}}}$$

P2.20

A new auditorium is being built in a region where the ground snow load is 75 psf. The surroundings of the building site can be classified as an exposure C with partial shelter. It is adequately insulated and kept well above freezing. What is the flat roof design snow load, in psf, which should be used for this building?

Risk category is defined in Table 2.8 of the text. An auditorium will be a category III where $I_s = 1.1$.

Exposure C with partial shelter means $C_e = 1.0$ as seen in Table 2.6 of the text.

Since this is not a greenhouse, and it is not kept just above or below freezing and it is not unheated then the only condition which applies in Table 2.7 of the text is "All other structures." This produces a $C_t = 1.0$

Flat roof snow:

$$p_f = 0.7C_eC_tI_s p_g = 0.7(1.0)(1.0)(1.1)(75psf) = \underline{\underline{57.75psf}}$$

Check the minimum requirement for the case where $p_g > 20psf$

$$p_{f_min} = (20psf)(I_s) = (20psf)(1.1) = 22psf \quad \text{Minimum doesn't control}$$

P2.21

A new fire station is being built in a region where the ground snow load is 15 psf. The surroundings of the building site can be classified as an exposure B and the roof is sheltered. It is adequately insulated and kept well above freezing. What is the flat roof design snow load, in psf, which should be used for this building?

Risk category is defined in Table 2.8 of the text. A fire station will be a category IV where $I_s = 1.2$.

Exposure B with shelter means $C_e = 1.2$ as seen in Table 2.6 of the text.

Since this is not a greenhouse, and it is not kept just above or below freezing and it is not unheated then the only condition which applies in Table 2.7 of the text is "All other structures." This produces a $C_t = 1.0$

Flat roof snow:

$$p_f = 0.7C_eC_tI_s p_g = 0.7(1.2)(1.0)(1.2)(15psf) = \underline{\underline{15.12psf}}$$

Check the minimum requirement for the case where $p_g < 20psf$

$$p_{f_min} = (15psf)(I_s) = (15psf)(1.2) = \underline{\underline{18psf}} \quad \underline{\underline{\text{Minimum snow controls}}}$$

P2.22

A pole-barn is being constructed to store hay and other farm equipment in a region where the ground snow load is 20 psf. The surroundings of the building site can be classified as an exposure D and the roof is unsheltered. It is an unheated building. What is the flat roof design snow load, in psf, which should be used for this building?

Risk category is defined in Table 2.8 of the text. An agricultural building will be a category I where $I_s = 0.8$.

Exposure D with no shelter means $C_e = 0.8$ as seen in Table 2.6 of the text.

Since this is not heated then Table 2.7 indicates $C_t = 1.2$

Flat roof snow:

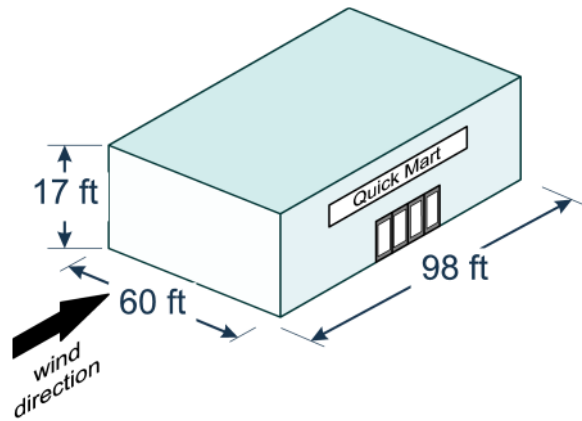
$$p_f = 0.7C_eC_tI_s p_g = 0.7(0.8)(1.2)(0.8)(20psf) = \underline{\underline{10.75psf}}$$

Check the minimum requirement for the case where $p_g \leq 20psf$

$$p_{f_min} = (p_g)(I_s) = (20psf)(0.8) = \underline{\underline{16psf}} \quad \textbf{Minimum controls}$$

P2.23

A convenience store is to be built in New Orleans, Louisiana ($lat = 29.97^\circ$, $long = -90.06^\circ$) in a location which could be classified as an exposure C. Calculate the design wind pressures for the MWFRS of the building for the wind direction shown in the accompanying figure. Provide a sketch of the wind pressure zones similar to what is presented in Example 2.8 of the text.



For the location given, find the wind speeds using <http://atcouncil.org/windspeed/>.

Referring to Table 2.8 of the text we see that this structure is not a large assembly area, not agricultural and not emergency type so we will classify this as a risk category II. The wind speed for risk category II of this location is:

$$V = 144 \text{ mph}$$

Assume $K_{zt} = 1.0$ for typical terrain.

Calculate the adjustment factor, λ , for the height and exposure of the building. The roof height is 17 ft and the exposure category is C. The adjustment factor table gives the following. Interpolate:

$$\frac{20 \text{ ft} - 17 \text{ ft}}{20 \text{ ft} - 15 \text{ ft}} = \frac{1.29 - \lambda}{1.29 - 1.21} \Rightarrow \lambda = 1.24$$

For the wind direction given, we need to find p_{s30} for zones A, C, E, F, G, H which will be for a roof angle of $\theta = 0^\circ$. The following pressures are found from Figure 28.6-1 of ASCE 7 in units of psf.

<u>Wind Speed</u>	<u>A</u>	<u>C</u>	<u>E</u>	<u>F</u>	<u>G</u>	<u>H</u>
140 mph	31.1	20.6	-37.3	-21.2	-26.0	-16.4
160 mph	40.6	26.9	-48.8	-27.7	-34.0	-21.5
Interpolating for 144 mph						
144 mph	33.0	21.9	-39.6	-22.5	-27.6	-17.4

Search Results

Latitude: 29.9700
Longitude: -90.0600

ASCE 7-10 Wind Speeds
(3-sec peak gust MPH*):

Risk Category I: 133
Risk Category II: 144
Risk Category III-IV: 153

Adjustment Factor
for Building Height and Exposure, λ

Mean roof height (ft)	Exposure		
	B	C	D
15	1.00	1.21	1.47
20	1.00	1.29	1.55

P2.23 (cont'd)

We need to adjust the wind pressures for the exposure and height using the coefficient $\lambda = 1.24$.

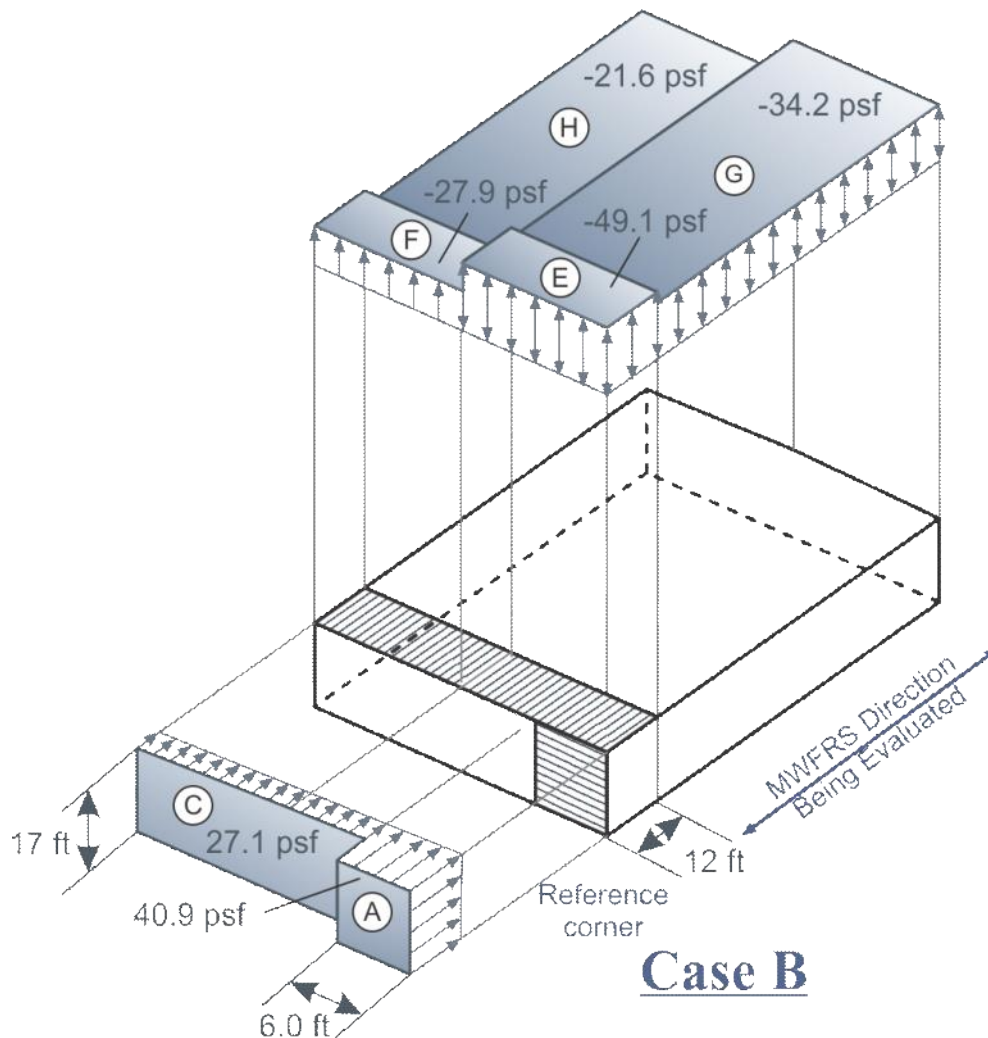
<u>Wind Speed</u>	<u>A</u>	<u>C</u>	<u>E</u>	<u>F</u>	<u>G</u>	<u>H</u>
144 mph	40.9	27.1	-49.1	-27.9	-34.2	-21.6

The dimension a is the least of the following values.

$$a = 0.10(60ft) = 6ft \leftarrow \text{(controls)}$$

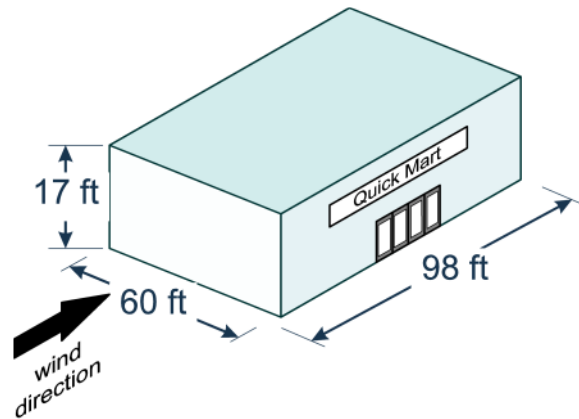
Or

$$a = 0.4(17ft) = 6.8ft$$



P2.24

A convenience store is to be built in Lincoln, Nebraska ($lat = 40.80^\circ$, $long = -96.67^\circ$) in a location which could be classified as an exposure B. Calculate the design wind pressures for the MWFRS of the building for the wind direction shown in the accompanying figure. Provide a sketch of the wind pressure zones similar to what is presented in Example 2.8 of the text.



For the location given, find the wind speeds using <http://atcouncil.org/windspeed/>.

Referring to Table 2.8 of the text we see that this structure is not a large assembly area, not agricultural and not emergency type so we will classify this as a risk category II. The wind speed for risk category II of this location is:

$$V = 115 \text{ mph}$$

Assume $K_{zt} = 1.0$ for typical terrain.

Calculate the adjustment factor, λ , for the height and exposure of the building. The roof height is 17 ft and the exposure category is B. The adjustment factor table gives the following. $\Rightarrow \lambda = 1.00$

Search Results

Query Date: Tue Oct 27 2015

Latitude: 40.8000

Longitude: -96.6700

**ASCE 7-10 Windspeeds
(3-sec peak gust in mph*):**

Risk Category I: 105

Risk Category II: 115

Risk Category III-IV: 120

Adjustment Factor
for Building Height and Exposure, λ

Mean roof height (ft)	Exposure		
	B	C	D
15	1.00	1.21	1.47
20	1.00	1.29	1.55

For the wind direction given, we need to find p_{s30} for zones A, C, E, F, G, H which will be for a roof angle of $\theta = 0^\circ$. The following pressures are found from Figure 28.6-1 of ASCE 7 in units of psf.

Wind Speed	A	C	E	F	G	H
115 mph	21.0	13.9	-25.2	-14.3	-17.5	-11.1

P2.24 (cont'd)

We need to adjust the wind pressures for the exposure and height using the coefficient $\lambda = 1.00$.

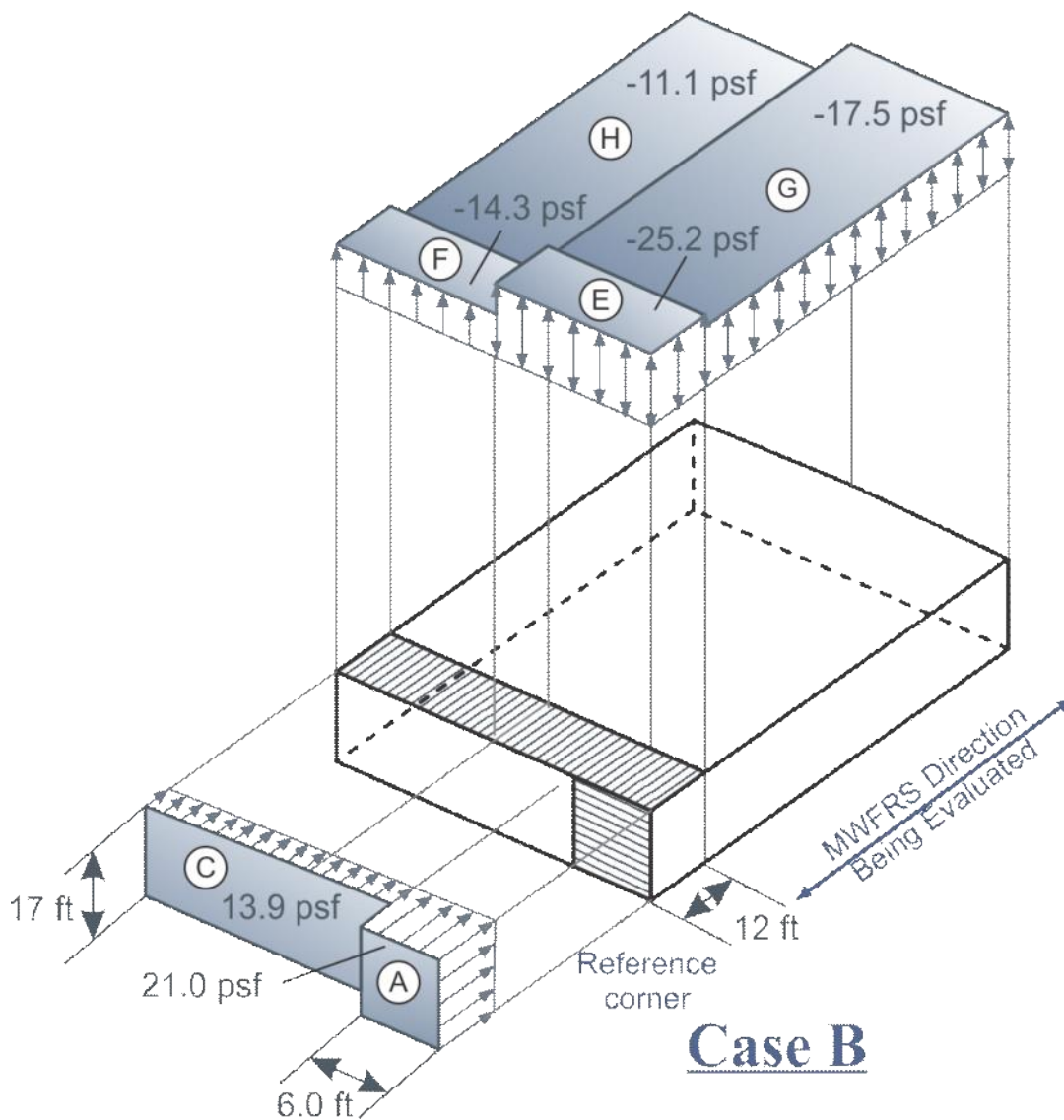
<u>Wind Speed</u>	<u>A</u>	<u>C</u>	<u>E</u>	<u>F</u>	<u>G</u>	<u>H</u>
115 mph	21.0	13.9	-25.2	-14.3	-17.5	-11.1

The dimension a is the least of the following values.

$$a = 0.10(60ft) = 6ft \leftarrow (\text{controls})$$

Or

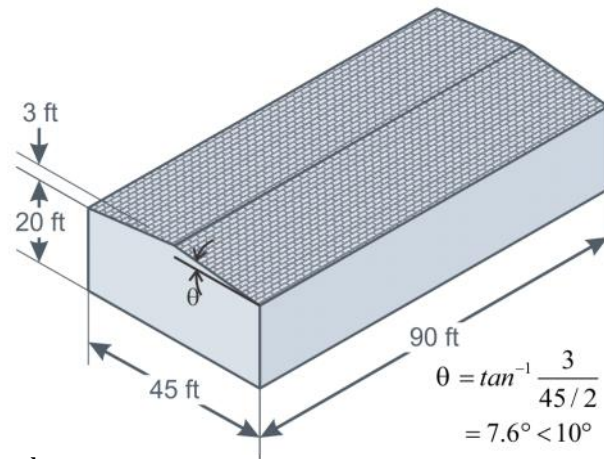
$$a = 0.4(17ft) = 6.8ft$$



P2.25

For the structure shown in Example 2.8, find the design wind pressures of the MWFRS for both wind directions. Assume that the design wind speed is 120 mph in an exposure B category. Provide a sketch of the wind pressures on the building similar to Example 2.8.

Assume $K_{zt} = 1.0$ for typical terrain.



Calculate the adjustment factor, λ , for the height and exposure of the building. The roof height is 20ft and the exposure category is B. The adjustment factor table gives the following $\Rightarrow \lambda = 1.00$

Adjustment Factor for Building Height and Exposure, λ			
Mean roof height (ft)	Exposure		
	B	C	D
15	1.00	1.21	1.47
20	1.00	1.29	1.55

For the wind direction given, we need to find p_{s30} for zones A, B, C, D, E, F, G, H which will be for a roof angle of $\theta = 7.6^\circ$. The following pressures are found from Figure 28.6-1 of ASCE 7 in units of psf. This is for a 120 mph wind speed.

Roof Angle	A	B	C	D	E	F	G	H
5°	22.8	-11.9	15.1	-7.0	-27.4	-15.6	-19.1	-12.1
10°	25.8	-10.7	17.1	-6.2	-27.4	-16.8	-19.1	-12.9
Interpolating for 7.6°								
7.6°	24.4	-11.3	16.1	-6.6	-27.4	-16.2	-19.1	-12.5

Since $\lambda = 1.00$ then no adjustment needs to be made to these pressures.

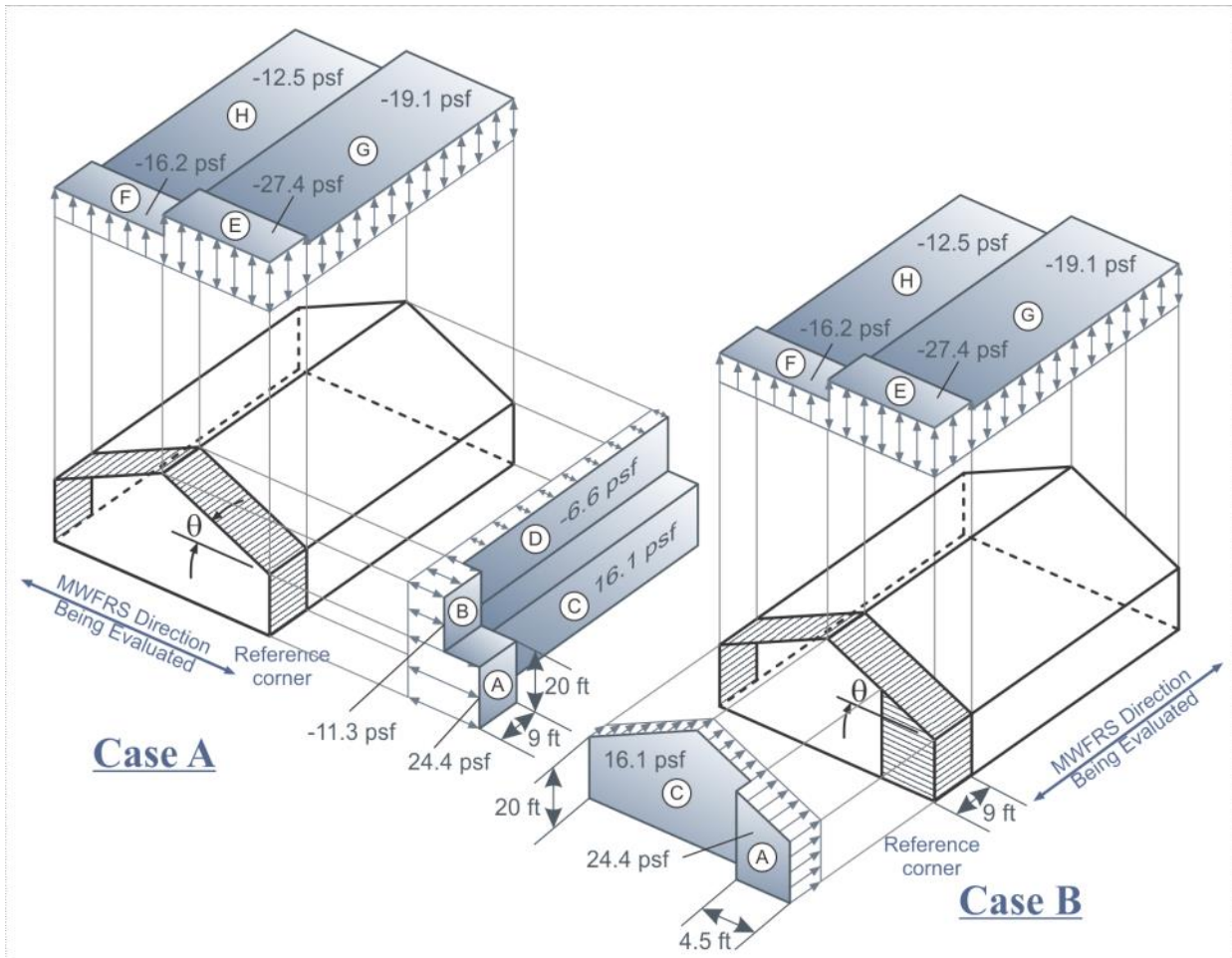
The dimension a is the least of the following values.

$$a = 0.10(45ft) = 4.5ft \leftarrow (controls)$$

Or

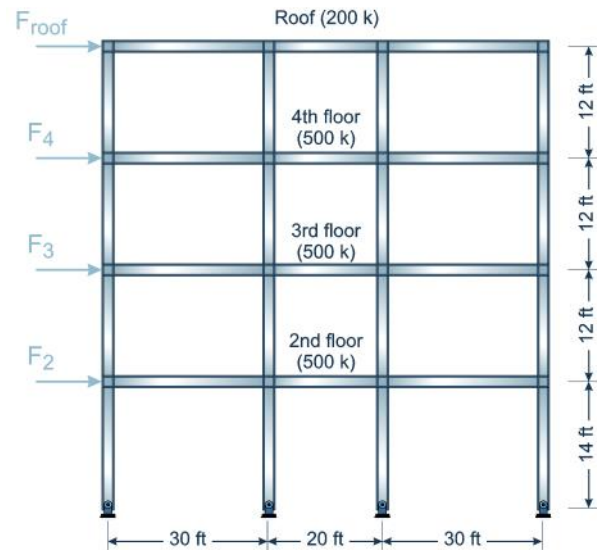
$$a = 0.4(20ft) = 8ft$$

P2.25 (cont'd)



P2.26

For the structure shown, calculate the equivalent seismic loads which should be applied at all levels (F_2 , F_3 , F_4 , F_{roof}). This structure will be used as a hospital located in Buffalo, New York ($lat = 42.89^\circ$, $long = -78.88^\circ$). It will be located a soil type C and will be constructed of a reinforced concrete moment frame ($R = 5$).



Solution:

1. $T_a = C_t h_n^x$ For a concrete moment frame $C_t = 0.016$, and $x = 0.9$. The overall height of the building is: $h_n = 50ft$. Therefore:

$$T_a = (0.016)(50ft)^{0.9} = 0.54seconds$$

2. Accelerations are determined from the USGS website. (Note that this building is an essential facility (see Table 2.8 of the text) and is thus a risk category IV.

Report Title Problem 2-26

Fri January 17, 2014 13:59:30 UTC

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 42.89°N, 78.88°W

Site Soil Classification Site Class C - "Very Dense Soil and Soft Rock"

Risk Category IV (e.g. essential facilities)

USGS-Provided Output

$S_s = 0.208 \text{ g}$	$S_{HS} = 0.250 \text{ g}$	$S_{DS} = 0.167 \text{ g}$
$S_1 = 0.060 \text{ g}$	$S_{M1} = 0.102 \text{ g}$	$S_{D1} = 0.068 \text{ g}$

3. The response modification factor is given in the problem statement. $R = 5$
4. The importance factor for a risk category IV building is found in Table 2.11 of the text.
 $I_e = 1.5$
5. The effective seismic weight is already indicated in the figure. Sum up all of the weights.

$$W = (3)(500k) + 200k = 1700k$$

P2.26 (cont'd)

6. Seismic base shear is calculated as: $V = \frac{S_{D1}}{T_a \left(\frac{R}{I_e}\right)} W = \frac{0.068}{(0.54) \left(\frac{5}{1.5}\right)} (1700k) = \underline{64.2k}$

V need not exceed: $V_{max} = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} W = \frac{0.167}{\left(\frac{5}{1.5}\right)} (1700k) = \underline{85.2k}$

It does need to meet the following minimum: $V_{min} = 0.044 S_{DS} I_e W = (0.044)(0.167)(1.5)(1700k) = \underline{18.7k}$

Use $V = \underline{64.2k}$

7. Distribute the forces to each story according to the following equation: $F_x = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k} V$

It requires a knowledge of k. $k = 1 + \frac{T-0.5}{2} = 1 + \frac{0.54-0.5}{2} = 1.02$

$$\sum_{i=1}^4 w_i h_i^k = (500k)(14ft)^{1.02} + (500k)(26ft)^{1.02} + (500k)(38ft)^{1.02} + (200k)(50ft)^{1.02} = 52502$$

$$F_2 = \frac{(500k)(14ft)^{1.02}}{52502} (64.2k) = (0.141)(64.2k) = \underline{9.05k}$$

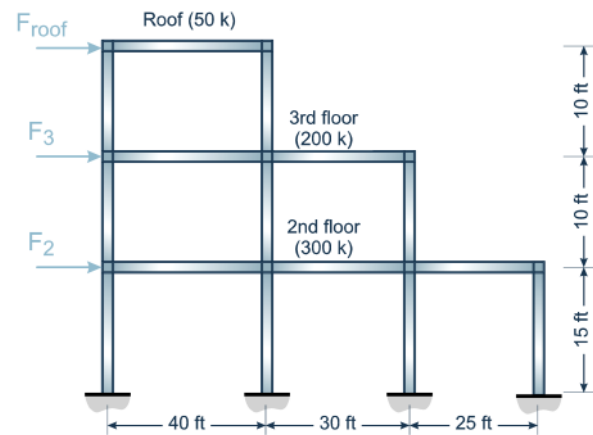
$$F_3 = \frac{(500k)(26ft)^{1.02}}{52502} (64.2k) = (0.264)(64.2k) = \underline{16.95k}$$

$$F_4 = \frac{(500k)(38ft)^{1.02}}{52502} (64.2k) = (0.389)(64.2k) = \underline{24.97k}$$

$$F_{roof} = \frac{(200k)(50ft)^{1.02}}{52502} (64.2k) = (0.206)(64.2k) = \underline{13.23k}$$

P2.27

For the ordinary steel moment frame ($R = 3.5$) structure shown, calculate the equivalent seismic loads which should be applied at all levels (F_2 , F_3 , F_{roof}). The structure will be used as an office building and will be located in Atlanta, Georgia ($lat = 33.75^\circ$, $long = -84.39^\circ$) where the soil type is classified as a type D.



Solution:

1. $T_a = C_t h_n^x$ For a steel moment frame $C_t = 0.028$, and $x = 0.8$. The overall height of the building is: $h_n = 35ft$. Therefore:

$$T_a = (0.028)(35ft)^{0.8} = 0.48 \text{ seconds}$$

2. Accelerations are determined from the USGS website. (Note that this building is an office building (see Table 2.8 of the text) and is thus a risk category II.

Report Title Problem 2-27

Fri January 17, 2014 14:39:23 UTC

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 33.75°N, 84.39°W

Site Soil Classification Site Class D - "Stiff Soil"

Risk Category I/II/III

USGS-Provided Output

$S_s = 0.185 \text{ g}$	$S_{HS} = 0.296 \text{ g}$	$S_{DS} = 0.197 \text{ g}$
$S_1 = 0.090 \text{ g}$	$S_{H1} = 0.215 \text{ g}$	$S_{D1} = 0.144 \text{ g}$

3. The response modification factor is given in the problem statement. $R = 3.5$
1. The importance factor for a risk category II building is found in Table 2.11 of the text.
 $I_e = 1.0$
4. The effective seismic weight is already indicated in the figure. Sum up all of the weights.

$$W = 300k + 200k + 50k = 550k$$

P2.27 (cont'd)

6. Seismic base shear is calculated as: $V = \frac{S_{D1}}{T_a \left(\frac{R}{I_e}\right)} W = \frac{0.144}{(0.48) \left(\frac{3.5}{1.0}\right)} (550k) = \underline{47.1k}$

V need not exceed: $V_{max} = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} W = \frac{0.197}{\left(\frac{3.5}{1.0}\right)} (550k) = \underline{31.0k}$

It does need to meet the following minimum: $V_{min} = 0.044 S_{DS} I_e W = (0.044)(0.197)(1.0)(550k) = \underline{4.8k}$

Use $V = \underline{31.0k}$

7. Distribute the forces to each story according to the following equation: $F_x = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k} V$

It requires a knowledge of k. Since $T_a = 0.48 < 0.5$ then $k = 1.0$

$$\sum_{i=1}^3 w_i h_i^k = (300k)(15ft)^{1.0} + (200k)(25ft)^{1.0} + (50k)(35ft)^{1.0} = 11250$$

$$F_2 = \frac{(300k)(15ft)^{1.0}}{11250} (31.0k) = (0.400)(31.0k) = \underline{\underline{12.40k}}$$

$$F_3 = \frac{(200k)(25ft)^{1.0}}{11250} (31.0k) = (0.444)(31.0k) = \underline{\underline{13.76k}}$$

$$F_{roof} = \frac{(50k)(35ft)^{1.0}}{11250} (31.0k) = (0.156)(31.0k) = \underline{\underline{4.84k}}$$