S.E. Exam Review: Lateral Loads

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Equivalent Lateral Force Procedure

ASCE 7-10 Notation

W → effective seismic weight, as defined in Section 12.7.2

W includes the dead load, as defined in Section 3.1, above the base and other loads above the base, including:

1. In areas used for storage, 25 percent of the floor live load (exceptions).
2. Where provision for partitions is required by Section 4.3.2 in the floor load design, the actual partition weight or a minimum weight of 10 psf of floor area, whichever is greater.
3. Total operating weight of permanent equipment.
4. Where the flat roof snow load, p_f, exceeds 30 psf, 20 percent of the uniform design snow load, regardless of actual roof slope.
5. Weight of landscaping and other materials at roof gardens and similar areas.

ASCE 7-10 Notation

C_s → seismic response coefficient

T → fundamental period of building

T_a → approximate fundamental period of building

T_L → long-period transition period as defined in Section 11.4.5

I_e → importance factor

R → response modification coefficient

C_d → deflection amplification factor

MCE_R → risk targeted maximum considered earthquake

S_S → mapped MCE_R, 5 percent damped, spectral response acceleration parameter at short periods

S_1 → mapped MCE_R, 5 percent damped, spectral response acceleration parameter at a period of 1 s

ASCE 7-10 Notation

S_{MS} → 5 percent damped, MCE_R spectral response acceleration at short periods, adjusted for site class

S_{M1} → 5 percent damped, MCE_R spectral response acceleration at 1 *s*, adjusted for site class

S_{DS} → 5 percent damped, design spectral response acceleration parameter at short periods

S_{D1} → 5 percent damped, design spectral response acceleration parameter at a period of 1 *s*

Equivalent Lateral Force Procedure

$V = C_s W$ (ASCE 7-10 Eq. 12.8-1)

$C_s = \frac{S_{DS}}{R/I_e}$ (ASCE 7-10 Eq. 12.8-2)

If $T \leq T_L$, C_s need not exceed $\frac{S_{D1}}{T(R/I_e)}$

If $T > T_L$, C_s need not exceed $\frac{S_{D1}T_L}{T^2(R/I_e)}$

C_s shall not be less than $0.044 S_{DS} I_e \geq 0.01$ (near fault minimum – applies to all SDCs)

If $S_1 \geq 0.6\text{g}$ C_s shall not be less than $\frac{0.5S_1}{(R/I_e)}$

ASCE 7-10 Table 12.2-1

Table 12.2-1 Design Coefficients and Factors for Seismic Force-Resisting Systems									
Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R[a]	Overstrength Factor, Ω_o[g]	Deflection Amplification Factor, C_d[b]	Structural System Limitations Including Structural Height, h_n (ft) Limits[c]				
					Seismic Design Category				
					B	C	D[d]	E[d]	F[e]
A. Bearing Wall Systems									
1. Special reinforced concrete shear walls[l,m]	14.2	5	2 ½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls[l]	14.2	4	2 ½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls[l]	14.2	2	2 ½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls[l]	14.2	1 ½	2 ½	1 ½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls[l]	14.2	4	2 ½	4	NL	NL	40[k]	40[k]	40[k]
6. Ordinary precast shear walls[l]	14.2	3	2 ½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4	5	2 ½	3 ½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4	3 ½	2 ½	2 ¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2 ½	1 ¾	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	2 ½	1 ¾	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	1 ½	2 ½	1 ¼	NL	NP	NP	NP	NP

Mixed Systems

Mixed systems different directions – may use *R* for respective direction

Mixed systems same direction – must use smallest *R*

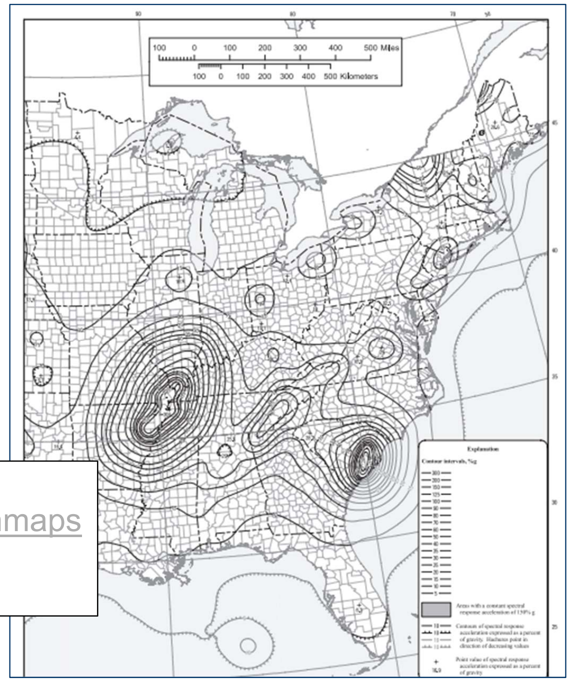
Vertically Stacked Mixed Systems

Where the lower system has a lower response modification coefficient, *R*, the design coefficients (R, Ω_o, and C_d) for the upper system are permitted to be used to calculate the forces and drifts of the upper system. For the design of the lower system, the design coefficients (R, Ω_o, and C_d) for the lower system shall be used. Forces transferred from the upper system to the lower system shall be increased by multiplying by the ratio of the higher response modification coefficient to the lower response modification coefficient.

Vertically Stacked Mixed Systems

Where the upper system has a lower response modification coefficient, the design coefficients (R, Ω_o, and C_d) for the upper system shall be used for both systems.

ASCE 7-10 Figure 22-1



http://earthquake.usgs.gov/designmaps/us/application.php

Equivalent Lateral Force Procedure

$$S_{M1} = F_v S_1$$

$$S_{MS} = F_a S_S$$

$$S_{D1} = \frac{2}{3} S_{M1}$$

$$S_{DS} = \frac{2}{3} S_{MS}$$

F_a and F_v taken from ASCE 7-10 Tables 11.4-1 and 11.4-2, or IBC 2012 Table 1613.3.3(1) and 1613.3.3(2)

ASCE 7-10 Tables 11.4-1,2

Table 11.4-1 Site Coefficient, F_a

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				

Note: Use straight-line interpolation for intermediate values of S_S.

Table 11.4-2 Site Coefficient, F_v

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7				

Note: Use straight-line interpolation for intermediate values of S_1.

ASCE 7-10 Site Classes

Table 20.3-1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard rock	> 5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	> 50	> 2,000 psf
D. Stiff soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	< 600 ft/s	< 15	< 1,000 psf
	Any profile with more then 10 ft of soil having the following characteristics: - Plasticity index PI > 20, - Moisture content w ≥ 40%, - Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		
For SI: 1 ft/s = 0.3048 m/s; 1 lb/ft^2 = 0.0479 kN/m^2.			

Building Period

Fundamental Period

ASCE 7-10 Section 12.8.2 – "The fundamental period of the structure, *T*, in the direction under consideration shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The fundamental period, *T,* shall not exceed the product of the coefficient for upper limit on calculated period (C_u) from Table 12.8-1 and the approximate fundamental period, T_a, determined in accordance with Section 12.8.2.1."

Approximate Fundamental Period

Moment resisting frames (ASCE 7-10 Eq. 12.8-7):

$$T_a = C_t h_n^x$$

h_n – height, in ft, above the base to the highest level of the structure

Alternately, for moment resisting systems made entirely of steel or concrete moment resisting frames, not exceeding 12 stories above the base and the average story height is at least 10 feet (ASCE 7-10 Eq. 12.8-8):

$$T_a = 0.1N$$

$N \rightarrow$ number of stories

ASCE 7-10 Table 12.8-2

Structure Type		C_t	x
Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:			
	Steel moment-resisting frames	0.028 (0.0724)	0.80
	Concrete moment-resisting frames	0.016 (0.0466)	0.90
Steel eccentrically braced frames in accordance with Table 12.2-1 lines B1 or D1		0.030 (0.0731)	0.75
Steel buckling-restrained braced frame		0.030 (0.0731)	0.75
All other structural systems		0.020 (0.0488)	0.75

Approximate Fundamental Period

Concrete or masonry shear wall structures (ASCE 7-10 Equations 12.8-9 and 12.8-10):

$$T_a = \frac{0.0019}{\sqrt{C_W}} h_n$$

$$C_W = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1+0.83\left(\frac{h_i}{D_i}\right)^2\right]}$$

Approximate Fundamental Period

A_B → area of the base of the structure, ft^2

A_i → web area of shear wall "*i*", ft^2

D_i → length of shear wall "*i*", ft

h_i → height of shear wall "*i*", ft

h_n → height above the base of the highest level of the structure, ft

x → number of shear walls in the building effective in resisting lateral forces in the direction under consideration

ASCE 7-10 Table 12.8-1

Table 12.8-1 Coefficient for Upper Limit on Calculated Period	
Design Spectral Response Acceleration Parameter at 1 *s*, S_{D1}	Coefficient C_u
≥ 0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
≤ 0.1	1.7

Limit on period T
$T \le C_u T_a$

Building Period Examples:

Building 1

Approximate Building Period Example – Building 1

Compute the approximate period of Building 1

ASCE 7-10 Eq. 12.8-7

$T_a = C_t h_n^x$

$T_a = (0.028)[(12\text{ ft})(10)]^{0.8} = 1.290\text{ s}$

Approximate Building Period Example – Building 1

Compute the approximate period of Building 1 if the material is reinforced concrete rather than steel

ASCE 7-10 Eq. 12.8-7

$T_a = C_t h_n^x$

$T_a = (0.016)[(12\text{ ft})(10)]^{0.9} = 1.189\text{ s}$

Approximate Building Period Example – Building 1

For the steel frame example done previously, compute the value of *T* that would be permitted by ASCE 7-10 if the period computed by an analysis is (a) 1.515 s and (b) 1.929 s. S_{D1} = 0.3240.

Part (a)

From Table 12.8-1, $C_u = 1.4$

$T = 1.515 \text{ s} < 1.4(1.290 \text{ s}) = 1.806 \text{ s}$

Use $T = 1.515 \text{ s}$

Approximate Building Period Example – Building 1

For the steel frame example done previously, compute the value of *T* that would be permitted by ASCE 7-10 if the period computed by an analysis is (a) 1.515 s and (b) 1.929 s. S_{D1} = 0.3240.

Part (b)

$T = 1.929 \text{ s} > 1.4(1.290 \text{ s}) = 1.806 \text{ s}$

Use $T = 1.806 \text{ s}$

Approximate Building Period Example – Building 2

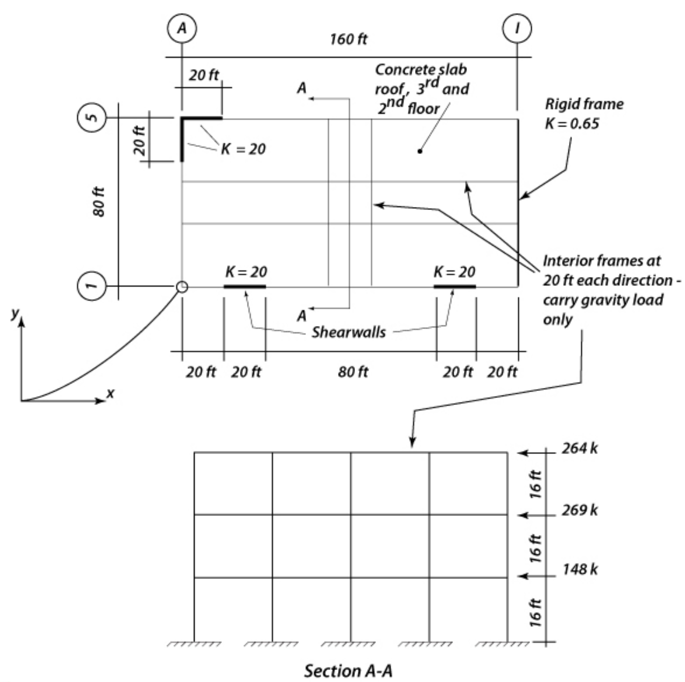
Calculate the approximate natural period in the horizontal direction for Building 2. The shear walls are 8 in. thick, made of concrete.

ASCE 7-10 Eq. 12.8-9, 12.8.10

$$T_a = \frac{0.0019}{\sqrt{C_W}} h_n$$

$$C_W = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1+0.83\left(\frac{h_i}{D_i}\right)^2\right]}$$

Approximate Building Period Example – Building 2



Approximate Building Period Example – Building 2

All walls in the horizontal direction have the same web area, length and height.

$$A_i = \frac{(8\text{ in})(20\text{ ft})}{12\text{ in/ft}} = 13.3\text{ ft}^2$$

$$D_i = 20\text{ ft}$$

$$h_n = h_i = 48\text{ ft}; \frac{h_n}{h_i} = 1$$

Approximate Building Period Example – Building 2

$$C_W = \frac{100}{(160\text{ ft})(80\text{ ft})}(3)\left[(1)^2 \frac{13.3\text{ ft}^2}{1+0.83\left(\frac{48\text{ ft}}{20\text{ ft}}\right)^2}\right] = 0.0539$$

$$T_a = \frac{0.0019}{\sqrt{0.0539}}(48\text{ ft}) = 0.393\text{ s}$$

Equivalent Lateral Force Method Example – Building 1

Equivalent Lateral Force Example – Building 1

Use the equivalent lateral force method to determine the base shear for Building 1. Allocate the base shear to each of the story levels and then allocate the story loads to individual frames. Use R = 3.5 and I_e = 1. Assume that the earthquake acts in the north-south direction.

Equivalent Lateral Force Example – Building 1

Building perimeter = 2 (75 ft + 125 ft) = 400 ft

Building area = (75 ft)(125 ft) = 9,375 ft^2

Cladding weight (400 ft)(9.5 stories)(12 ft)(77 psf)/1,000 lb/k = 3,511 k

Parapet weight = (5 ft)(400 ft)(40 psf)/1,000 lb/k = 80 k

Approximate frame weight = 915 k

Roof dead load = (9,375 ft^2)(10.2 psf) = 95.6 k

Floor dead load = (9,375 ft^2)(9 stories)(60 psf) = 5,062 k

W = 3,511 k + 80 k + 915 k + 95.6 k + 5,062 k = 9,664 k

Equivalent Lateral Force Example – Building 1

ASCE 7-10 Figures 22-1 and 22-2 or USGS tool:

$S_S = 1.123$

$S_1 = 0.357$

Adjustment for site conditions (ASCE Tables 11.4-1 and 11.4-2)

$F_a = 1.0$

$F_v = 1.443$

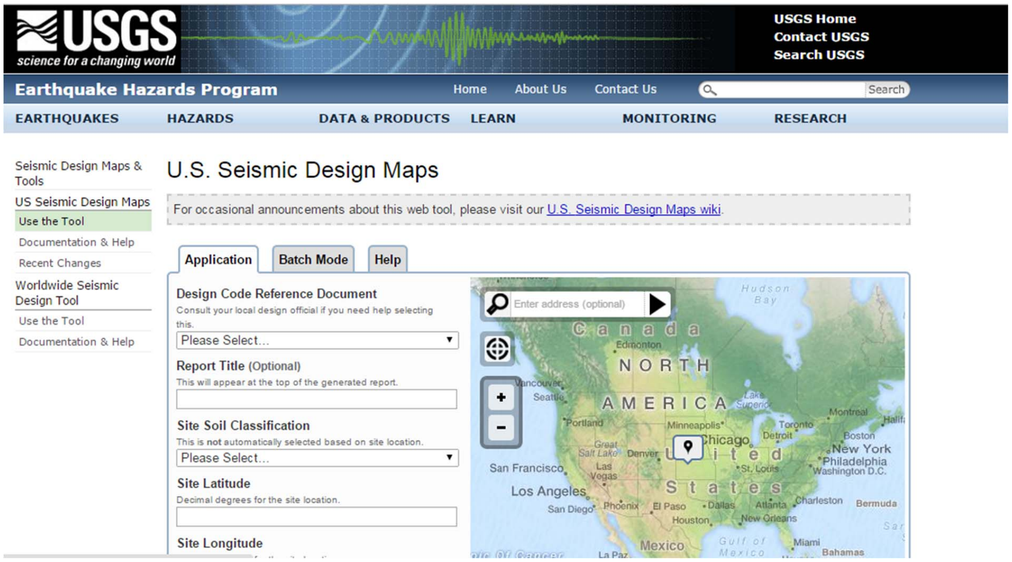
$S_{MS} = F_a S_S = 1.123$

$S_{M1} = F_v S_1 = 0.515$

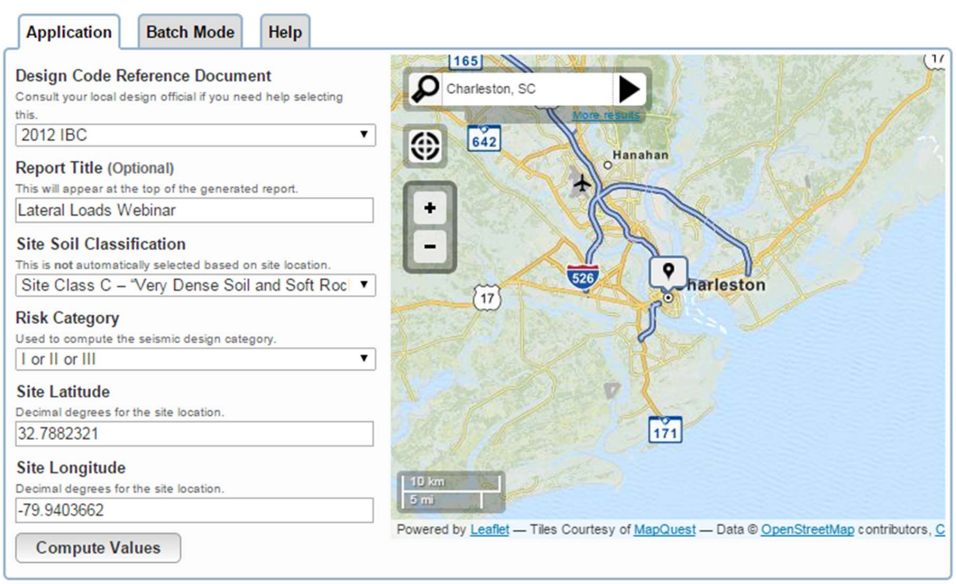
$S_{DS} = \frac{2}{3} S_{MS} = 0.749$

$S_{D1} = \frac{2}{3} S_{M1} = 0.343$

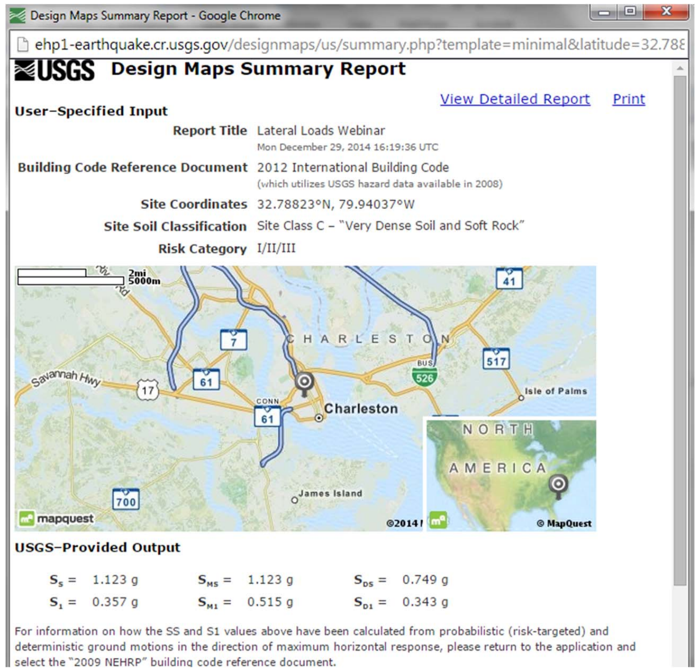
USGS Tool



USGS Tool



USGS Tool



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ASCE 7-10 Risk Category and Importance (I)

Table 1.5-1 Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent low risk to human life in the event of failure.	I
All buildings and other structures except those listed in Risk Categories I, III, and IV.	II
Buildings and other structures, the failure of which could pose a substantial risk to human life. Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure. Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.[a]	III
Buildings and other structures designated as essential facilities. Buildings and other structures, the failure of which could pose a substantial hazard to the community. Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.[a] Buildings and other structures required to maintain the functionality of other Risk Category IV structures.	IV

[a]Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.3 that a release of the substances is commensurate with the risk associated with that Risk Category.

Table 1.5-2 Importance Factors by Risk Category of Buildings and Other Structures for Snow, Ice, and Earthquake Loads[a]

Risk Category from Table 1.5-1	Snow Importance Factor I_s	Ice Importance Factor – Thickness, I_t	Ice importance Factor – Wind, I_w	Seismic Importance Factor, I_s
I	0.8	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.25	1.00	1.25
IV	1.20	1.25	1.00	1.50

[a]The component importance factor, I_p, applicable to earthquake loads, is not included in this table because it depends on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

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Equivalent Lateral Force Example – Building 1

$T = T_a = 1.290 \text{ s}, T_L = 8 \text{ s}$

$C_s = \frac{S_{DS}}{R/I_e} = \frac{0.749}{3.5/1} = 0.214$

$0.044(1)(0.749) = 0.0330 > 0.01$

$0.0330 \leq C_s \leq \frac{S_{D1}}{T(R/I_e)}$

$0.0330 \leq C_s \leq \frac{0.343}{(1.290\ s)\left(\frac{3.5}{1}\right)} = 0.0758$

$C_s = 0.0758$

$V = 0.0758(9{,}664 \text{ k}) = 731.6 \text{ k}$

Vertical Force Distribution – Building 1

Vertical Force Distribution

ASCE Section 12.8.3

$F_x = C_{vx}V$ (Eq. 12.8-11)

$C_{vx} = \frac{w_x h_x^k}{\sum_i w_i h_i^k}$ (Eq. 12.8-12)

k = 1 for structures with a period less than or equal to 0.5 s

k = 2 for structures with a period at least equal to 2.5 s

Use linear interpolation between 1 and 2 for other period values

Equivalent Lateral Force Example

ASCE 7-10 Section 12.8.3

$F_x = C_{vx}V$ (Eq. 12.8-11)

$C_{vx} = \frac{w_x h_x^k}{\sum_i w_i h_i^k}$ (Eq. 12.8-12)

$T = 1.290\ \text{s}$

$k = 1 + \frac{(1.290\ \text{s} - 0.5\ \text{s})}{(2.5\ \text{s} - 0.5\ \text{s})}(2 - 1) = 1.395$

Weight of one story of cladding $= \frac{(400\ \text{ft})(12\ \text{ft})(77\ \text{psf})}{\left(1{,}000\frac{\text{lb}}{\text{k}}\right)} = 369.6\ \text{k}$

Dead load assigned to roof = $95.6\ \text{k} + 80\ \text{k} + \frac{369.6\ \text{k}}{2} + \frac{915\ \text{k}}{10} = 451.9\ \text{k}$

Dead load assigned to floors other than roof $= \frac{(9{,}375\ \text{ft}^2)(60\ \text{psf})}{1{,}000\ \text{lb/k}} + 369.6\ \text{k} + \frac{915\ \text{k}}{10} = 1{,}024\ \text{k}$

Equivalent Lateral Force Example

$$\sum_i w_i h_i^k = (1{,}024\text{ k})\begin{bmatrix}(12\text{ ft})^{1.395} + (24\text{ ft})^{1.395} + (36\text{ ft})^{1.395} + \\ (48\text{ ft})^{1.395} + (60\text{ ft})^{1.395} + (72\text{ ft})^{1.395} + \\ (84\text{ ft})^{1.395} + (96\text{ ft})^{1.395} + (108\text{ ft})^{1.395}\end{bmatrix}$$

$$+(451.9\text{ k})(120\text{ ft})^{1.395} = 3{,}357{,}305\text{ k}\cdot\text{ft}$$

$$C_{v2} = \frac{(1{,}024\text{ k})(12\text{ ft})^{1.395}}{3{,}357{,}305\text{ k}\cdot\text{ft}} = 0.00976$$

$$C_{v8} = \frac{(1{,}024\ k)(84\text{ ft})^{1.395}}{3{,}357{,}305\text{ k}\cdot\text{ft}} = 0.147$$

$$C_{vr} = \frac{(451.9\text{ k})(120\text{ ft})^{1.395}}{3{,}357{,}305\text{ k}\cdot\text{ft}} = 0.107$$

$$F_2 = 0.00976(731.6\text{ k}) = 7.14\text{ k}$$

$$F_8 = 0.147(731.6\text{ k}) = 107.5\text{ k}$$

$$F_r = 0.107(731.6\text{ k}) = 78.3\text{ k}$$

Story Loads

Framing Supporting Floor	C_{vx}	Story Load (k)
2	0.00976	7.15
3	0.0257	18.8
4	0.0452	33.1
5	0.0675	49.4
6	0.0921	67.5
7	0.119	87.0
8	0.147	107.9
9	0.177	129.9
10	0.209	153.1
Roof	0.107	78.3
	Total	732.2

Horizontal Force Distribution – Building 1

Frame Stiffness Assumptions

Column lines A and F – K = 1.0

Column lines B to E – K = 1.25

Column lines 1 and 4 – K = 1.35

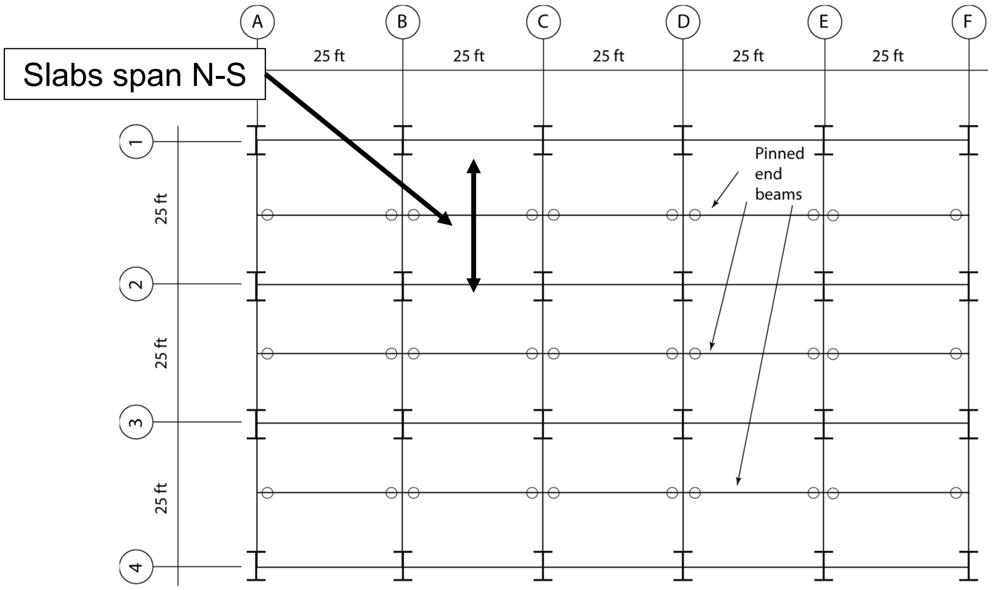
Column lines 2 and 3 – K = 1.5

$J = \sum K_x d_y^2 + \sum K_y d_x^2$

$J = 2[1.5(12.5)^2 + 1.35(37.5)^2 + 1.0(62.5)^2 + 1.25(37.5)^2 + 1.25(12.5)^2] = 15{,}984$

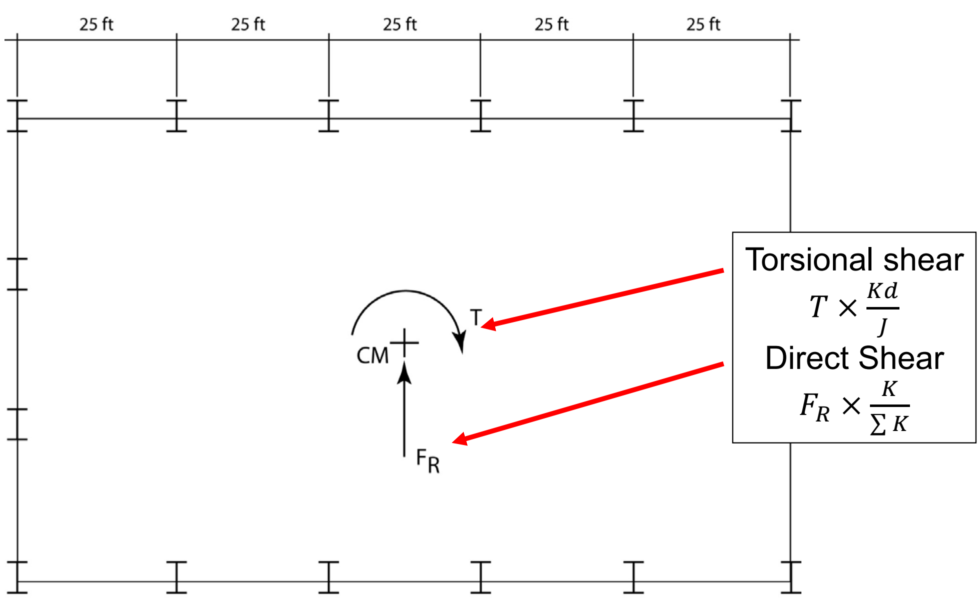
2 and 3   1 and 4   A and F   B and E   C and D

Building 1 Floor Plan



Typical Floor Plan

Equivalent Seismic Loads



Equivalent Seismic Loads

Equivalent Lateral Force Example

Distribution to roof

Direct load per frame

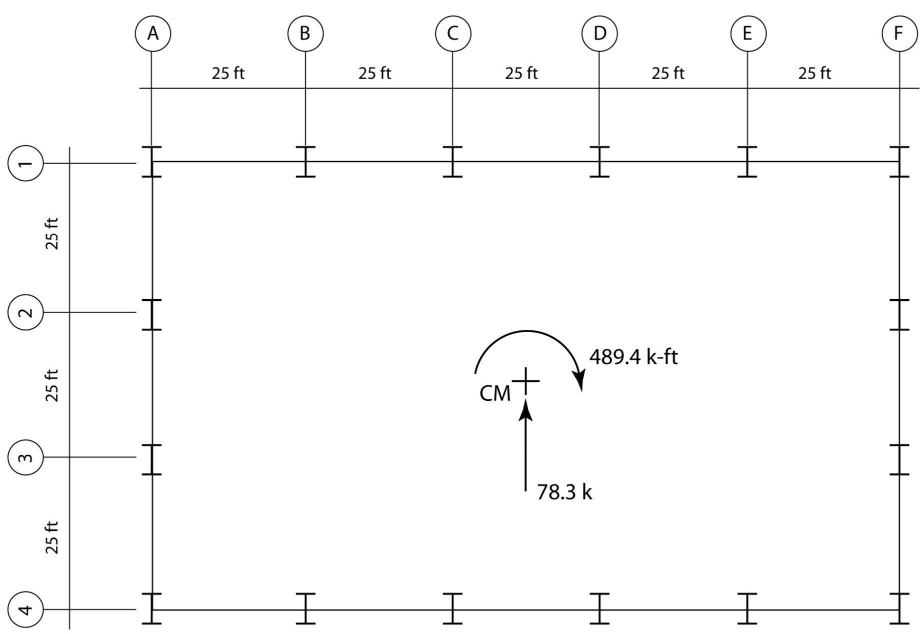
$\sum K = 2(1) + 4(1.25) = 7.0$

Frame *A* and *F* $= (78.3\ \text{k})\dfrac{1}{7.0} = 11.19\ \text{k}$

Frame *B* to *E* $= (78.3\ \text{k})\dfrac{1.25}{7.0} = 13.98\ \text{k}$

Loads to Frame 1 to 4 = 0

Building 1 Roof Seismic Loads



Roof Seismc Direct Shear and Torsion

Equivalent Lateral Force Example

Accidental eccentricity $= 0.05(125 \text{ ft}) = 6.25 \text{ ft}$

Torsion $= (6.25 \text{ ft})(78.3 \text{ k}) = 489.4 \text{ k} \cdot \text{ft}$

Load to Frame $A = 11.19 \text{ k} + \frac{(489.4 \text{ k·ft})(1.0)(62.5 \text{ ft})}{15{,}984} = 13.10 \text{ k}$

Load to Frame $F = 11.19 \text{ k} - \frac{(489.4 \text{ k·ft})(1.0)(62.5 \text{ ft})}{15{,}984} = 9.276 \text{ k } (11.19 \text{ k})$

Load to Frame $B = 13.98 \text{ k} + \frac{(489.4 \text{ k·ft})(1.25)(37.5 \text{ ft})}{15{,}984} = 15.42 \text{ k}$

Load to Frame $E = 13.98 \text{ k} - \frac{(489.4 \text{ k·ft})(1.25)(37.5 \text{ ft})}{15{,}984} = 12.55 \text{ k } (13.98 \text{ k})$

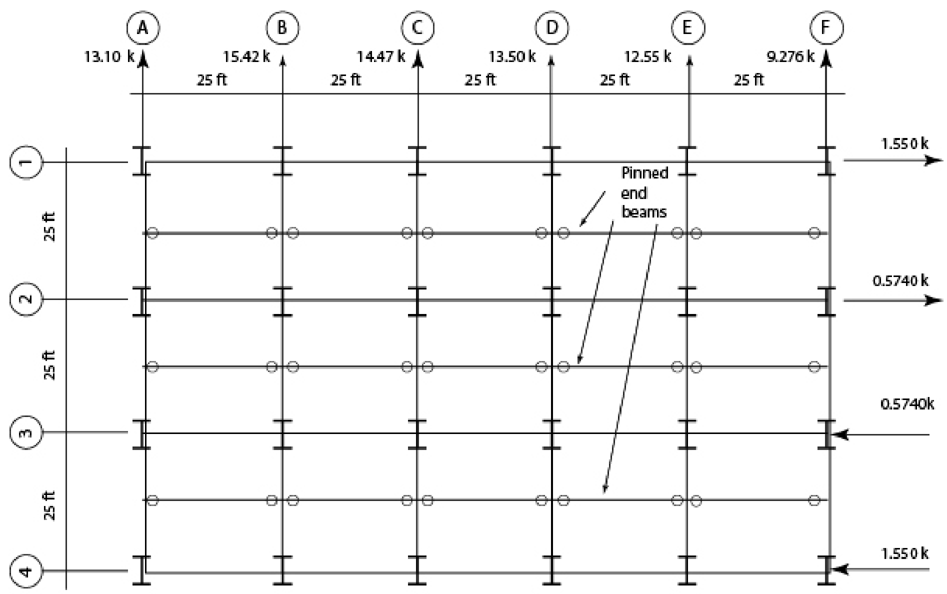
Load to Frame $C = 13.98 \text{ k} + \frac{(489.4 \text{ k·ft})(1.25)(12.5 \text{ ft})}{15{,}984} = 14.47 \text{ k}$

Load to Frame $D = 13.98 \text{ k} - \frac{(489.4 \text{ k·ft})(1.25)(12.5 \text{ ft})}{15{,}984} = 13.50 \text{ k } (14.68 \text{ k})$

Loads to Frame 1 and 4 $= \frac{(489.4 \text{ k·}ft)(1.35)(37.5 \text{ ft})}{15{,}984} = \pm 1.550 \text{ k}$

Loads to Frame 2 and 3 $= \frac{(489.4 \text{ k·ft})(1.5)(12.5 \text{ ft})}{15{,}984} = \pm 0.5740 \text{ k}$

Building 1 Roof Equivalent Seismic Loads



Roof Seismc Loads

Equivalent Lateral Force Example

Distribution to eighth floor

Direct load per frame

$\sum K = 2(1) + 4(1.25) = 7.0$

Frame A and F $= (107.9\text{ k})\frac{1}{7.0} = 15.41\text{ k}$

Frame B to E $= (107.9\text{ k})\frac{1.25}{7.0} = 19.27\text{ k}$

Loads to Frame 1 to 4 = 0

Equivalent Lateral Force Example

Torsion $= (6.25\text{ ft})(107.9\text{ k}) = 674.4\text{ k}\cdot\text{ft}$

Load to Frame $A = 15.41\text{ k} + \frac{(674.4\text{ k}\cdot\text{ft})(1.0)(62.5\text{ ft})}{15{,}984} = 18.05\text{ k}$

Load to Frame $F = 15.41\text{ k} - \frac{(674.4\text{ k}\cdot\text{ft})(1.0)(62.5\text{ ft})}{15{,}984} = 12.77\text{ k}\ (15.41\text{ k})$

Load to Frame $B = 19.27\text{ k} + \frac{(674.4\text{ k}\cdot\text{ft})(1.25)(37.5\text{ ft})}{15{,}984} = 21.25\text{ k}$

Load to Frame $E = 19.27\text{ k} - \frac{(674.4\text{ k}\cdot\text{ft})(1.25)(37.5\text{ ft})}{15{,}984} = 17.29\text{ k}\ (19.27\text{ k})$

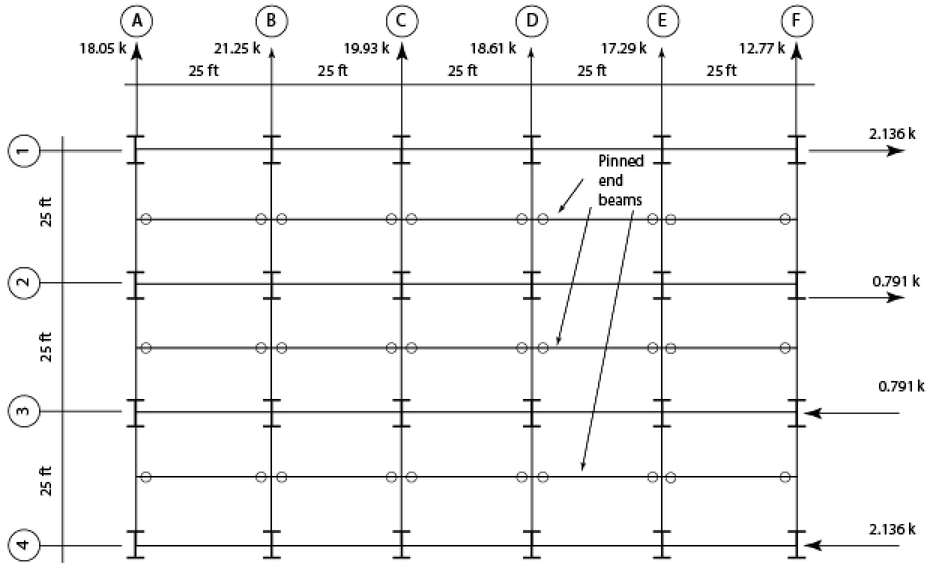
Load to Frame $C = 19.27\text{ k} + \frac{(674.4\text{ k}\cdot\text{ft})(1.25)(12.5\text{ ft})}{15{,}984} = 19.93\text{ k}$

Load to Frame $D = 19.27\text{ k} - \frac{(674.4\text{ k}\cdot\text{ft})(1.25)(12.5\text{ ft})}{15{,}984} = 18.61\text{ k}\ (19.27\text{ k})$

Loads to Frame 1 and 4 $= \frac{(674.4\text{ k}\cdot ft)(1.35)(37.5\text{ ft})}{15{,}984} = \pm 2.136\text{ k}$

Loads to Frame 2 and 3 $= \frac{(674.4\text{ k}\cdot\text{ft})(1.5)(12.5\text{ ft})}{15{,}984} = \pm 0.791\text{ k}$

Building 1 Eight Floor Equivalent Seismic Loads



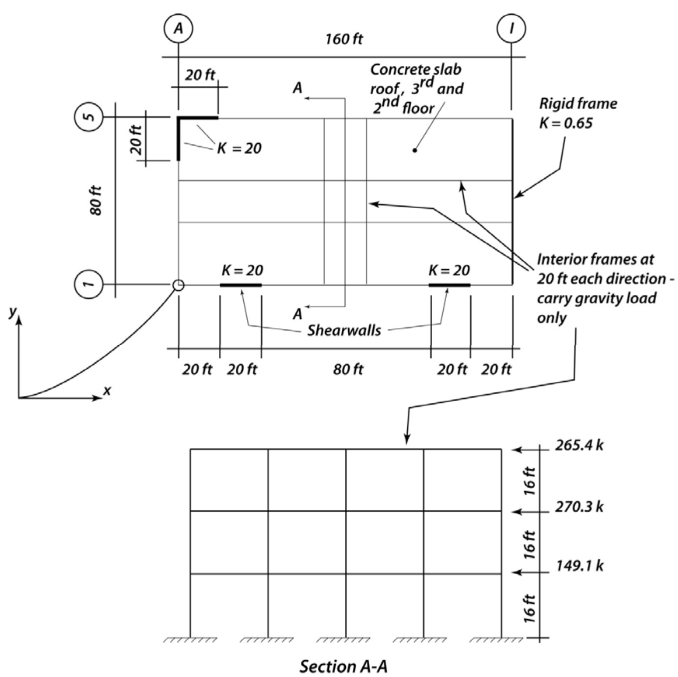
Eighth Floor Seismc Loads

Equivalent Lateral Force Method Example – Building 2

Equivalent Lateral Force Example – Building 2

For the level between the third floor and the roof, determine the force in each of the lateral force resisting elements in Building 2. Dead loads are 60 psf, 90 psf and 100 psf for the roof, third level and second level, respectively. The shear wall elements are special reinforced concrete shear walls (R = 5). Use I_e = 1.

Equivalent Lateral Force Example – Building 2



Equivalent Lateral Force Example – Building 2

Wall weight

One story $= (4\text{ walls})(20\text{ ft})(16\text{ ft})(70\text{ psf}) = 89{,}600\text{ lb} = 89.6\text{ k}$

Total for building $= 2.5(89.6\text{ k}) = 224\text{ k}$

$$W = \frac{(160\text{ ft})(80\text{ ft})(60+90+100)\text{ psf}}{1{,}000\text{ lb/k}} + 224\text{ k} = 3{,}424\text{ k}$$

$T = T_a = 0.393\text{ s}$

$S_{DS} = 1$

$S_{D1} = 0.52$

$$C_s = \frac{S_{DS}}{R/I_e} = \frac{1}{5/1} = 0.2$$

$0.044(0.52)(1) = 0.0229 > 0.01$

$$0.0229 \le C_s \le \frac{0.52}{(0.393\text{ s})(5/1)} = 0.265$$

$C_s = 0.2$

$V = 0.2(3{,}424\text{ k}) = 684.8\text{ k}$

Equivalent Lateral Force Example – Building 2

Calculate the load tributary to each story of the building

$A = (160\text{ ft})(80\text{ ft}) = 12{,}800\text{ ft}^2$

$$w_2 = \frac{(12{,}800\text{ ft}^2)(100\text{ psf}) + 4(20\text{ ft})(16\text{ ft})(70\text{ psf})}{1{,}000\text{ lb/k}} = 1{,}370\text{ k}$$

$$w_3 = \frac{(12{,}800\text{ ft}^2)(100\text{ psf}) + 4(20\text{ ft})(16\text{ ft})(70\text{ psf})}{1{,}000\text{ lb/k}} = 1{,}242\text{ k}$$

$$w_r = \frac{(12{,}800\text{ ft}^2)(60\text{ psf}) + 4(20\text{ ft})(8\text{ ft})(70\text{ psf})}{1{,}000\text{ lb/k}} = 812.8\text{ k}$$

Vertical Force Distribution – Building 2

Vertical Force Distribution

ASCE Section 12.8.3

$F_x = C_{vx} V$ (Eq. 12.8-11)

$C_{vx} = \frac{w_x h_x^k}{\sum_i w_i h_i^k}$ (Eq. 12.8-12)

k = 1 for structures with a period less than or equal to 0.5 s

k = 2 for structures with a period at least equal to 2.5 s

Use linear interpolation between 1 and 2 for other period values

Equivalent Lateral Force Example

ASCE Section 12.8.3

$F_x = C_{vx}V$ (Eq. 12.8-11)

$C_{vx} = \frac{w_x h_x^k}{\sum_i w_i h_i^k}$ (Eq. 12.8-12)

k = 1 (for a period less than or equal to 0.5 s)

$\sum_i w_i h_i^k = (1{,}370\text{ k})(16\text{ ft}) + (1{,}240\text{ k})(32\ ft) + (813\text{ k})(48\text{ ft}) = 100{,}660\text{ k}\cdot\text{ft}$

$C_{v2} = \frac{(1{,}370\text{ k})(16\text{ ft})}{100{,}660\text{ k}\cdot\text{ft}} = 0.2177$

$C_{v3} = \frac{(1{,}242\text{ k})(32\text{ ft})}{100{,}660\text{ k}\cdot\text{ft}} = 0.3947$

$C_{vr} = \frac{(812.8\text{ k})(48\text{ ft})}{100{,}660\text{ k}\cdot\text{ft}} = 0.3876$

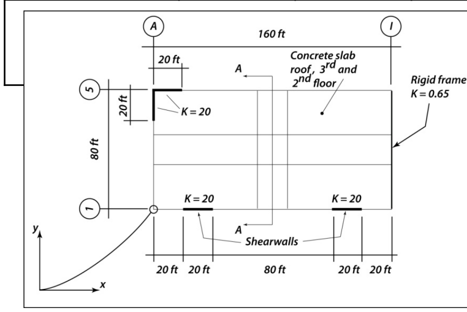
$F_2 = 0.2177(684.8\text{ k}) = 149.1\text{ k}$

Story Loads		
Story	C_{vx}	Load (k)
2nd	0.2177	149.1
3rd	0.3947	270.3
Roof	0.3876	265.4
Total		684.8

Horizontal Force Distribution – Building 2

Equivalent Lateral Force Example

Wall/Frame	x	y	K_x	K_y	K_x*y	K_y*x
	ft	ft			ft	ft
Line A	0	70	0	20	0	0
Line I	160	40	0	0.65	0	104
Line 1-1	30	0	20	0	0	0
Line 1-2	130	0	20	0	0	0
Line 5	10	80	20	0	1600	0
Σ			60	20.7	1600	104
				x_{cr}	5.02	ft
				y_{cr}	26.7	ft



$$x_{cr} = \frac{\sum K_y x}{\sum K_y} = \frac{104}{20.7} = 5.02$$

$$y_{cr} = \frac{\sum K_x y}{\sum K_x} = \frac{1,600}{60} = 26.7$$

Equivalent Lateral Force Example

$$x_{cm} = \frac{160\text{ ft}}{2} = 80\text{ ft}$$

$$y_{cm} = \frac{80\text{ ft}}{2} = 40\text{ ft}$$

$$e_{ix} = 80\text{ ft} - 5.02\text{ ft} = 75.0\text{ ft}$$

$$e_{iy} = 40\text{ ft} - 26.7\text{ ft} = 13.3\text{ ft}$$

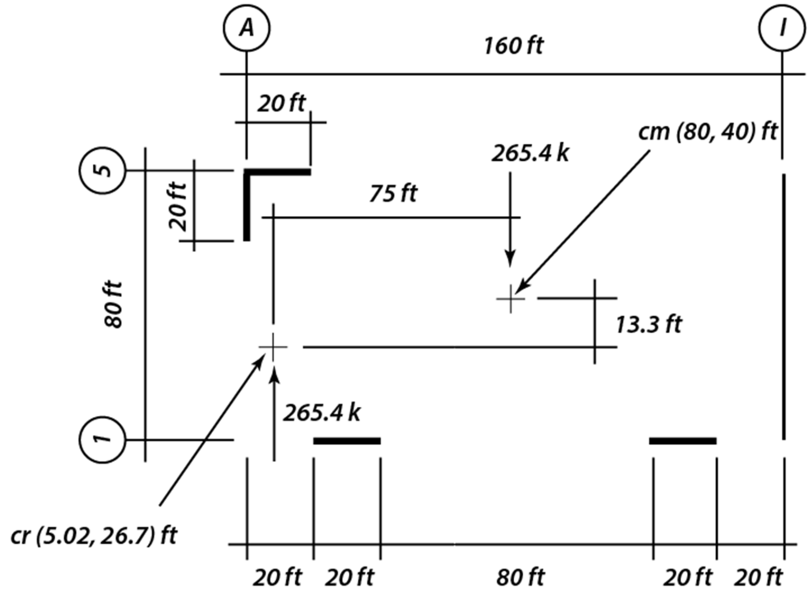
$$e_{ax} = 0.05(160\text{ ft}) = 8\text{ ft}$$

$$e_{ay} = 0.05(80\text{ ft}) = 4\text{ ft}$$

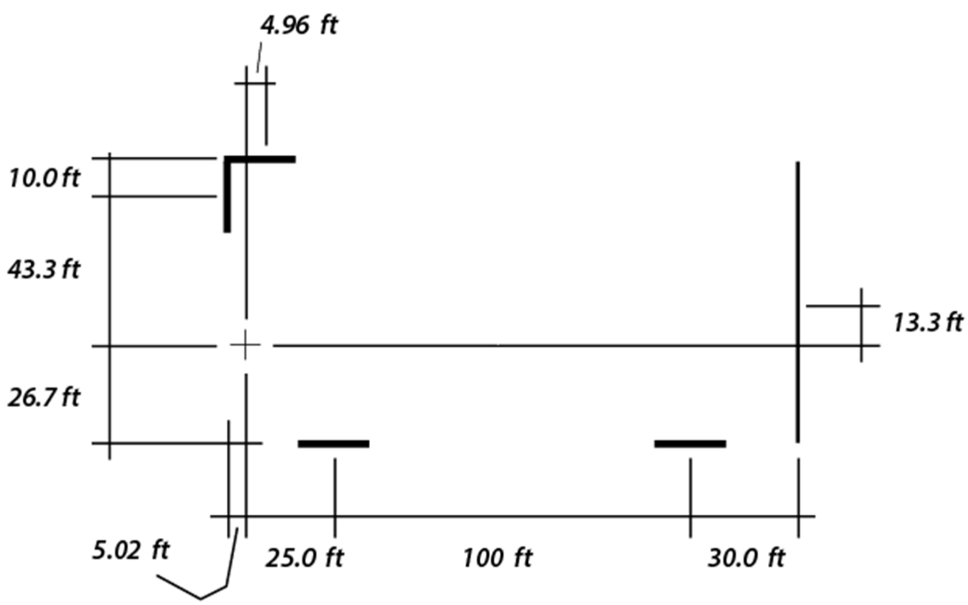
$$(265.4\text{ k})(75.0\text{ ft} + 8\text{ ft}) = 22,028\text{ k}\cdot\text{ft}$$

Eccentricities and Torsion, V = 265.4 k						
	Inherent	Accidental	Inherent + Accidental	Inherent - Accidental	Torsion +	Torsion -
	(ft)	*(ft)*	*(ft)*	*(ft)*	*(k-ft)*	*(k-ft)*
x	75	8	83	67	22,028	17,782
y	13.3	4	17.3	9.3	4,591	2,468

CM and CR



Wall Locations Relative to CR



Equivalent Lateral Force Example

$$J = \sum K_x d_y^2 + \sum K_y d_x^2$$

Direct shear force = Story force $\times \frac{K}{\sum K}$

$$(265.4\ k)\left(\frac{20}{20.7}\right) = 256.4$$

Torsional shear = Torsional moment $\times \frac{Kd}{J}$

$$(22{,}028\ \text{k} \cdot \text{ft})\left(\frac{-100\ \text{ft}}{102{,}000\ \text{ft}^2}\right) = -21.6\ \text{k}$$

3rd Floor to Roof Level Distribution, V = 265.4 k										
Wall / Frame	**K_x**	**K_y**	**d_x**	**d_y**	**$K_x d_y$ or $K_y d_x$**	**$K_x{*}d_y^2$**	**$K_y{*}d_x^2$**	**Direct Shear**	**Torsional Shear**	**Total Shear**
			(ft)	*(ft)*	*(ft)*	*(ft²)*	*(ft²)*	*(k)*	*(k)*	*(k)*
Line A	0	20	-5.02	43.3	-100	0	504	256.4	-21.6	256.4
Line I	0	0.65	155	13.3	101	0	15,600	8.33	21.8	30.1
Line 1-1	20	0	25	-26.7	-534	14,300	0	0	-115	115
Line 1-2	20	0	125	-26.7	-534	14,300	0	0	-115	115
Line 5	20	0	4.98	53.3	1,070	56,800	0	0	231	231
						J	102,000	265		

ASCE 7-10 Chapters 26 to 31
Wind Loads
(IBC Section 1609.1.1 incorporates ASCE 7-10 by reference)

ASCE 7-10 versus ASCE 7-05

Chapter 6 – Reserved for future use

Chapter 26 – General Requirements

Chapter 27 – MWFRS (Directional Procedure)

Chapter 28 – MWFRS (Envelope Procedure)

Chapter 29 – MWFRS - Other Structures and Building Appurtenances

Chapter 30 – Components and Cladding (C&C)

Chapter 31 – Wind Tunnel Procedure

ASCE 7-10 versus ASCE 7-05

- Three wind speed maps versus one. Called wind hazard maps in ASCE 7 and ultimate wind speed maps in 2012 IBC.
- Wind speeds vary with risk category.
- Revised load factors for wind in allowable stress design (ASD) and load and resistance factor design (LRFD) load combinations
- Removal of the Importance Factor (I)

ASCE 7-10 versus ASCE 7-05

- Reinstatement of the applicability of Exposure *D* in hurricane prone regions
- Revised wind speed triggers for definition of hurricane prone region and wind-borne debris region
- Revised pressure values for minimum design loads

Chapter 26 – General Requirements

- Wind speed
- Wind directionality
- Exposure category
- Topographic effect gust effect factor enclosure classification
- Wind-borne debris regions
- Internal pressure coefficients
- Symbols
- Definitions

Chapter 27 – MWFRS (Directional Procedure)

Part 1 – Analytical procedure

- Enclosed, partially enclosed or open buildings (buildings of any height)
- Windward, leeward, and side walls and roof pressures, including internal pressures

Chapter 27 – MWFRS (Directional Procedure)

Part 2 – Simplified procedure

- Roof height less than or equal to 160 feet
- Enclosed buildings
- Simple diaphragms

Chapter 28 – MWFRS (Envelope Procedure)

Low-rise buildings – roof height less than or equal to 60 feet, height does not exceed lesser horizontal dimension

Part 1 – Analytical procedure

Windward, leeward, and side walls and roof pressures

Part 2 – Simplified procedure

Horizontal and design pressures in tabular form

Chapter 30 – Components and Cladding

Part 1 – low-rise buildings

Part 2 – simplified approach for low-rise buildings

Part 3 – buildings of any height

Part 4 – simplified approach for buildings with roof height less than or equal to 160 feet

Part 5 – open buildings

Part 6 – building appurtenances, parapets

Definitions

MWFRS

Main wind force resisting system (MWFRS)

- An assemblage of structural elements assigned to provide support and stability for the overall structure. The system generally receives wind loading from more than one surface.

Components and Cladding (C&C)

Components and Cladding

- Elements of the building envelope that do not qualify as part of the MWFRS

Examples – roof decking, roof trusses, girts, steel wall panels, masonry walls

Element Classification

Some elements may comprise MWRFS for one loading and comprise C&C for another loading (for example, a masonry wall)

Basic Wind Speed (*V*)

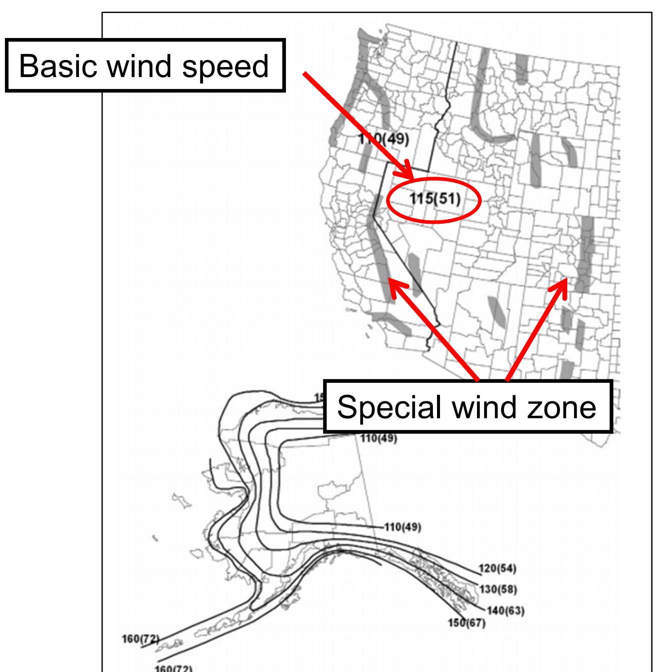
Three-second gust speed at 33 feet (10 m) above the ground in Exposure Category C

Determined as specified in Section 26.5.1

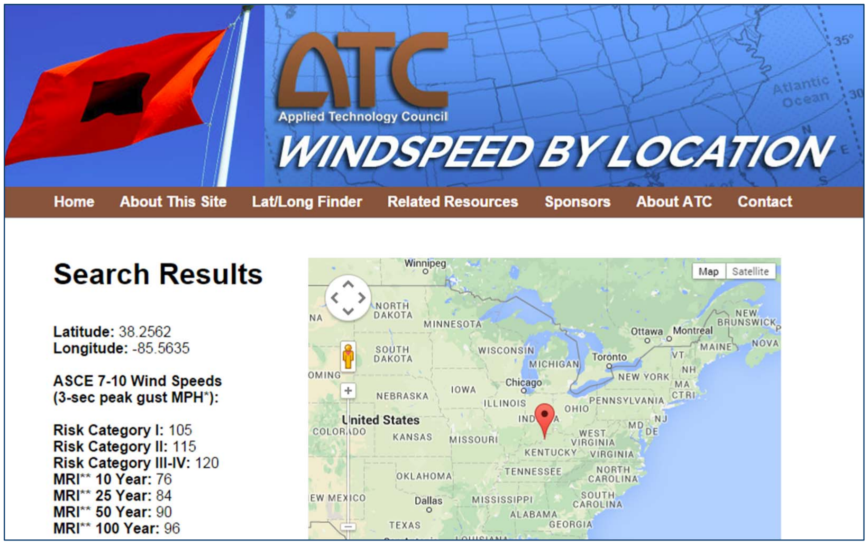
Use maps in figures 26.5-1A, 26.5-1B, 26.5-1C

Exposure categories are defined in Section 26.7.3

ASCE 7-10 Figure 26.5-1A Wind Hazard Map – Basic Wind Speed (*V*) – Risk Category II



http://windspeed.atcouncil.org/



Example
Building 3
Directional Procedure
Part 1

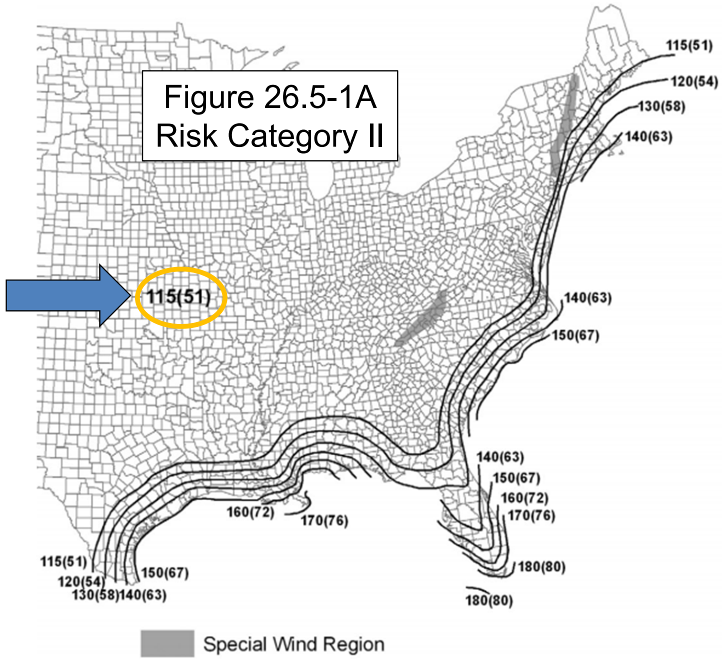
Design Parameters

- Analytical procedure Chapter 27
- Enclosed building
- Risk Category II
- Basic wind speed – Fig. 26.5-1A – 115 mph
- Exposure *C* (consider *B*?)
- G = 0.85 (gust-effect factor)

Design Parameters

- K_{zt} = 1 (topographic factor – Section 26.8)
- K_d = 0.85 (directional factor – Table 26.6-1)
- (GC_{pi}) = +/- 0.18 (internal pressure coefficient – Table 26.11-1)
- C_p – external pressure coefficient

Basic Wind Speed



MWFRS

Velocity Pressure (q_z)

Main Wind Force Resisting System – Part 1	All Heights
Velocity Pressure Exposure Coefficients, K_h and K_z	
Table 27.3-1	

Height Above Ground Level, z		Exposure		
ft	(m)	B	C	D
0-15	(0-4.6)	0.57	0.85	1.03
20	(6.1)	0.62	0.90	1.08
25	(7.6)	0.66	0.94	1.12
30	(9.1)	0.70	0.98	
40	(12.2)	0.76	1.04	
50	(15.2)	0.81	1.09	

Eq. 27.3-1

$$q_z = 0.00256K_zK_{zt}K_dV^2 = 0.00256(0.85)(1)(115)^2K_z = 28.78K_z$$

Height	q_z (psf)
0-15	24.46
15-20	25.90
20-25	27.05
25-30	28.20
30-40	29.93
40-48	31.37

External Pressure Coefficients C_p (Figure 27.4-1)

Figure 27.4-1

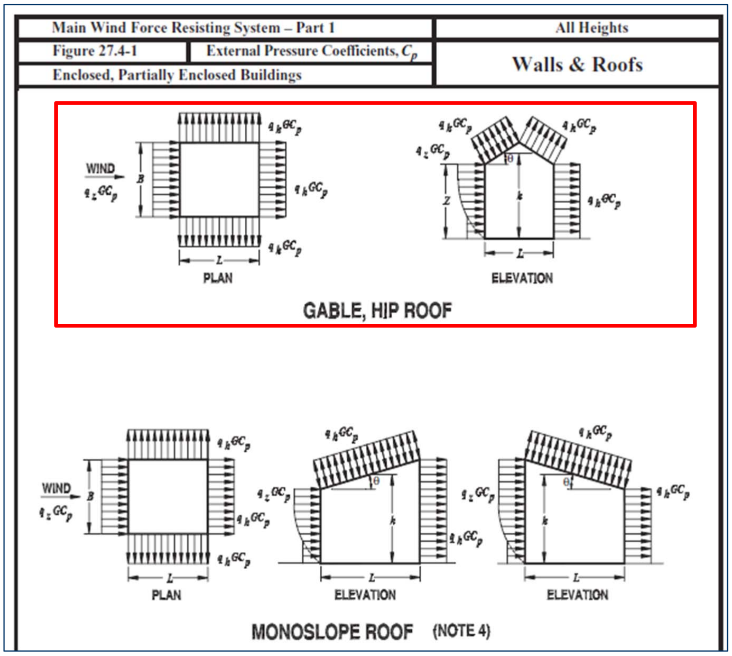


Figure 27.4-1

Main Wind Force Resisting System – Part 1		All Heights
Figure 27.4-1 (cont.)	External Pressure Coefficients, C_p	Walls & Roofs
Enclosed, Partially Enclosed Buildings		

Wall Pressure Coefficients, C_p			
Surface	**L/B**	**C_p**	**Use With**
Windward Wall	All Values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥ 4	-0.2	
Side Wall	All Values	-0.7	q_h

External Pressure Coefficients – C_p – Walls

- Windward wall = 0.8
- Sidewalls = -0.7
- Leeward walls – a function of *L*/*B*
  - *L*/*B* (160/80 = 2) is -0.3
  - *L*/*B* (80/160 = 0.5) is -0.5

External Roof Pressure Coefficients – C_p – Roof

Roof Pressure Coefficients, C_p, for Use with q_h

Wind Direction	Windward									Leeward		
	Angle, θ (degrees)									Angle, θ (degrees)		
	h/L	10	15	20	25	30	35	45	≥ 60#	10	15	≥ 20
Normal to ridge for θ ≥ 10°	≤ 0.25	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.3	-0.2 0.3	0.0* 0.4	0.4	0.01 θ	-0.3	-0.5	-0.6
	0.5	-0.9 -0.18	-0.7 -0.18	-0.4 0.0*	-0.3 0.2	-0.2 0.2	-0.2 0.3	0.0* 0.4	0.01 θ	-0.5	-0.5	-0.6
	≥ 1.0	-1.3** -0.18	-1.0 -0.18	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.2	0.0* 0.3	0.01 θ	-0.7	-0.6	-0.6

Wind Direction	h/L	Horizontal distance from windward edge	C_p
Normal to ridge for θ < 10 and Parallel to ridge for all θ	≤ 0.5	0 to h/2	-0.9, -0.18
		h/2 to h	-0.9, -0.18
		h to 2h	-0.5, -0.18
		> 2h	-0.3, -0.18
	≥ 1.0	0 to h/2	-1.3**, -0.18
		> h/2	-0.7, -0.18

* Value is provided for interpolation purposes.
** Value can be reduced linearly with area over which it is applicable as follows

Area (sq ft)	Reduction Factor
≤ 100 (9.3 sq m)	1.0
250 (23.2 sq m)	0.9
≥ 1,000 (92.9 sq m)	0.8

External Pressure Coefficients – C_p – Roof

- A function of slope (0°) and *h*/*L* (48/160 = 0.3), (48/80 = 0.6)
- *h*/*L* = 0.3, 0 to *h*, -0.9 and -0.18

  h to *2h,* -0.5 and -0.18

  > 2h, -0.3 and -0.18

External Pressure Coefficients – C_p – Roof

- *h*/*L* = 0.6, interpolate:

$(160)(48/2) = 3{,}840$ ft^2

$(-1.3)(0.8) = -1.04$

0 to *h*/2

$$-0.9 + \frac{(-0.6-0.5)}{(1.0-0.5)}\big(-1.04 - (-0.9)\big) = -0.93$$

> *h*/2

$$-0.5 + \frac{(0.6-0.5)}{(1.0-0.5)}\big(-0.7 - (-0.5)\big) = -0.54$$

Design Pressures – MWFRS

ASCE 7-10 Eq. 27.4-1

$$p = q_z GC_p - q_h(GC_{pi})$$

$$p_{0-15} = (24.46)(0.85)(0.8) - (31.37)(\pm 0.18) = 16.63 \pm 5.647 \text{ psf}$$

Height	q_z (psf)
0-15	24.46
15-20	25.90
20-25	27.05
25-30	28.20
30-40	29.93
40-48	31.37

Windward Wall	
Height	*p* (psf)
0-15	16.63 +/- 5.647
15-20	17.61 +/- 5.647
20-25	18.39 +/- 5.647
25-30	19.18 +/- 5.647
30-40	20.35 +/- 5.647
40-48	21.33 +/- 5.647

Design Pressures

$$p = q_h GC_p - q_h(GC_{pi})$$

$$p = (31.37)(0.85)(-0.7) - (31.37)(\pm 0.18) = -18.67 \pm 5.647 \text{ psf}$$

Element	*p* (psf)
Sidewall	-18.67 +/- 5.647
Leeward wall (-0.5)	-13.33 +/- 5.647
Leeward wall (-0.3)	-8.00 +/- 5.647

Design Pressures

$$p = q_h G C_p - q_h (G C_{pi})$$

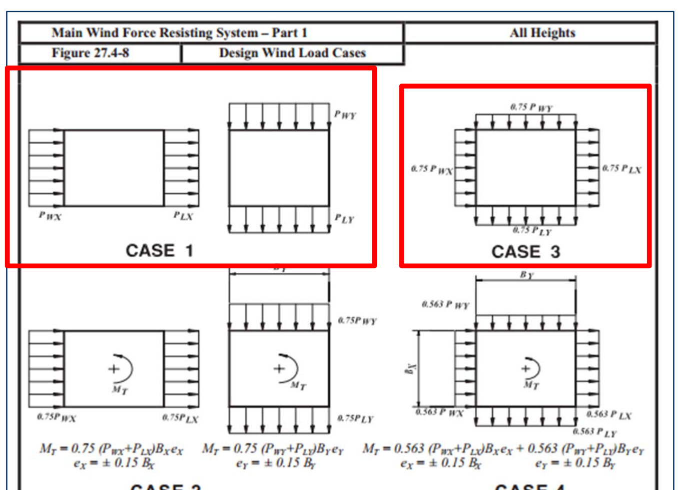
$$p = (31.37)(0.85)(-0.9) - (31.37)(\pm 0.18) = -24.0 \pm 5.647 \text{ psf}$$

$$p = (31.37)(0.85)(-0.18) - (31.37)(\pm 0.18) = -4.80 \pm 5.647 \text{ psf}$$

Roof			
$h/L = 0.3$		$h/L = 0.6$	
Distance	For C_p *Slide 101*	Distance	For C_p *Slide 102*
0 - *h*	-24.0/ -4.80	0 – *h*/2	-24.8 / -4.80
h - 2*h*	-13.3 / -4.80	> *h*/2	-14.4 / -4.80
> 2*h*	-8.0 / -4.80	-----	-----

Load Cases

If the building is torsionally regular under wind load, only Cases 1 and 3 must be considered –



Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

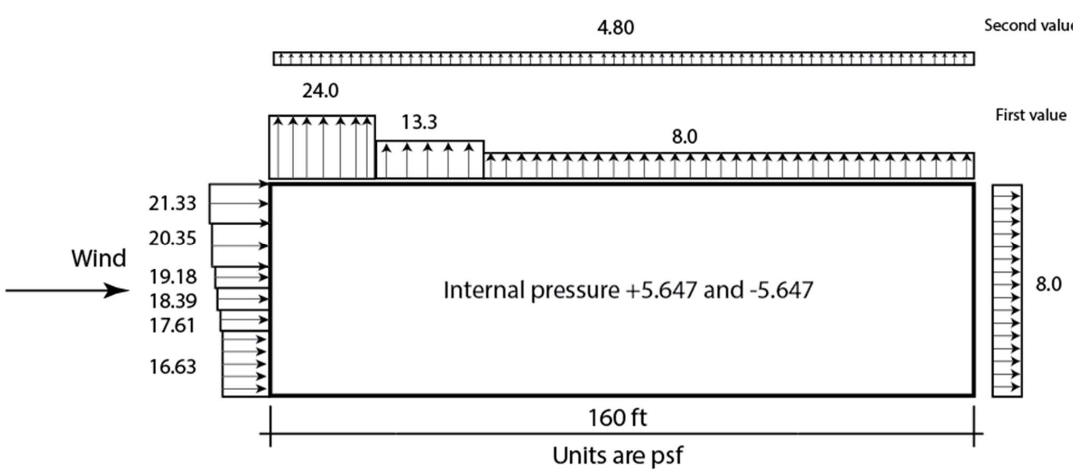
Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

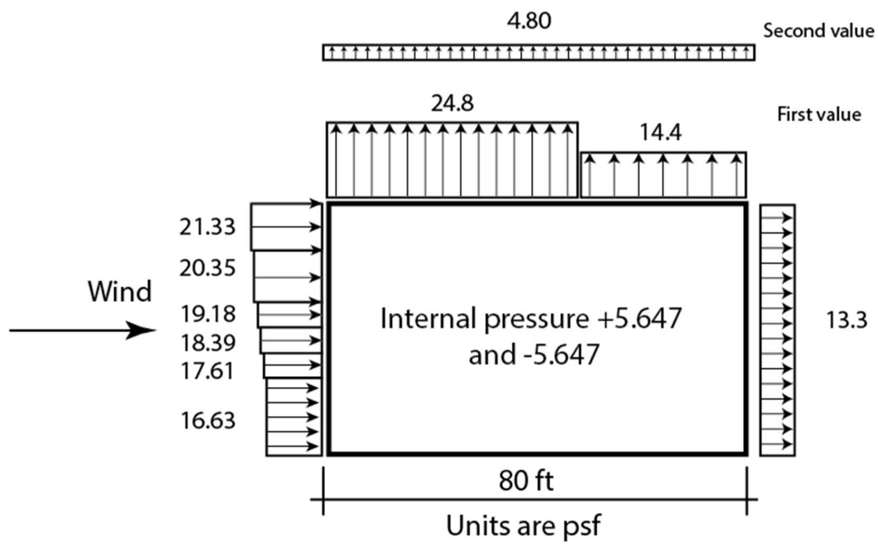
Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Load Case 1

Design Pressures – *h/L=0.3*

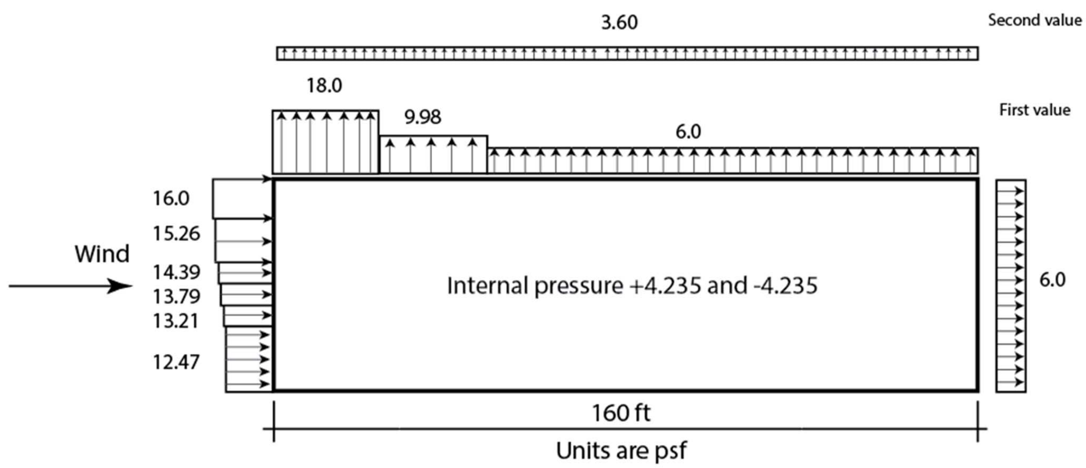


Design Pressures – *h/L=0.6*

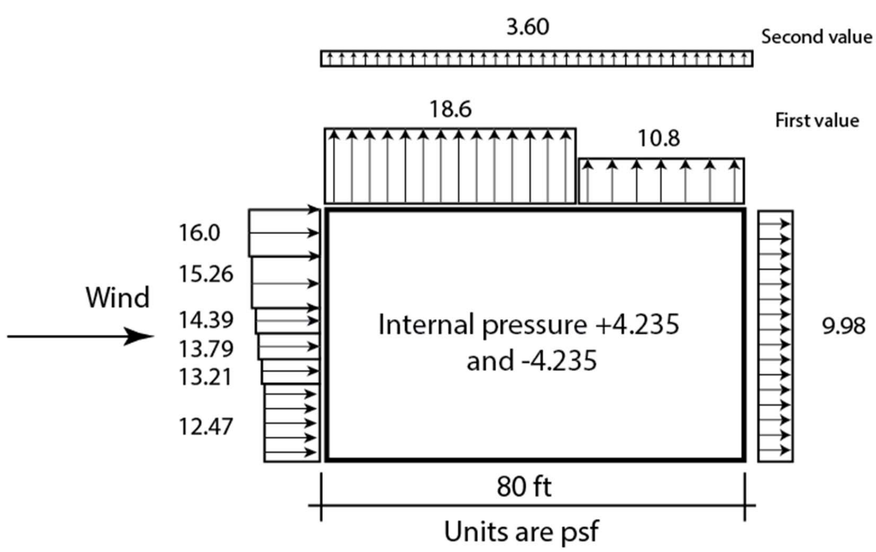


Load Case 3

Design Pressures



Design Pressures



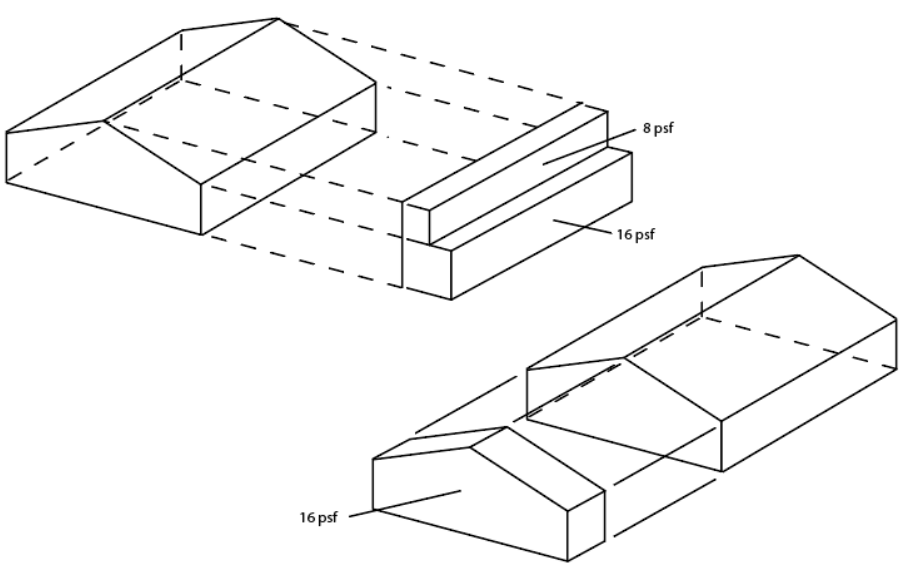
Minimum Loads

Section 27.1.5 Minimum Design Wind Loads

For enclosed and partially enclosed buildings:

- 16 psf times wall area for walls
- 8 psf times roof area for roofs

Minimum Loads



Minimum Loads

Load Case 1 – wind parallel to 160 foot dimension

Load on windward wall in combination with wind on projected area of windward roof

Wall area = (48 ft)(80 ft) = 3,840 ft^2

Vertical projection of roof area = 0

(3,840 ft^2)(16 psf) = 61,440 lb

C&C
Chapter 30 – Part 1

Effective Wind Area

"The area used to determine (GC_p). For component and cladding elements, the effective wind area in Figs. 30.4-1 through 30.4-7, 30.5-1, 30.6-1, and 30.8-1 through 30.8-3 is the span length multiplied by an effective width that need not be less than one-third the span length. For cladding fasteners, the effective wind area shall not be greater than the area that is tributary to an individual fastener."

May be different from tributary area.

Design Pressures – C&C

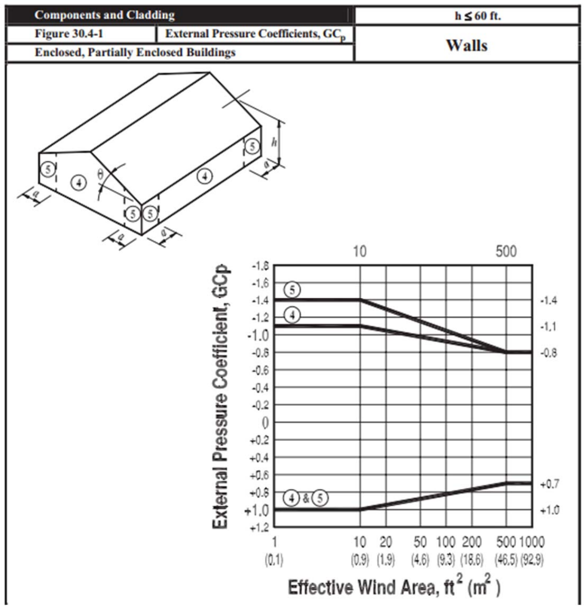
$$p = q_h\left[(GC_p) - (GC_{pi})\right]$$

$$q_h = 31.37 \text{ psf}$$

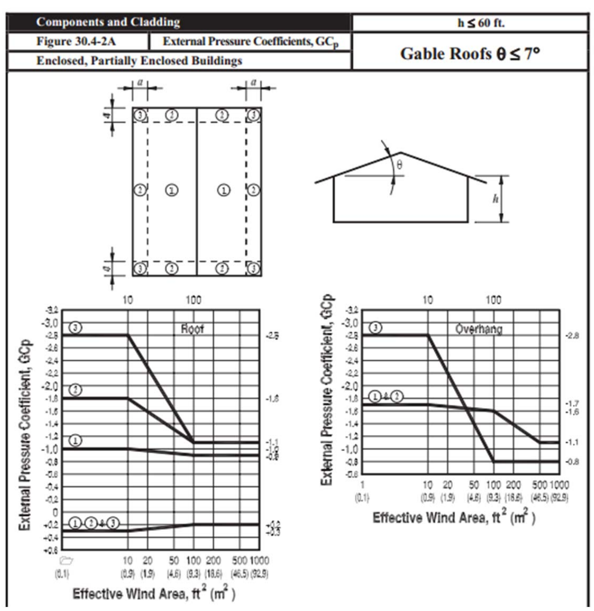
$$(GC_{pi}) = \pm 0.18$$

(GC_p) values obtained from Figures 30.4-1 and 30.4-2A of the Standard – they are a function of effective area and zone

Zones and (GC_p) for Walls



Zones and (GC_p) for Roofs



Calculation of ‘*a*’

Smaller of:

10% of least horizontal dimension =

0.10(80 ft) = 8 ft ←

0.4h = 0.4(48 ft) = 19.2 ft

But not less than 4% of least horizontal dimension =

0.04(80 ft) = 3.2 ft

Or 3 ft

∴ *a* = 8 ft

C&C – Walls

Effective wind area – assume that walls are supported at floor level – effective width need not be less than one-third of the length

$$16(16/3) = 85.33 \text{ ft}^2$$

C&C – Walls

Figure 30.4-1, Note 5 suggests a 10% reduction in tabulated values for roof slopes less than 10° – Equations are from Mehta & Coulbourne

Zone 4

$$(GC_p) = 1.1766 - 0.1766 \log A =$$

$$1.1766 - 0.1766 \log(85.33 \text{ ft}^2) = 0.836$$

$$(0.9)(0.836) = 0.752$$

$$(GC_p) = -1.2766 + 0.1766 \log A =$$

$$-1.2766 + 0.1766 \log(85.33 \text{ ft}^2) = -0.936$$

$$(0.9)(-0.936) = -0.842$$

C&C – Walls

Zone 5

$$(GC_p) = 1.1766 - 0.1766 \log A =$$

$$1.1766 - 0.1766 \log(85.33 \text{ ft}^2) = 0.836$$

$$(0.9)(0.836) = 0.752$$

$$-1.7532 + 0.3532 \log A = -1.7532 + 0.3532 \log(85.33 \text{ ft}^2) = -1.07$$

$$(0.9)(-1.07) = -0.963$$

C&C – Walls

$$p = q_h[(GC_p) - (GC_{pi})]$$

Zone 4

$$p = (31.37)[-0.842 - (+0.18)] = -32.06 \text{ psf}$$

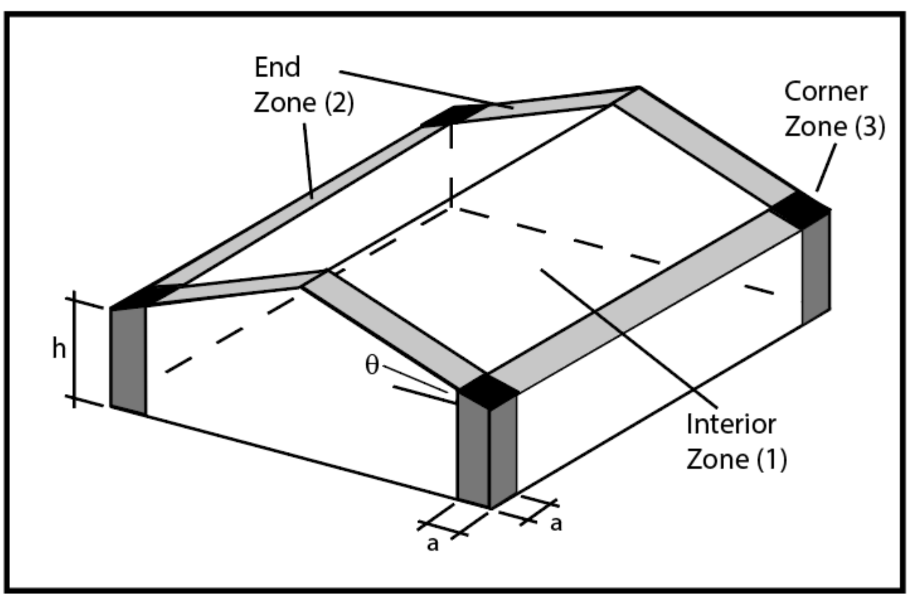
$$p = (31.37)[0.752 - (-0.18)] = 29.24 \text{ psf}$$

Zone 5

$$p = (31.37)[-0.963 - (+0.18)] = -35.86 \text{ psf}$$

$$p = (31.37)[0.752 - (-0.18)] = 29.24 \text{ psf}$$

Roof Zones



Effective Wind Area – Roof Joists

Roof joists span the 80 ft dimension and are spaced at 10 ft

Tributary area, interior joist $= (10 \text{ ft})(80 \text{ ft}) = 800 \text{ ft}^2$

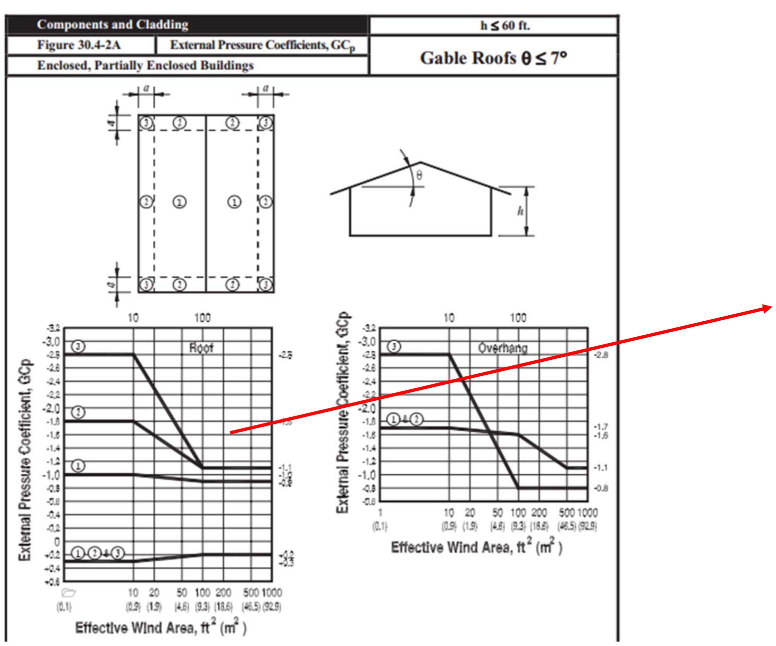
Span = 80 ft

Effective width need not be less than $\frac{80 \text{ ft}}{3} = 26.7 \text{ ft}$

Use 26.7 ft

Effective wind area $= (80 \text{ ft})(26.7 \text{ ft}) = 2{,}136 \text{ ft}^2$

C&C – Roof Joists



Zone 1, 2, 3

$(GC_p) = 0.2$

Zone 1

$(GC_p) = -0.9$

Zone 2, 3

$(GC_p) = -1.1$

C&C – Roof Joists

$p = q_h[(GC_p) - (GC_{pi})]$

Zone 1 (Interior)

$p = (31.37)[-0.9 - (+0.18)] = -33.89 \text{ psf}$

$p = (31.37)[0.2 - (-0.18)] = 11.92 \text{ psf}$

Zone 2 (End or Eave)

$p = (31.37)[-1.1 - (+0.18)] = -40.15 \text{ psf}$

$p = (31.37)[0.2 - (-0.18)] = 11.92 \text{ psf}$

Zone 3 (Corner)

$p = (31.37)[-1.1 - (+0.18)] = -40.15 \text{ psf}$

$p = (31.37)[0.2 - (-0.18)] = 11.92 \text{ psf}$

Effective Wind Area – Roof Deck

Span = 10 ft

Width $= \frac{10 \text{ ft}}{3} = 3.33 \text{ ft}$

Effective wind area $(10 \text{ ft})(3.33 \text{ ft}) = 33.3 \text{ ft}^2$

C&C – Roof Deck

Zone 1

$$(GC_p) = 0.4000 - 0.1000 \log A =$$

$$0.4000 - 0.1000 \log(33.3 \text{ ft}^2) = 0.247$$

$$(GC_p) = -1.1000 + 0.1000 \log A$$

$$= -1.1000 + 0.1000 \log(33.3 \text{ ft}^2)$$

$$= -0.948$$

C&C – Roof Deck

Zone 2

$$(GC_p) = 0.4000 - 0.1000 \log A =$$

$$0.4000 - 0.1000 \log(33.3 \text{ ft}^2) = 0.247$$

$$(GC_p) = -2.5000 + 0.7000 \log A$$

$$= -2.5000 + 0.7000 \log(33.3 \text{ ft}^2)$$

$$= -1.43$$

C&C – Roof Deck

Zone 3

$$(GC_p) = 0.4000 - 0.1000 \log A =$$

$$0.4000 - 0.1000 \log(33.3 \text{ ft}^2) = 0.247$$

$$(GC_p) = -4.5000 + 1.7000 \log A$$

$$= -4.5000 + 1.7000 \log(33.3 \text{ ft}^2)$$

$$= -1.91$$

C&C – Roof Deck

$$p = q_h[(GC_p) - (GC_{pi})]$$

Zone 1 (Interior)

$$p = (31.37)[-0.948 - (+0.18)] = -35.39 \text{ psf}$$

$$p = (31.37)[0.247 - (-0.18)] = 13.39 \text{ psf}$$

Zone 2 (End or Eave)

$$p = (31.37)[-1.43 - (+0.18)] = -50.51 \text{ psf}$$

$$p = (31.37)[0.247 - (-0.18)] = 13.39 \text{ psf}$$

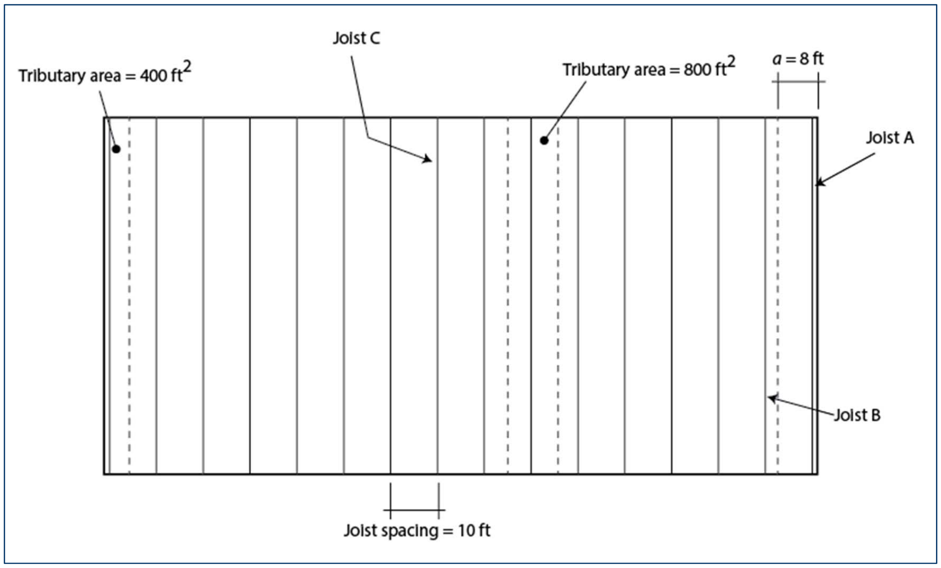
Zone 3 (Corner)

$$p = (31.37)[-1.91 - (+0.18)] = -65.56 \text{ psf}$$

$$p = (31.37)[0.247 - (-0.18)] = 13.39 \text{ psf}$$

Design Loads for Roof Joists

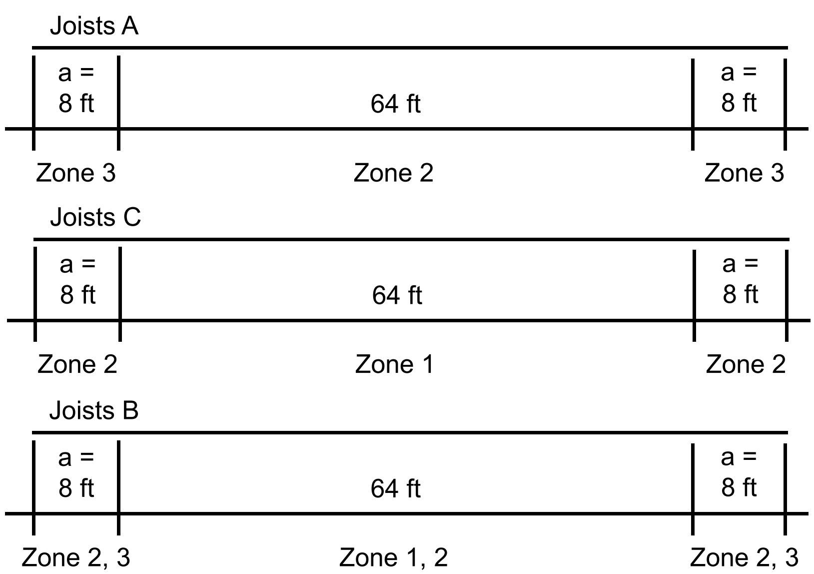
Building 2 – Roof Plan



Design Loads for Roof Joists

- Joists adjacent to walls (Joists A) are in Zones 2 and 3
- Next joists toward interior of building (Joists B) are in all three zones
- All other joists (Joists C) are in Zones 1 and 2

Design Loads for Roof Joists



Design Loads – Roof Joists

Load Combination 1

$$1.4D = 1.4(60\ \text{psf}) = 84.0\ \text{psf}$$

Joists A

$$(84.0\ \text{psf})(5\ \text{ft}) = 420\ \text{lb/ft}$$

Joists B and C

$$(84.0\ \text{psf})(10\ \text{ft}) = 840\ \text{lb/ft}$$

Design Loads – Roof Joists

Load Combination 2

$$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$$

$$= 1.2(60\ \text{psf}) + 0.5(16\ \text{psf}) = 80.0\ \text{psf}$$

Joists A

$$(80.0\ \text{psf})(5\ \text{ft}) = 400\ \text{lb/ft}$$

Joists B and C

$$(80.0\ \text{psf})(10\ \text{ft}) = 800\ \text{lb/ft}$$

Design Loads – Roof Joists

Load Combination 3

The downward acting wind load is 11.92 psf in all zones

$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5W \downarrow$

$= 1.2(60 \text{ psf}) + 1.6(16 \text{ psf}) + 0.5(11.92 \text{ psf}) = 103.6 \text{ psf}$

Joists A

$(103.6 \text{ psf})(5 \text{ ft}) = 518 \text{ lb/ft}$

Joists B and C

$(103.6 \text{ psf})(10 \text{ ft}) = 1040 \text{ lb/ft}$

Design Loads – Roof Joists

Load Combination 4

$1.2D + 0.5(L_r \text{ or } S \text{ or } R) + 1.0W \downarrow$

$= 1.2(60 \text{ psf}) + 0.5(16 \text{ psf}) + 1.0(11.92 \text{ psf}) = 91.9 \text{ psf}$

Joists A

$(91.9 \text{ psf})(5 \text{ ft}) = 460 \text{ lb/ft}$

Joists B and C

$(91.9 \text{ psf})(10 \text{ ft}) = 919 \text{ lb/ft}$

Design Loads – Roof Joists

Load Combination 5

$1.2D + E + L \rightarrow 1.2D$

Load Combination 7

$0.9D + E + L \rightarrow 0.9D$

Design Loads – Roof Joists

Load Combination 6

$0.9D + 1.0W(\uparrow)$

Joists A – Zones 2 and 3

Pressure is -40.15 psf for both zones

$[0.9(60\text{ psf}) + 1.0(-40.15\text{ psf})](5\text{ ft}) = 69.25\text{ lb/ft}(\downarrow)$

Note: net force is down because of the large dead load.

If the dead load was only 40 psf or less, the net force would be up (suction).

Design Loads – Roof Joists

Load Combination 6

$0.9D + 1.0W(\uparrow)$

Upward acting wind pressure varies with zone

Joists C – Zones 1 and 2

Pressure is -33.88 psf for Zone 1 and -40.15 psf for Zone 2

$[0.9(60 \text{ psf}) + 1.0(-33.88 \text{ psf})](10 \text{ ft}) = 201.2 \text{ lb/ft}(\downarrow)$

$[0.9(60 \text{ psf}) + 1.0(-40.15 \text{ psf})](10 \text{ ft}) = 138.5 \text{ lb/ft}(\downarrow)$

Net force is down.

Controlling Loads – Roof Joist Design

Joists must be designed to resist downward load and potentially upward acting load. LC 3 controls over LCs 1, 2, 4, 5 and 7 for downward acting load. LC 6 involves upward acting wind load. Net loads are down on the joists for this LC. Therefore, LC 3 controls for joist design.

Questions

Thank you!