PDHonline Course S187 (10 PDH)

2006 International Building Code - Structural Design

Instructor: John C. Huang, Ph.D, PE

2020

PDH Online | PDH Center

5272 Meadow Estates Drive
Fairfax, VA 22030-6658
Phone: 703-988-0088
www.PDHonline.com

Design Load Calculations Per IBC 2006

for

Metro Centre

(A Two-Story Office Building)

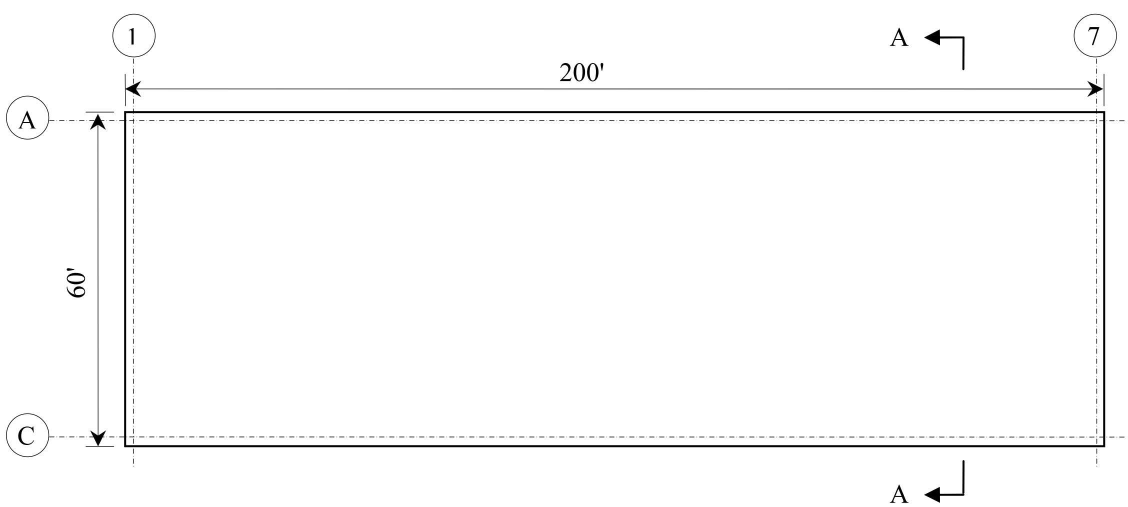
in

La Plata, MD 20464

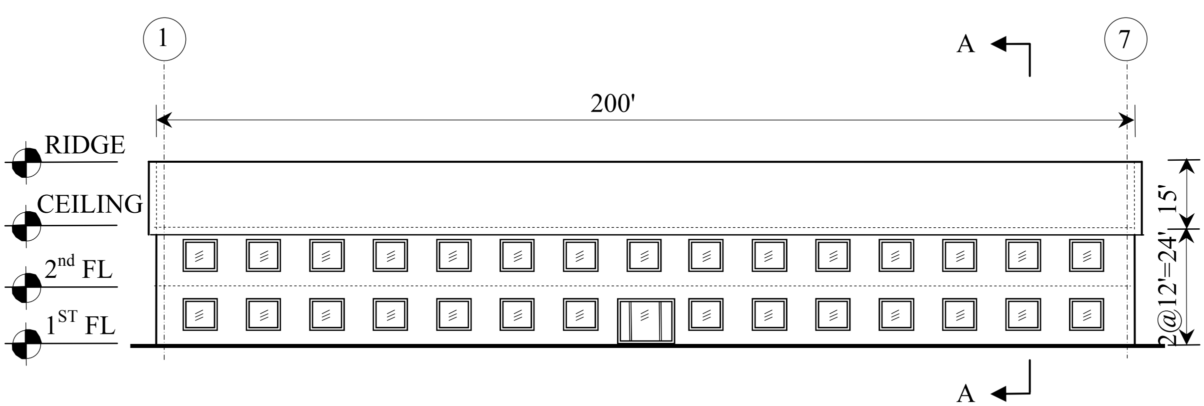
***Note**: Unlike previous versions of IBC, IBC 2006 relies on ASCE 7-05 almost entirely for the calculation of wind loads, snow loads and seismic loads. To understand the calculation procedures, you must obtain a copy of ASCE 7-05 (available from www.ASCE.org or www.Amazon.com). An instant download of an electronic version is also available from www.techstreet.com/ascegate.tmpl.*

Building Location & Geometry

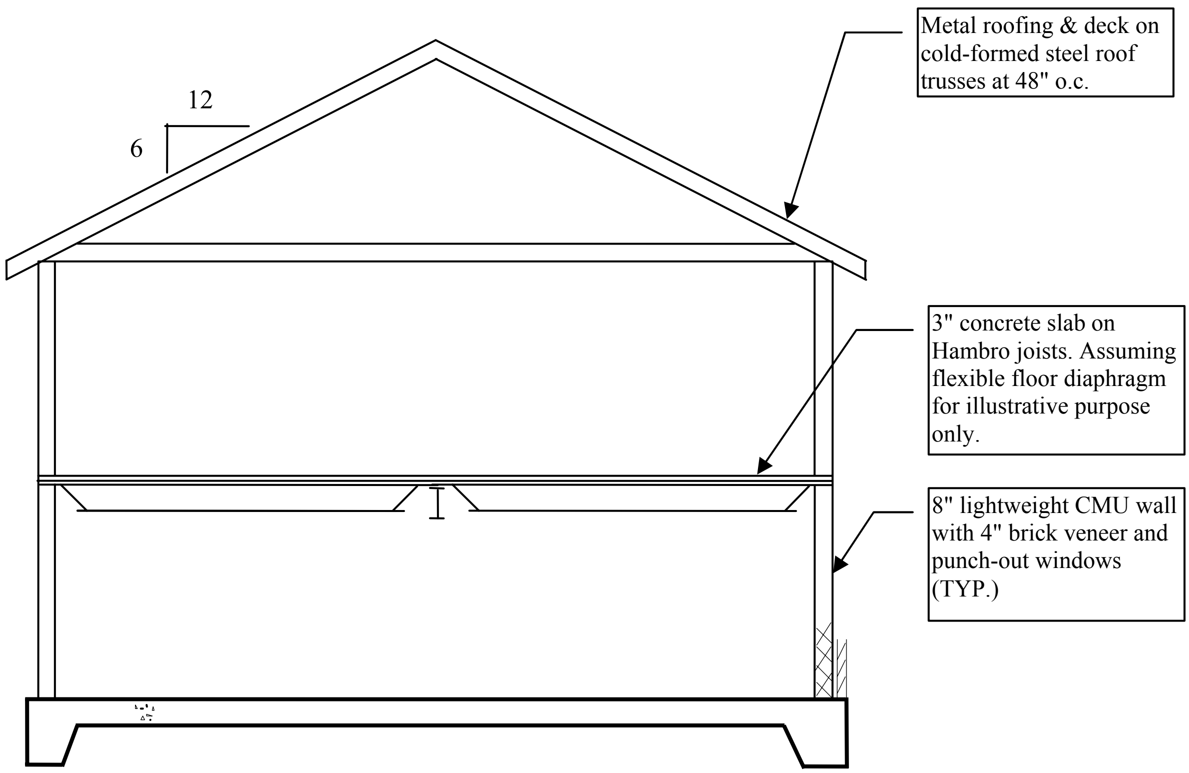
This building is located in La Plata, Maryland, approximately 30 miles southeast of Washington, DC (see map on Page 1). The building is approximately 200' long x 60' wide. The floor to floor height is 12'. The gable roof has a slope of 6 in 12.



BUILDING PLAN VIEW
(NOT TO SCALE)



BUILDING NORTH ELEVATION
NOT TO SCALE



BUILDING SECTION A-A
NOT TO SCALE

Project Description

Metro Centre is a two-story office building located in an open terrain with scattered obstructions. Building components utilized for this project are as follows:

- 3 inch thick concrete slab supported on Hambro joists spaced at 4'-1¼" apart for the second floor
- Hambro joists supported by steel girders or masonry load-bearing walls
- Steel columns supporting steel girders
- Spread footings under steel columns
- Architectural metal roof on metal deck and cold-formed steel roof trusses spaced at 48" o.c.
- Masonry shear walls for the lateral stability of the building
- Concrete grade beams on all four sides of the building

Lateral Load Calculations

Wind Pressure Calculations:

The lateral wind load is calculated per ASCE 7-05 Figure 6-2.

3-sec gust wind speed = 90 mph

Exposure Category = "C"

Wind Importance Factor = 1.0

Least Horizontal Dimension of the building = 60'-0"

Mean roof height = 31'-6"

Width of the edge strip = Least of 10% of Least Horizontal Dimension (LHD) or 40% of eave height but not less than 4% of LHD or 3.0 ft (per Note 10a of Figure 6-2)
= 0.1 x 60' or 0.4 x 24'
= 6.0 ft or 9.6 ft but not less than 2.4ft or 3.0 ft

Width of the edge strip = 6.0 ft.
Width of the end zone = 2 x 6'
= 12.0 ft per Page 37 Figure 6-2

Roof Slope = 6:12
Roof Angle θ = 26.6^o

Height & Exposure Adjustment Factor: λ = 1.42 per Page 40, Figure 6-2 (mean roof ht. 31'-6")

The structure is classified as an **Enclosed Building.**

Horizontal pressure calculation for transverse wind direction (perpendicular to ridge)
(See ASCE 7-05 Page 37 Figure 6-2)

Pressure @ End Zone

Roof Pressure (HBE) = 4.9 x 1.42 = 7.0 psf. (4.9 is obtained by interpolating with θ=26.6^o)
Wall Pressure (HAE) = 15.6 x 1.42 = 22.2 psf

Pressure @ Interior Zone

Roof Pressure (HD) = 4.4 x 1.42 = 6.2 psf.
Wall Pressure (HC) = 11.7 x 1.42 = 16.6 psf

Horizontal pressure calculation for longitudinal wind direction (parallel to ridge)
(See ASCE 7-05 Page 37 Figure 6-2)

Pressure @ End Zone

Roof Pressure (HEE) = 0 psf. (use zero if horizontal roof pressure is less than zero per Note 7 of Figure 6-2)

Wall Pressure (HAE) = 12.8 x 1.42 = 18.2 psf (using $\theta = 0^{o}$ for longitudinal direction per Note 3 of Figure 6-2)

Pressure @ Interior Zone

Roof Pressure (HG) = 0 psf.

Wall Pressure (HC) = 8.5 x 1.42 = 12.1 psf

Lateral Wind Loads Calculation (see Page 37, Figures 6-2)

A. At Roof Level

Wind force in N-S dir. (from front to rear)

Total Lateral Force = (End zone roof pressure) x (End zone roof projected area) +
(End zone wall pressure) x (End zone wall area) +
(Interior zone roof pressure) x (Int. zone roof projected area) +
(Interior zone wall pressure) x (Int. zone wall area)

Total Lateral Force = 7.0 psf x 12'x 2x 15' + 22.2 psf x 12'x 2 x 12'/2
+ 6.2 psf x (200'-2x12') x 15' + 16.6 psf x (200'-2x12')x 12'/2

= 39614 lbs

Wind force in E-W dir. (from left to right)

Total Lateral Force = (End zone roof pressure) x (End zone roof projected area) +
(End zone wall pressure) x (End zone wall area) +
(Interior zone roof pressure) x (Int. zone roof projected area) +
(Interior zone wall pressure) x (Int. zone wall area)

Total Lateral Force = 0 + 18.2 psf x 12'x 6'/2x 2 + 18.2 psf x 12'x 12'/2 x2
+ 0 + 12.1 psf x(60'-24') x 9'/2 + 12.1 psf x (60'-24')x(6'+12'/2)
= 11119 lbs

B. At Second Floor

Wind force in N-S dir. (from front to rear)

Total Lateral Force = (End zone wall pressure) x (End zone tributary wall area) + (Interior zone wall pressure) x (Int. zone tributary wall area)

Total Lateral Force = 22.2 psf x 12'x 2 x 12' + 16.6 psf x (200'-2x12')x 12'

= 41453 lbs

Wind force in E-W dir. (from left to right)

Total Lateral Force = (End zone wall pressure) x (End zone tributary wall area) + (Interior zone wall pressure) x (Int. zone tributary wall area)

Total Lateral Force = 18.2 psf x 12'x 2 x 12' + 12.1 psf x (200'-2x12')x 12'

= 10469 lbs

Vertical roof pressure calculation for transverse wind direction (perpendicular to ridge)
(See ASCE 7-05 Page 37 Figure 6-2)

Pressure @ End Zone

Roof Pressure (VEE) = -3.1 x 1.42 = -4.4 psf (upward)

Roof Pressure (VFE) = -9.5 x 1.42 = -13.5 psf (upward)

Roof Overhang Pressure = -10.7 x 1.42 = -15.2 psf (upward)

Pressure @ Interior Zone

Roof Pressure (VG) = -2.0 x 1.42 = -2.84 psf (upward)

Roof Pressure (VH) = -7.7 x 1.42 = -10.9 psf (upward)

Roof Overhang Pressure = -9.6 x 1.42 = -13.6 psf (upward)

<u>Vertical roof pressure calculation for longitudinal wind direction (parallel to ridge)</u>
(See ASCE 7-05 Page 37 Figure 6-2)

Pressure @ End Zone

Roof Pressure (VEE) = -15.4 x 1.42 =-21.9 psf (upward) (using $\theta=0^o$)

Roof Pressure (VFE) = -8.8 x 1.42 = -12.5 psf (upward)

Roof Overhang Pressure = -21.6 x 1.42 = -30.7 psf (upward)

Pressure @ Interior Zone

Roof Pressure (VGE) = -10.7 x 1.42 = -15.2 psf (upward)

Roof Pressure (VHE) = -6.8 x 1.42 = -9.7 psf (upward)

Roof Overhang Pressure = -16.9 x 1.42 = -24.0 psf (upward)

<u>*Comments:*</u>

1. *See diagrams on the next page for wind pressure direction.*

2. *Winds parallel to the ridge cause maximum suction on the entire roof. Net uplifting force (wind suction – roof dead load) should be used for the roof tie-down design. Please note that the total net uplifting force on the entire roof could be zero for some buildings, but each individual roof member and its connections should be designed for the maximum local pressure based on the component wind pressure calculation (see Figure 6-3). The calculations above are for the main windforce resisting system only.*

3. *The wind pressures above are confirmed using the Excel spreadsheet program IBC2006 developed by Yo Ratanapeanchai, SE. The computed results using Method 1(Simplified Procedure) and Method 2 (Analytical Procedure) are presented in the following pages.*

ASCE 7-05 (IBC 2006) WIND: BUILDING DATA:

Basic wind speed (3 sec gust) = 90 MPH

Exposure C

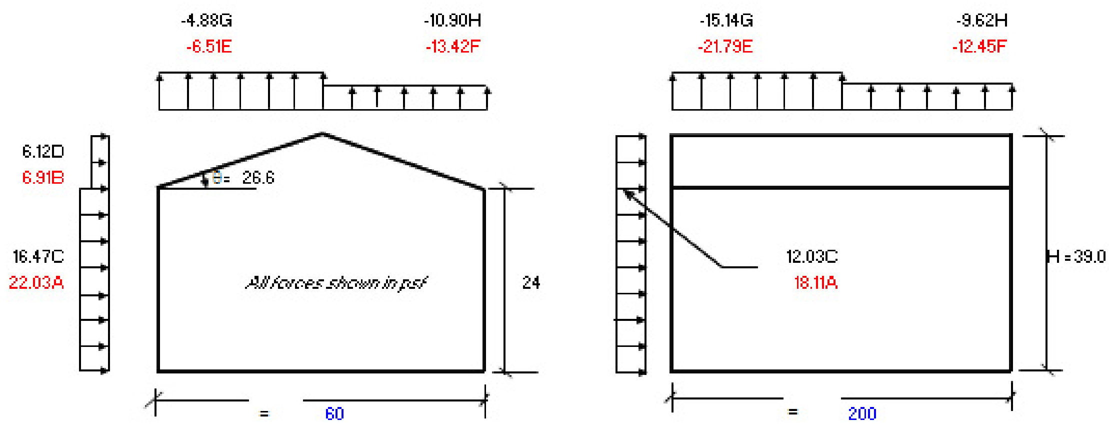
Roof Pitch = 6.00 :12

Mean Roof Height h = 31.5 ft

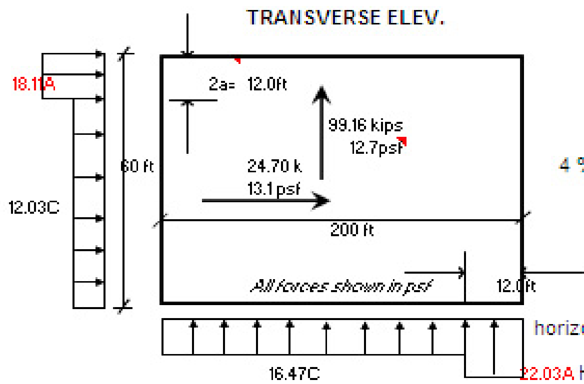
Importance factor I_w = 1.00 T-6-1

6.4 MOTHOD 1- SIMPLIFIED PROCEDURE (LOW-RISE, 60 FT) Print

Height Adjustment factor λ = 1.42 Fig 6-2



TRANSVERSE ELEV. LONGITUDINAL ELEV.



PLAN VIEW

10 % of least dimension= 6.0 ft

40 % of the eave height = 9.6 ft

4 % of least dimension or 3 ft= 3.0 ft

therefore a = 6.0 ft

Example: $p_s = \lambda K_{zt} I p_{s30}$ (6-1)

K_{zt} = 1.00 6.5.7

horizontal load at end zone p_{s30} = 12.8 Fig 6-2

x

Height Adjustment factor λ = 1.42 Fig 6-2

x

Importance factor Iw = 1.00 6.2

18.11 psf

FIGURE 6.2, Main Wind Force System

MWFRS

Load Direction	Roof Angle	Horizontal Loads				Vertical Loads					
		End Zone		Interior zone		End Zone		Interior zone		Overhang	
		Wall (A)	Roof (B)	Wall (C)	Roof (D)	WW (E)	LW (F)	WW (G)	LW (H)	E_{OH}	G_{OH}
Transverse	26.6	22.03	6.91	16.47	6.12	-6.51	-13.42	-4.88	-10.90	-15.19	-13.65
Longitudinal	All	18.112	-9.481	12.028	-5.66	-21.791	-12.45	-15.14	-9.622	-30.56	-23.9135

* If roof pressure under horizontal loads is less than zero, use zero

Plus and minus signs signify pressures acting toward and away from projected surfaces, respectively.

For the design of the longitudinal MWFRS use ϴ = 0', and locate the zone E/F, G/H boundary at the mid-length of the building

FIGURE 6-3, COMPONENT AND CLADDING

Roof effective area = 300 sq. ft, θ= 26.6

Interior Zone 1 = 8.54 -17.12 psf

End Zone 2 = 8.54 -23.98 psf

Conner Zone 3 = 8.54 -37.68 psf

Roof Overhang effective area = 2 sq. ft

Interior Zone 2 = -38.41 psf

End Zone 3 = -64.02 psf

Effective Area for wall element = 45 Sq. ft

Wall, Interior Zone 4 = 18.61 -20.42 psf

End Zone 5 = 18.61 -23.75 psf

IBC 1605.2.1(LRFD) U = 0.9D + 1.6W

IBC 1605.3.1(ASD), U = 06D + W, increase in allowable shall not be used.

4\. *IBC 1605.3.2(ASD), U = D + 1.3 W, allowable stress are permit to be increased.*

6.5 METHOD 2- ANALYTICAL PROCEDURE

Print

6.5.12. Design Wind Loads on Enclosed and Partially Enclosed Buildings. (all Heights)

MWFRS Velocity pressure q_z = .00256 K_z K_{zt} K_d V^2 I_w (6-15)

Exposure C Roof Height h = 31.5 feet

Exposure coefficient K_z = Section 6.5.6.6, is obtained from Table 6-3, Case 2 for MWFRS

Topography factor K_{zt} = 1.00 6.5.7.2, Figure 6-2

Directionality factor K_d = 0.85 Table 6-4

Wind Speed V = 90 mph

Importance factor I_w = 1.00 Table 6-1

q_z = 17.63 K_z psf

Internal Pressure Coefficient (GC_{pi}) = ± 0.18 Figure 6-5 for Enclosed Buildin

Gust effect factor G = 0.85 6.5.8.1

essures for MWFRS p = qGC_p - q_i (GC_{pi}) (6-17)

Wall and Roof External pressure Coefficients C_p from Fig. 6-6 or 6-8

Wind Normal to Ridge (⊥ to 200) L/B = 0.30 h/L = 31.5/60 = 0.53 θ= 26.6

Windward wall C_p = 0.8 Windward roof C_p = -0.28

Leeward wall C_p = -0.500 for L/B = 0.30 .eeward roof C_p = -0.60

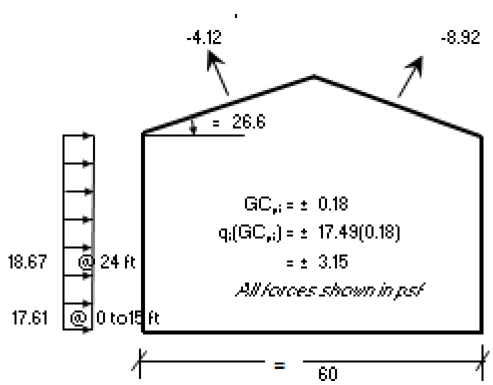
Side wall C_p = -0.7 or Roof C_p =

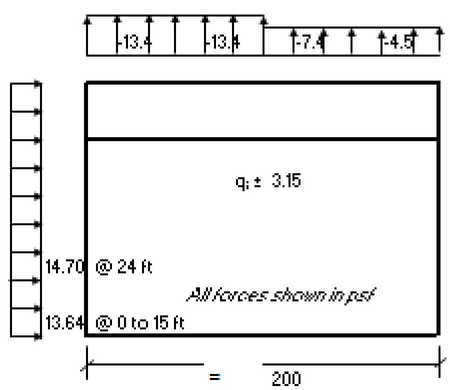
Wind Parallel to Ridge (⊥ to 60) L/B = 3.33

Windward wall C_p = 0.8 h/L = 31.5/200 = 0.16

Leeward wall C_p = -0.233 for L/B = 3.33 Roof C_p = -0.9 -0.9 -0.50 -0.30

Side wall C_p = -0.7 for dist 15.8 31.5 63.00





p = qGC_p - q_i (GC_{pi}) Eq. 6-15 For Exp C

where q = q_z for windward z_g = 900 ∝ = 9.5

q = q_h for leeward wall, side wall and r @31.5 ft K_z = 2.01$(z/z_g)^{2/\alpha}$

q_i = q_h for enclosed building @31.5 ft K_z (min) = 2.01$(15/z_g)^{2/\alpha}$

Roof Ht, h = 31.5 ft				Normal to Ridge ⊥ to 200		Parallel to ridge ⊥ to 60		
	Height	K_h	q_h	C_p	q_hGC_p	C_p	q_hGC_p	
Leeward wall	all	0.992	17.49	-0.5	-7.43	-0.23	-3.47	
Side wall	all	0.992	17.49	-0.7	-10.41	-0.70	-10.41	
Roof	ww	-0.277				-0.90	-13.38	fr 0 - 15.8
	Lw	-0.600				-0.90	-13.38	fr > 15.8
						-0.50	-7.43	fr 31.5-63
						-0.30	-4.46	fr 63

				Wind Normal to Ridge			Wind Parallel to Ridge		
	z, Ht. (ft)	K_z	q_z	C_p	p = q_zGC_p	WW+LW	C_p	p = q_zGC_p	WW+LW
Windward wall	0 to 15	0.849	14.96	0.8	10.17	17.61	0.80	10.17	13.64
	15.0	0.849	14.96	0.8	10.17	17.61	0.80	10.17	13.64
	24.0	0.937	16.52	0.8	11.23	18.67	0.80	11.23	14.70
	31.5	0.992	17.49	0.8	11.89	19.33	0.80	11.89	15.36

Seismic Load Calculations Using "Simplified Analysis Procedure" Per ASCE 7-05

This building belongs to Occupancy Category II. In absence of soil data, use **Site Class D** per Section 1613.5.2 of IBC 2006.

Per Section 12.14.1, Simplified Analysis Procedure in Section 12.14.8 can be used for this building (Note: the second floor is assumed to be a flexible diaphragm for illustrative purpose only.) See the definition of Flexible Diaphragm on Figure 12.3-1, Page 124 of ASCE 7-05.

The mapped maximum considered earthquake spectral response acceleration at short period, S_s and earthquake spectral response acceleration at 1.0 second period, S_1 per Figures 1613.5 (1) & (2) of IBC 2006 for the given site location are:

$\boxed{\mathbf{S_s = 0.151}\ \textit{(0.178 per IBC2003)}}$ $\boxed{\mathbf{S_1 = 0.050}\ \textit{(0.063 per IBC2003)}}$

Note: The above accurate reading of S_s is obtained from a computer program called the Ground Motion Parameter Calculator provided by USGS website using the zip code 20646.

Per Table 11.4-1 & 2, Site Coefficients F_a and F_v as a function of site class and mapped spectral response acceleration at short periods and 1-second period is, respectively,

$\boxed{\mathbf{F_a = 1.60}}$ $\boxed{\mathbf{F_v = 2.4}}$

Per Section 11.4.3	S_{MS} =	$F_a S_s$ =	1.6 x 0.151	= 0.242
	S_{M1} =	$F_v S_1$ =	2.4 x 0.050	= 0.120
Per Section 11.4.4	S_{DS} =	$(2/3)S_{MS}$ =	(2/3) x 0.242	= 0.161
	S_{D1} =	$(2/3)S_{M1}$ =	(2/3) x 0.120	= 0.080

This building belongs to Seismic Design Category A per Table 11.6-1 and Category B per Table 11.6-2 on Page 116**.**

Per Table 12.14-1, the basic seismic force resistance system of the building is chosen as "Intermediate reinforced masonry shear walls under Building Frame Systems (B18) - masonry shear walls need to satisfy the requirements of Section 14.4 of ASCE 7-05."

$\boxed{\mathbf{R = 4.0}}$

Per Section 12.14.8.1, the seismic base shear $V = FS_{DS} W/R$ F=1.1 for two-story buildings

$\boxed{\mathbf{V=0.044W}\ \textit{(0.057W per IBC 2003, 23\% reduction)}}$

Per Section 12.14.8.2, the lateral seismic forces F_x at each level = $w_x V/W$

$\boxed{\mathbf{F_x=0.044w_x}}$

Comment:

The seismic base shear is 4.4% of the building weight based on the simplified analysis procedure using IBC 2006, which represents 23% reduction compared to the value calculated using IBC 2003.

Seismic Load Calculation Using "Equivalent Lateral Force Procedure" Per ASCE 7-05

Design information: Occupancy Category II and Site Class D (see previous page).

Step1: Calculation of seismic response coefficient C_s

$C_S = S_{DS} / (R/I_E)$ --------------------- (Eq. 12.8-2 of ASCE 7-05)

$S_{DS} = (2/3).S_{MS} = (2/3).F_a .S_S$ ------- (Eqs. 11.4-1 & 3 of ASCE 7-05)

Therefore, S_{DS} = (2/3) x 1.6 x 0.151 = 0.161

R = 4----(Per Table 12.2-1) [Building is considered to have intermediate reinforced masonry shear walls satisfying the requirements of Section 14.4 of ASCE 7-05.]

$I_E = 1$

Therefore, C_S = 0.161/(4/1)

$\boxed{\mathbf{C_S = 0.040}}$

Check if $0.01 < C_S < S_{D1}/[(R/I_E) \times T]$ {see Eq. 12.8-5 for Lower Limit of C_S}

Where $T=Ta=C_T h_n^{3/4}$ = 0.020 x $31.5^{3/4}$ = 0.266 (Per Eq. 12.8-7 of ASCE 7-05)

$S_{D1} = (2/3) \times S_{M1} = (2/3) \times F_V \times S_1 = (2/3) \times 2.4 \times 0.050$
-----------------(Per Eqs. 11.4-2 & 11.4-4)

Therefore,

S_{D1}= 0.08

0.08/(4 x 0.266/1) = 0.075 {Upper Limit of C_S, per Eq. 12.8-3 of ASCE 7-05 when $T<=T_L$=8 per Figure 22-15 on Page 229}

0.01 < 0.0403 < 0.075 **O.K.**

Use

$\boxed{\mathbf{C_S = 0.040} \textit{ (0.0475 per IBC 2003)}}$

Therefore, the seismic base shear **$V = C_S W$ = 0.040W** (per Eq. 12.8-1 of ASCE 7-05)

Comment:

The above value is 9% lower than the value obtained using the simplified analysis procedure, and 15% lower than the value obtained using IBC 2003.

<u>Step2:</u> Calculation of building weight

Design data: Roof Dead Load = 20 psf (see calculation on Page 19)
Floor Dead Load = 70 psf (including partition loads, see Page 20)
Ext. Masonry Wall Avg. Wt. = 65 psf (8" LWT. CMU + Brick & wall openings)

Dead Load @ roof level w_x = Roof load + Wall load
= 20 psf x 60' x 200' + 65 psf x12'/2 x (60' x 2 + 200' x 2)
= 240000 + 202800
= **442800 lbs**

Dead Load @ 2nd Floor w_x = Floor load + Wall load
= 70 psf x 60' x 200' + 65 psf x12'x (60' x 2 + 200' x 2)
= 840000 + 405600
= **1245600 lbs**

Total dead load of the structure, W = 1688400 lbs

<u>Step3:</u> Calculation of base shear (per Eq. 12.8-1 of ASCE 7-05)

$V = C_S W$
$= 0.040 \times 1688400$
$= 67536$ lbs

<u>Step4</u>: Calculation of lateral force at each story (Per Eqs. 12.8-11 & 12.8-12 of ASCE 7-05)

$F_x = C_{VX} V$

Where, $C_{VX} = W_x h^k / (\sum W \times h^k)$ Since T=0.266 < 0.5 => k=1

At Roof Level h = 24 ft. W_x x h = 442800 x 24' = 10627200 lb-ft.

At 2nd Floor Level h = 12ft. W_x x h = 1245600 x 12' = 14947200 lb-ft

$\sum$ W x h = 25574400 lb-ft

Lateral force @ roof level due to seismic load

Cvx = 10627200/25574400 = 0.42

F_x = 0.42 x 67536 = **28365 lbs**

Lateral force @ 2nd level due to seismic load

Cvx = 14947200/25574400 = 0.58

F_x = 0.58 x 67536 = **39171 lbs**

<u>Seismic Loads for Bearing Wall Components:</u>

Calculation of lateral force perpendicular to wall (Per Section 12.14.7.6 of ASCE 7-05)

Seismic load perpendicular to the wall is taken as the greater of the following two values

0.1 x weight of wall (8" LWT CMU plus 4" brick veneer) = 0.1 x 93 = 9.3 psf

or

$0.40 \times I_E \times S_{DS} \times w_w = 0.40 \times 1.0 \times 0.161 \times 93 = 6.0$ psf

Seismic load perpendicular to the wall = 9.3 psf < 12.1 psf (the lowest wind pressure on walls)

Therefore, wind pressure will govern the out-of-plane design of all the exterior masonry shear walls.

***

<u>Comments</u>:

The Excel spreadsheet program IBC2006 is used again to verify the above calculations (see the computed results in the next two pages). In addition, the design response spectrum is constructed in the spreadsheet.

IBC2006 (1613), ASCE 7-05 CHAPTER 11, 12, 13 SEISMIC DESIGN CRITERIA

Soil Site Class	D	Table 20-3-1, Default = D		
Response Spectral Acc. (0.2 sec) S_s =	15.10%g	= 0.151g	Figure 22-1 through 22-14	
Response Spectral Acc.(1.0 sec) S_1 =	5.00%g	= 0.050g	Figure 22-1 through 22-14	
Site Coefficient F_a =	1.600		Table 11.4-1	
Site Coefficient F_v =	2.400		Table 11.4-2	
Max Considered Earthquake Acc. S_{MS} =	$F_a.S_s$	= 0.242	(11.4-1)	
Max Considered Earthquake Acc. S_{M1} =	$F_v.S_1$	= 0.120	(11.4-2)	
@ 5% Damped Design S_{DS} =	$2/3(S_{MS})$	= 0.161	(11.4-3)	
S_{D1} =	$2/3(S_{M1})$	= 0.080	(11.4-4)	
Building Occupancy Categories	II, Standard		Table 1-1	
Design Category Consideration:	Flexible Diaphragm		with dist. between seismic resisting system >40ft	
Seismic Design Category for 0.1sec	A		Table 11.6-1	
Seismic Design Category for 1.0sec	B		Table 11.6-2	
S1< .75g	NA		Section 11.6	
Since Ta < .8Ts (see below), SDC =	B	Control (exception of Section 11.6 does not apply)		

Comply with Seismic Design Category B IRC, Seismic Design Category = A T-R301.2.2.1.1

12.8 Equivalent lateral force procedur

B. BUILDING FRAME SYSTEMS T-12.2-1

Seismic Force Resisting Systems 18. Intermediate reinforced masonry shear walls

C_t =	0.02	x = 0.75	T-12.8-2	
Building ht. H_n =	31.5	ft	Limited Building Height (ft) = NL	
C_u =	1.700	for S_{D1} of 0.080g	Table 12.8-1	
Approx Fundamental period, T_a =	$C_t(h_n)^x$	= 0.266	12.8-7	T_L = 8 Sec
Calculated T shall not exceed ≤	Cu.Ta	= 0.452		Use T = 0.266 sec.
0.8Ts =	$0.8(S_{D1}/S_{DS})$	= 0.397	Control (exception of Section 11.6 does not apply)	
Is structure Regular & ≤ 5 stories ?	Yes		12.8.1.3	
Response Spectral Acc.(0.2 sec) S_s =	0.151g		*Max Ss ≤ 0.15g*	
F_a =	1.60			
@ 5% Damped Design S_{DS} =	$⅔(F_a.S_s)$	= 0.161g	(11.4-3)	
Response Modification Coef. R =	4		Table-12.2-1	
Over Strength Factor Ω_o =	2		foot note g	
Importance factor I =	1		Table 11.5-1	
Seismic Base Shear V =	$C_s W$			

$$C_s = \frac{S_{DS}}{R/I} = 0.040 \qquad (12.8\text{-}2)$$

or need not to exceed, $$C_s = \frac{S_{D1}}{(R/I).T} = 0.075 \qquad \text{For } T \le T_L \qquad (12.8\text{-}3)$$

or $$C_s = \frac{S_{D1}T_L}{T^2(R/I)} \quad \text{N/A} \qquad \text{For } T > T_L \qquad (12.8\text{-}4)$$

C_s shall not be less than =	0.01			(12.8-5)
Min C_s =	$0.5S_1/R$	N/A	For S_1 ≥ 0.6g	(12.8-6)
Use C_s =	0.040			
Design base shear V =	0.040 W	Control		

12.14 Simplified Seismic base shear 18. Intermediate reinforced masonry shear walls T-12.14-1

@ 5% Damped Design S_{DS} =	0.161	SDC = A	Limitations: P
F =	1.1	For two-story building	R = 4

$$V = \frac{FS_{DS}(W)}{R} = 0.044\ W$$

13.3 Seismic Demands on Nonstructural Componel

$F_p = \dfrac{0.4a_pS_{DS}W_p(1+2z/h)}{(R_p/I_p)}$ (13.3-1) $S_{DS} = 0.161$

$a_p = 1$ $R_p = 2.5$ T-13.5-1 or 13.6-1

$I_p = 1.0$ 13.1.3

$z = 10$ ft $h = 31.5$ ft $F_p = 0.042$ Wp

Max $F_p = 1.6S_{DS}I_pW_p$ = 0.258Wp (13.3-2)

Min $F_p = 0.3S_{DS}I_pW_p$ = 0.048Wp (13.3-3)

$F_p = 0.048$ Wp

12.11.1 Structural Walls and Their Anchorage

$F_p = 0.40S_{DS}IW_w$ 12.11.1

= 0.064(W)

12.11.2 Anchorage of Concrete or Masonry structural Walls (flexible diaphragm)

or $F_p = 400S_{DS}I$ = 64 shall be ≥ 280 #/ft

$F_p = 0.8S_{DS}I(W_w)$ (12.11.1)

= 0.129 Wp

Max Seismic Load $E_M = \Omega Q_E \pm 0.2S_{DS}D$ (12.4.4), (12.4.5), (12.4.6), (12.4.7)

Where $\Omega_o = 2$

$0.2S_{DS}D = 0.033(D)$

Deflection Amplification factor $C_d = 4$

Nonbuilding structures, Section 15

Response Modification Coef. R = 4 T-15.4-1 or T-15.4-2

Importance factor I = 1 15.4.1.1

For flexible nonbuilding, $C_s = S_{DS}I/R$ = 0.040

Min $C_s = 0.03$ (15.4-1)

or $C_s = 0.8\,S_1I/R$ = 0.010 (15.4-2)

V = 0.040 W

For rigid nonbuilding, $C_s = 0.3\,S_{DS}\,I$ (15.4-5)

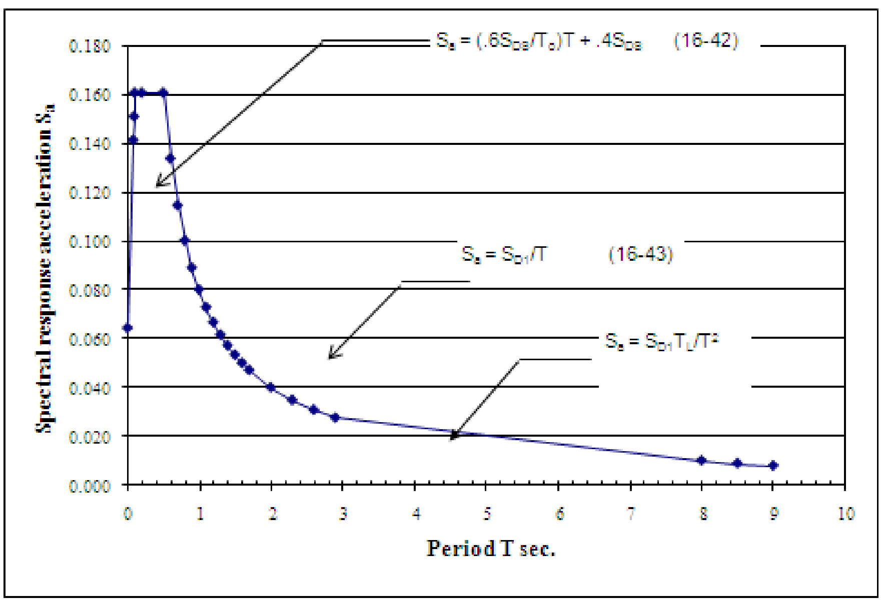
= 0.048 W

DESIGN RESPONSE SPECTRAL CURVE

$T_0 = 0.2S_{D1}/S_{DS}$ = 0.099 sec.

$T_s = S_{D1}/S_{DS}$ = 0.497 sec.

T_L = 8 Sec 11.4.5

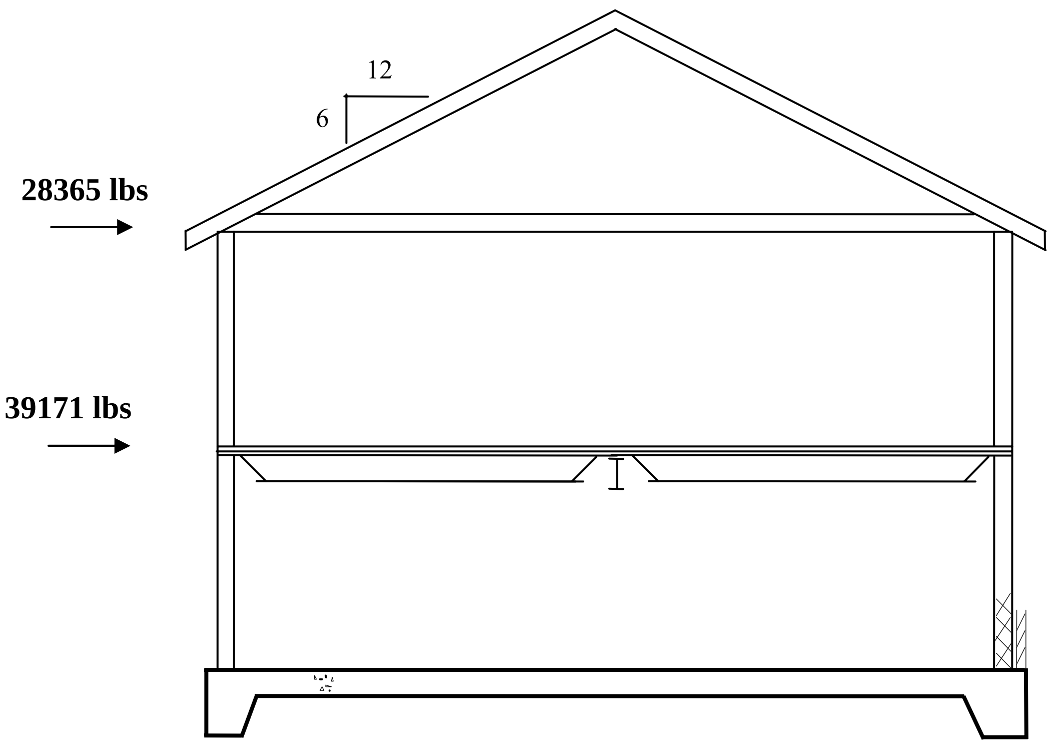


Design spectral curve

	Period, T	S_a
PGA		0.064
	0.079	0.142
	0.089	0.151
T_0	0.099	0.161
	0.200	0.161
T_s	0.497	0.161
	0.597	0.134
	0.697	0.115
	0.797	0.100
	0.897	0.089
	0.997	0.080
	1.097	0.073
	1.197	0.067
	1.297	0.062
	1.397	0.057
	1.497	0.053
	1.597	0.050
	1.697	0.047
	1.997	0.040
	2.297	0.035
	2.597	0.031
	2.897	0.028

Summary of Lateral Loads for Main Lateral Force Resisting System

Lateral Load Summary				
Level	Wind Loads (kips)		Seismic Loads (kips)	
	N-S Direction	E-W Direction	N-S Direction	E-W Direction
Roof Level	39.6	11.1	28.4	
2nd Floor	41.5	10.5	39.2	
Total Base Shear	81.1	21.6	67.6	



Seismic Load Distribution

Comments:

1. *The wind load will govern the lateral design in the N-S direction for this building. However, the seismic load will govern the lateral design in the E-W direction due to the building's narrow shape.*
2. *To further refine the seismic load calculations, it is necessary to obtain detailed soil properties on the site. If the site class changes from D to C, the seismic loads will drop 25% due to the reduction in Site Coefficient F_a.*
3. *One way to reduce seismic load is lower the weight of the structure. If the exterior 4" brick veneer is changed to 2" EIFS, the seismic load will drop approximately 14%. Further reduction in seismic load can be achieved if the exterior CMU walls are replaced by cold-formed steel stud walls with X-braces.*

Gravity Load Calculations

Gravity loads include dead loads, live loads and snow loads. Dead loads can be calculated based on the weight of the construction materials. Live loads on a floor can be determined from Table 1607.1. For this sample project, we can use the Excel spreadsheet IBC2006 to calculate the roof live load, floor live load reduction and roof snow loads (the computed results are presented below).

1607.11.2 MINIMUM ROOF LOAD

Tributary area A_t = 1200.0 SQ.FT
R_1 = 0.60 Eq. 16-30
= 0.60
Rise per feet, F = 5 :12
R_2 = 1.2-.05F Eq. 16-32
= 0.95
L_r = $20R_1R_2$ Eq. 16-24
= 20(0.6)(0.95)
Min Design Roof Load L_r = 12.00 psf

1607.9 REDUCTION OF LIVE LOAD

1607.9.1 General

$L = L_o[0.25+15/(K_{LL}A_T)]^{1/2}$ (6-24)
L_o = 50 psf T-1607.1
K_{LL} = 4 Interior columns T-1607.9.1
A_T = 750.0 SQ.FT
L = 26.2 psf = 0.52 L_o

L shall not be less than 0.50 L_o for members supporting one floor and L shall not be less than 0.40 L_o for members supporting two or more floors

1607.9.2 Alternate Live Load reduction for Vertical member

L_o = 50 psf
R = r(A-150) (16-25)
A = 750.0 SQ.FT
r = 0.08 for floor
R = 48 60% Max for vertical member

L = 26 psf

And R = $23.1(1+D/L_o)$ (16-26)

Dead load D = 70.0 psf
R = 55.4 60% Max for vertical member
L = 22.3 psf
Min Design Live Load L = 26.0 psf

ASCE 7.3 FLAT ROOF SNOW LOAD (slope ≤ 5°)

Flat-roof snow load, p_f = $0.7C_eC_tI_sp_g$ (7-1)
Ground snow load, p_g = 25 psf Figure 7-1
Terrain Category = C (see Section 6.5.6)
Exposure of Roof = Fully Exposed
Thermal Condition = All structures except as indicated below
Snow load importance factor, I_s = 1 Table 7-4
Snow exposure factor, C_e = 0.9 Table 7-2
Thermal factor, C_t = 1.0 Table 7-3
Flat-roof snow load, p_f = 15.75 psf
Min p_f = 20 psf 7.3
Design p_f = 20.00 psf

7.4 SLOPE ROOF SNOW LOAD (slope > 5°)

p_s = C_sp_f (7-2)
Design p_f = 20.00 psf
Slope Θ = 26.6 ° Unobstructed Slippery Surfaces
C_t = 1.0
Slope factor C_s = 0.67 7.4.1, 7.4.2, 7.4.3 Figure 7.2a,b and c
p_s = 13.35 psf

Balanced and Unbalanced Snow Load for Hip and Gable Roof

Snow Density γ = $.13p_g + 14 \le 30$ pcf (7-3)
= 17.25 pcf
S =1/tanΘ = 2.00
Height of Snow Drift h_d = 1 ft
$h_d\gamma/S^{1/2}$ = 12.21 psf
$^8/_3h_dS^{1/2}$ = 3.77 ft

See Figure 7.3 for Curved Roof
See Figure 7.4 for Cont Beam
See Figure 7-6 for Sawtooth Roof
See Figure 7-8 for Snow Drifts

BALANCED

UNBALANCED W > 20 N/A

UNBALANCED OTHER

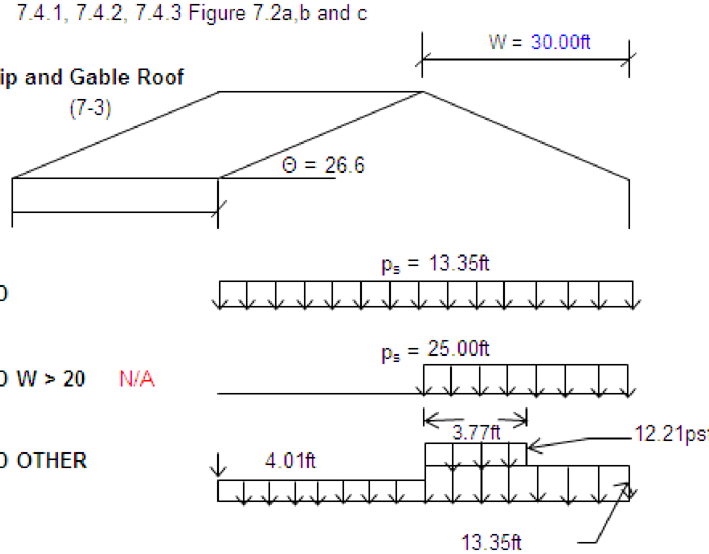


Figure 7-5

<u>*Comments*</u>*:*

1. *Architectural metal roofs are considered as unobstructed slippery surfaces.*
2. *Because p_g exceeds 20 lb/ft^2, the minimum snow load on flat roof $p_f = 20(I_S)$ per Section 7.3 of ASCE 7-05.*
3. *The unbalanced snow load shown above will produce a greater roof truss reaction than the balanced snow load, and may govern the design of the roof trusses.*

The dead and live loads on the roof and floor for this project are tabulated below along with the permitted live load reductions and the combined load values from different load combinations.

Roof Design Loads

Items	Description		Max. Load	Min. Load
			(psf)	(psf)
Roofing	Architectural metal roof		1.5	1.0
Deck	1.5" x22 ga. metal roof deck		1.7	1.2
Framing	Cold-formed steel roof trusses @ 48"		3.0	2.0
Insulation	8" R-30 Fiberglass insulation		7.2	3.6
Ceiling	1/2" gypsum board on furring channels		2.5	2.2
Sprinklers	Sprinklers		2.0	1.5
MEP	Mech. Elec. & Plumbing		2.0	0.5
		Calculated Dead Load	19.9	12.0
		Use this DL instead	**20.0**	**12.0**
		Live Load	18.0	0
		Snow Load	13.4	0
		Wind (zone 2 - 100sf)	10.0	-24.1
ASD Load Combinations:		Dead + Snow Load	38.0	-
		Dead + 0.75(Wind+Snow)Load	41.0	-
		0.6*Dead + Wind Load	-	-16.9
LRFD Load Combinations:		1.2D + 1.6S + 0.8W	60.8	-
		1.2D + 1.6W + 0.5S	49.0	-
		0.9D + 1.6W	-	-27.8
Roof Live Load Reduction		Roof angle	6:12	or 26.6^{o}

0 to 200 sf:	18 psf
200 to 600 sf:	21.6 - 0.018Area, but not less than 12 psf
over 600 sf:	12 psf

Comments:

1. *The roof uplifting force will be controlled by the load combination of 0.6D + W for ASD. The maximum roof truss uplifting force will occur in the roof end zones and shall be calculated using wind loads for components. All roof trusses shall be secured to CMU walls to resist the uplifting forces.*
2. *Although the calculated wind load values on roof are all negative (suction), the minimum design wind pressure is 10 psf acting in either direction normal to the surface per Section 6..1.4 of ASCE 7-05.*
3. *Minimum roof load shall be used in the calculation of shear wall anchorage design.*
4. *Furring channels are required for ceiling gypsum boards because roof trusses are spaced at 48" on center.*

Floor Design Loads

Items	Description	Max. Load (psf)	Min. Load (psf)
Flooring	Carpet & pad	1.0	1.0
Topping	3" regular wt. concrete slab	37.5	36.0
Deck/sub-floor	None	0.0	0.0
Framing	Hambro steel bar joists & steel I-girders	6.0	5.0
Ceiling	Suspended acoustical tile	1.5	1.0
Sprinklers	Sprinklers	2.0	1.5
MEP	Mech. Elec. & Plumbing	2.0	0.5
Partitions	Partitions	20.0	0.0
	Calculated Dead Load	70.0	45.0
	Use this DL instead	**70.0**	**45.0**
	Live Load	50.0	
	Total Load	130.0	45.0

Floor Live Load Reduction

$L = L_o \{0.25+15/(K_{LL}A_T)^{1/2}\}$

Unreduced design live load: L_o = 50 psf

Floor girders	K_{LL} =	2
Tributary Area	A_T =	750 sf
Reduced live load:	L =	32 psf (>0.5L_o O.K.)
Columns (one floor)	K_{LL} =	4
Tributary Area	A_T =	750 sf
Reduced live load:	L =	26 psf (>0.5L_o O.K.)

Comments:

1. *The weights of all partitions are to be considered as dead loads. Provisions for the weight of partitions must be made in the structural design, except where the specified minimum uniformly distributed live load exceeds 80 psf.*

2. *Because of the different levels of live load reduction, the total design load for floor girders is different from the load for columns.*

3. *Minimum floor load shall be used in the calculation of shear wall anchorage design.*

According to Section 1603 of the IBC 2006, the design loads and other information pertinent to the structural design shall be clearly indicated on the construction documents. A sample design loads summary is presented on the next page for this project.

DESIGN LOADS SUMMARY

Code:			International Building Code 2006	
Live Loads: Roof	0 to 200 sf:		18	psf
	200 to 600 sf:		21.6 - 0.018Area, but not less than 12 psf	
	over 600 sf:		12	psf
Floor		=	50	psf (live load reduction permitted)
Stairs		=	100	psf
Partitions		=	20	psf
Dead Loads: Floor		=	80	psf
Roof		=	20	psf
Roof Snow Loads:				
Design Roof Snow load		=	13.4	psf
Flat Roof Snow Load	P_f	=	20.0	psf
Snow Exposure Factor	C_e	=	0.9	
Importance Factor	I_S	=	1	
Thermal Factor	C_t	=	1	
Ground Snow Load	P_g	=	25	psf
Rain on Snow Surcharge		=	0	
Sloped-roof Factor	C_S	=	0.67	
Wind Design Data:				
Basic Wind speed		=	90	mph
Mean Roof Height	h	=	31.5	ft
Building Category		=	II	
Importance Factor	I_W	=	1	
Exposure Category		=	C	
Enclosure Classification		=	Enclosed Building	
Internal pressure Coef.	GC_{pi}	=	±0.18	
Directionality Factor	K_d	=	0.85	
Earthquake Design Data:				
Analysis Procedure		=	Equivalent Lateral-Force Analysis	
Seismic Use Group		=	I	
Importance Factor	I_E	=	1	
Mapped spectral response	S_s	=	15.1	%g
accelerations	S_1	=	5.0	%g
Site Class		=	D	
Spectral Response Coef.	S_{DS}	=	0.161	
	S_{D1}	=	0.08	
Seismic Design Category		=	B	
Basic Structural System		=	Building Frame Systems	
Seismic Resisting System		=	Intermediate reinforced masonry shear walls	
Response Modification Factor	R	=	4	
Seismic Response Coef.	C_s	=	0.040	
Design Base Shear	V	=	0.040	W