Roof Component

Distance "a":

smaller of

$0.1\ (40) = 4$ ft (controls)

or

$0.4\ (12.1) = 4.8$ ft

Roof trusses are 32 ft long and spaced 4 ft apart.

Effective area:

larger of

$32 \times 4 = 128\ \text{ft}^2$

or

$32 \times 32/3 = 341\ \text{ft}^2$ (controls)

From Figure 6-11C of the Standard, for $\theta = 15°$:

$(GC_p) = +0.3$ for Zones 1, 2, and 3 (Note: Zone 3 covers very small area of truss)

$(GC_p) = -0.8$ for Zone 1

$(GC_p) = -2.2$ for Zone 2

Design pressures:

$p = 12.3\ (0.3 + 0.18) = +5.9$ psf (all zones)

$p = 12.3\ (-0.8 - 0.18) = -12.1$ psf (Zone 3)

$p = 12.3 \times (-2.2) = -27.1$ psf (Zone 2)

Overhang pressures to be used for reaction and anchorage:

$p = 12.3\ (-2.2) = -27.1$ psf (edge of roof)

$p = 12.3\ (-2.5) = -30.8$ psf (roof corners)

Roof Panels

Effective area $= 4 \times 8 = 32\ \text{ft}^2$

From Figure 6-11C of the Standard, for $\theta = 15°$ (Note: Zones 2 and 3 are regarded as overhang):

$(GC_p) = +0.4$ for Zones 1, 2, and 3

$(GC_p) = -0.85$ for Zone 1

$(GC_p) = -2.2$ for Zone 2 (with overhang)

$(GC_p) = -2.2$ for Zone 3 on hip roofs (with overhang)

$(GC_p) = -3.1$ for Zone 3 on gable roofs (with overhang)

Design pressures:

$p = 12.3\ (0.4 + 0.18) = +7.1$ psf (all zones)

$p = 12.3 \times (-0.85 - 0.18) = -12.7$ psf (Zone 1)

$p = 12.3 \times (-2.2) = -27.1$ psf (Zone 2)

$p = 12.3 \times (-2.2) = -27.1$ psf (Zone 3 on hip roofs)

$p = 12.3 \times (-3.1) = -38.1$ psf (Zone 3 on gable roofs)

Fasteners

Effective area = 10 ft^2:

$(GC_p) = +0.5$ for Zones 1, 2, and 3

$(GC_p) = -0.9$ for Zone 1

$(GC_p) = -2.2$ for Zone 2 (with overhang)

$(GC_p) = -2.2$ for Zone 3 on hip roofs (with overhang)

$(GC_p) = -3.7$ for Zone 3 on gable roofs (with overhang)

Design pressures:

$p = 12.3\ (0.5 + 0.18) = +8.4$ psf (all zones)

$p = 12.3 \times (-0.9 - 0.18) = -13.3$ psf (Zone 1)

$p = 12.3 \times (-2.2) = -27.1$ psf (Zone 2)

$p = 12.3 \times (-2.2) = -27.1$ psf (Zone 3 on hip roofs)

$p = 12.3 \times (-3.7) = -45.5$ psf (Zone 3 on gable roofs)

3.6 Example 6 200-ft × 250-ft Gable Roof Commercial/Warehouse Building Using Buildings of All Height Provisions

In this example, design wind pressures for a large, one-story commercial/warehouse building are determined. Figure 3-13 shows the dimensions and framing of the building. The building data are as follows:

Location:	Memphis, Tennessee
Terrain:	Flat farmland
Dimensions:	200 ft × 250 ft in plan Eave height of 20 ft Roof slope 4:12 (18.4°)
Framing:	Rigid frames span the 200-ft direction Rigid frame bay spacing is 25 ft Lateral bracing in the 250-ft direction is provided by a "wind truss" spanning the 200 ft to side walls and cable/rod bracing in the planes of the walls Girts and purlins span between rigid frames (25-ft span) Girt spacing is 6 ft 8 in. Purlin spacing is 5 ft
Cladding:	Roof panel dimensions are 2 ft wide Roof fastener spacing on purlins is 1 ft on center Wall panel dimensions are 2 ft × 20 ft Wall fastener spacing on girts is 1 ft on center Openings are uniformly distributed

Exposure and Building Classification

The building is located on flat and open farmland. It does not fit Exposures B or D; therefore, Exposure C is used (Sections 6.5.6.2 and 6.5.6.3 of the Standard).

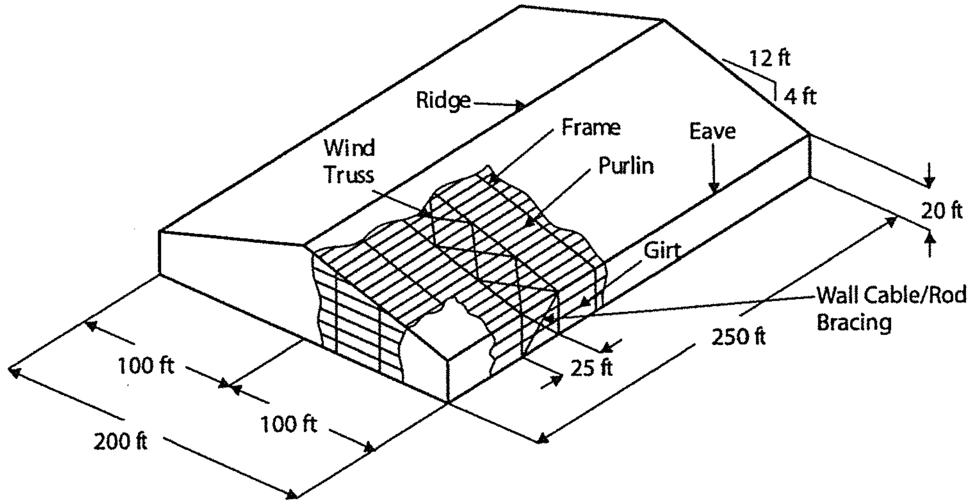


Figure 3-13 Dimensions and Framing of the Building of Examples 6 and 7

The building function is commercial-industrial. It is not considered an essential facility or likely to be occupied by 300 persons at one time. Category II is appropriate. Table 1-1 and Table 6-1 of the Standard specify an importance factor $I = 1.0$.

Basic Wind Speed

Selection of basic wind speed is addressed in Section 6.5.4 of the Standard. Memphis, Tennessee, is not located in the special wind region, nor is there any reason to suggest that winds at the site are unusual and require additional attention. Therefore, the basic wind speed is $V = 90$ mph (see Figure 6-1 of the Standard).

Analytical Procedure

Method 2, Analytical Procedure, is used in this example (see Section 6.5 of the Standard). In addition, provisions of buildings of all heights, given in Section 6.5.12.2.1 and Figure 6-6 for MWFRS, will be used. Alternate provisions of low-rise buildings are illustrated in Ex. 7 (Section 3.7 of this guide).

Wind Directionality

Wind directionality factor is given in Table 6-4 of the Standard. For MWFRS and C&C, the factor $K_d = 0.85$.

Velocity Pressures (Table 3-17)

The velocity pressures are computed using the following equation:

$$q_z = 0.00256\ K_z K_{zt} K_d V^2 I \text{ psf} \qquad \text{(Eq. 6-15)}$$

where

K_z = Value obtained from Table 6-3 of the Standard
K_{zt} = 1.0 (no topographic effect)
I = 1.0 for Category II building
K_d = 0.85
V = 90 mph

Substituting these values into Eq. 6-15 yields:

$$q_z = 0.00256 K_z (1.0)(0.85)(90)^2(1.0)$$
$$= 17.6\ K_z \text{ psf}$$

Table 3-17 Velocity Pressures (psf)

Height	*ft*	K_z	q_z *(psf)*
	0–15	0.85	15.0
Eave	20	0.90	15.8
	30	0.98	17.3
h	36.7	1.02	18.0*
	40	1.04	18.3
	50	1.09	19.2
Ridge	53.3	1.10	19.4

*q_h = 18.0 psf.

Values for K_z are the same for Cases 1 and 2 for Exposure C (see Table 6-3 of the Standard). Mean roof height h = 36.7 ft.

Design Wind Pressure

Design wind pressures for MWFRS of this building can be obtained using Section 6.5.12.2.1 of the Standard for buildings of all heights or Section 6.5.12.2.2 for low-rise buildings. Pressures determined in this example are using buildings of all heights criteria. Ex. 7 illustrates use of low-rise building criteria.

$$p = qGC_p - q_i(GC_{pi}) \qquad \text{(Eq. 6-17)}$$

where

q = q_z for windward wall at height z above ground
q = q_h for leeward wall, side walls, and roof
q_i = q_h for enclosed buildings
G = Gust effect factor
C_p = Values obtained from Figure 6-6 of the Standard
(GC_{pi}) = Values obtained from Figure 6-5

For this example, when the wind is normal to the ridge, the windward roof experiences both positive and negative external pressures. Combining these external pressures with positive and negative internal pressures will result in four loading cases when wind is normal to the ridge.

When wind is parallel to the ridge, positive and negative internal pressures result in two loading cases. The external pressure coefficients, C_p for $\theta = 0°$, apply in this case.

Gust Effect Factor

For rigid structures, G can be calculated using Eq. 6-4 (see Section 6.5.8.1 of the Standard) or alternatively taken as 0.85. For simplicity, G = 0.85 is used in this example.

External Wall C_p from Figure 6-6

The pressure coefficients for the windward wall and for the side walls are 0.8 and –0.7, respectively, for all L/B ratios.

The leeward wall pressure coefficient is a function of the L/B ratio. For wind normal to the ridge, L/B = 200/250 = 0.8; therefore, the leeward wall

Table 3-18 External Wall C_p

Surface	*Wind direction*	*L/B*	C_p
Windward wall	All	All	0.80
Leeward wall	Normal to ridge	0.8	–0.50
	Parallel to ridge	1.25	–0.45*
Side wall	All	All	–0.70

*By linear interpolation.

Table 3-19 Roof C_p (Wind Normal to Ridge)

Surface	*15°*	*18.4°*	*20°*
Windward roof	–0.5	–0.36*	–0.3
	0.0	0.14*	0.2
Leeward roof	–0.5	–0.57*	–0.6

*By linear interpolation.

pressure coefficient is –0.5. For flow parallel to the ridge, $L/B = 250/200 = 1.25$; the value of C_p is obtained by linear interpolation. The wall pressure coefficients are summarized in Table 3-18.

External Roof C_p from Figure 6-6 (Wind Normal to Ridge)

The roof pressure coefficients for the MWFRS (Table 3-19) are obtained from Figure 6-6 of the Standard. For the roof angle of 18.4°, linear interpolation is used to establish C_p. For wind normal to the ridge, $h/L = 36.7/200 = 0.18$; hence, only single linear interpolation is required. Note that interpolation is only carried out between values of the same sign.

Internal GC_{pi}

Values for GC_{pi} for buildings are addressed in Section 6.5.11.1 and Figure 6-5 of the Standard.

The openings are evenly distributed in the walls (enclosed building) and Memphis, Tennessee, is not in a hurricane-prone region. The reduction factor of Section 6.5.11.1.1 is not applicable for enclosed buildings; therefore,

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

MWFRS Net Pressures

$$p = qGC_p - q_i(GC_{pi}) \qquad \text{(Eq. 6-17)}$$

$$p = q(0.85)\,C_p - 18.0(\pm 0.18)$$

where

q = q_z for windward wall
q = q_h for leeward wall, side wall, and roof
q_i = q_h for windward walls, side walls, leeward walls, and roofs of enclosed buildings

Typical Calculation

Windward wall, 0–15 ft, wind normal to ridge:

$$p = 15.0(0.85)(0.8) - 18.0(\pm 0.18)$$

$$p = 7.0 \text{ psf with } (+) \text{ internal pressure}$$

$$p = 13.4 \text{ psf with } (-) \text{ internal pressure}$$

The net pressures for the MWFRS are summarized in Table 3-20.

Table 3-20 MWFRS Pressures: Wind Normal to Ridge

Surface	z *(ft)*	q *(psf)*	G	C_p	*Net pressure psf with* $(+GC_{pi})$	*Net pressure psf with* $(-GC_{pi})$
Windward wall	0–15	15.0	0.85	0.8	7.0	13.4
	20	15.8	0.85	0.8	7.5	14.0
Leeward wall	All	18.0	0.85	–0.5	–10.9	–4.4
Side walls	All	18.0	0.85	–0.7	–14.0	–7.5
Windward	–	18.0	0.85	–0.36	–8.8	–2.3
Roof*				0.14	–1.1	5.4
Leeward roof	–	18.0	0.85	–0.57	–12.0	–5.5

$q_h = 18.0$ psf; $(GC_{pi}) = \pm 0.18$; $q_h(GC_{pi}) = \pm 3.2$ psf.

*Two loadings on windward roof and two internal pressures yield a total of four loading cases (see Figures 3-14 and 3-15).

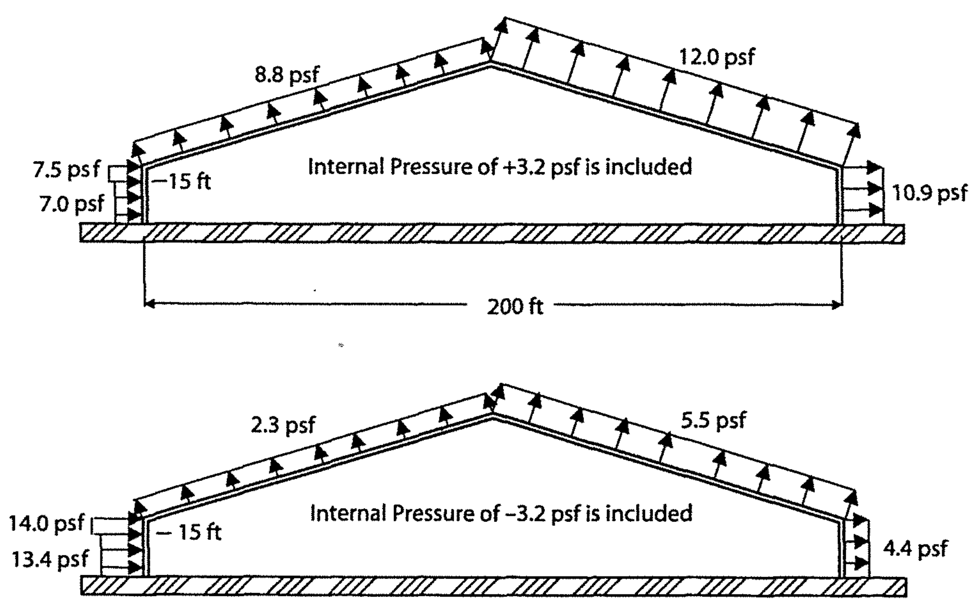
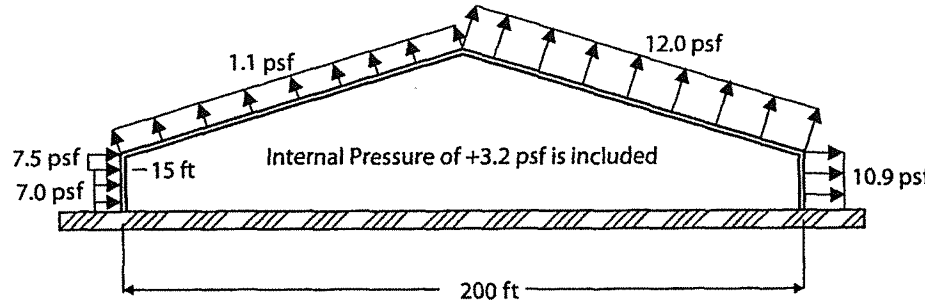


Figure 3-14 Net Design Wind Pressures for MWFRS when Wind is Normal to Ridge with Negative Windward External Roof Pressure Coefficient



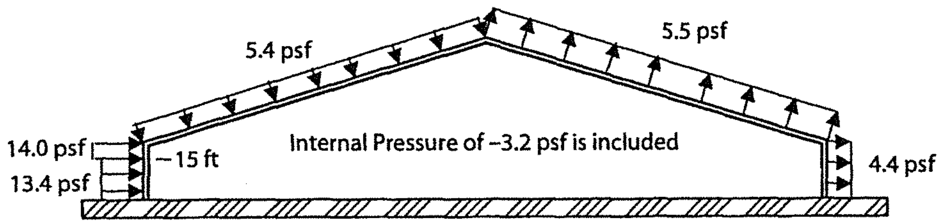


Figure 3-15 Net Design Wind Pressures for MWFRS when Wind is Normal to Ridge with Positive Windward External Roof Pressure Coefficient

External Roof C_p from Figure 6-6 for Wind Parallel to Ridge

For wind parallel to the ridge, $h/L = 36.7/250 = 0.147$ and $\theta < 10°$. The values of C_p for wind parallel to ridge are obtained from Figure 6-6 of the Standard and are shown in Tables 3-21 and 3-22.

Design Wind Load Cases

Section 6.5.12.3 of the Standard requires that any building whose wind loads have been determined under the provisions of Sections 6.5.12.2.1 and 6.5.12.2.3 shall be designed for wind load cases as defined in Figure 6-9 of the Standard. Case 1 includes the loadings shown in Figures 3-14 through 3-17. A combination of windward (P_W) and leeward (P_L) loads is applied for Load Cases 2, 3, and 4 as shown in Figure 6-9 of the Standard. Section 6.5.12.3 of the Standard has exception that if a building is designed with flexible diaphragm, only Load Cases 1 and 3 need to be considered. There is not enough structural information given in this example to assess flexibility of roof diaphragm. Structural designer will have to make judgment in each building.

Design Pressures for C&C (Section 6.5.12.4)

Eq. 6-22 of the Standard is used to obtain the design pressures for components and cladding:

$$p = q_h[(GC_p) - (GC_{pi})] \qquad \text{(Eq. 6-22)}$$

where

q_h = 18.0 psf
(GC_p) = Values obtained from Figure 6-11
(GC_{pi}) = ± 0.18 for this building

Wall C&C Pressures

The pressure coefficients (GC_p) (Table 3-23) are a function of effective wind area. The definition of effective wind area for a component or clad-

Table 3-21 Roof C_p (Wind Parallel to Ridge)

Surface	*h/L*	*Distance from windward edge*	C_p
Roof	≤ 0.5	0 to h	–0.9, –0.18*
		h to $2h$	–0.5, –0.18*
		> $2h$	–0.3, –0.18*

*The values of smaller uplift pressures on the roof can become critical when wind load is combined with roof live load or snow load; load combinations are given in Sections 2.3 and 2.4 of the Standard. For brevity, loading for this value is not shown in this example.

Table 3-22 MWFRS Pressures: Wind Parallel to Ridge

Surface	*z (ft)*	*q (psf)*	*G*	C_p	*Net pressure psf with* $(+GC_{pi})$	*Net pressure psf with* $(-GC_{pi})$
Windward wall	0–15	15.0	0.85	0.8	7.0	13.4
	20	15.8	0.85	0.8	7.5	14.0
	30	17.3	0.85	0.8	8.5	15.0
	40	18.3	0.85	0.8	9.2	15.7
	53.3	19.4	0.85	0.8	10.0	16.4
Leeward wall	All	18.0	0.85	–0.45	–10.1	–3.7
Side walls	All	18.0	0.85	–0.7	–14.0	–7.5
Roof*	0 to h*	18.0	0.85	–0.9	–17.0	–10.5
	h to $2h$*	18.0	0.85	–0.5	–10.9	–4.4
	> 2h*	18.0	0.85	–0.3	–7.8	–1.4

q_h = 18.0 psf; $(GC_{pi}) = \pm 0.18$; h = 36.7 ft; $q_h(GC_{pi}) = \pm 3.2$ psf.

*Distance from windward edge.

ding panel is the span length multiplied by an effective width that need not be less than one-third the span length; however, for a fastener it is the area tributary to an individual fastener.

Girt:

larger of

$$A = 25(6.67) = 167 \text{ ft}^2$$

or

$$A = 25(25/3) = 208 \text{ ft}^2 \text{ (controls)}$$

Wall Panel:

larger of

$$A = 6.67(2) = 13.3 \text{ ft}^2$$

or

$$A = 6.67(6.67/3) = 14.8 \text{ ft}^2 \text{ (controls)}$$

Fastener:

$$A = 6.67(1) = 6.7 \text{ ft}^2$$

Typical calculations of design pressures for girt in Zone 4 are shown below and wall C&C pressures are summarized in Table 3-24:

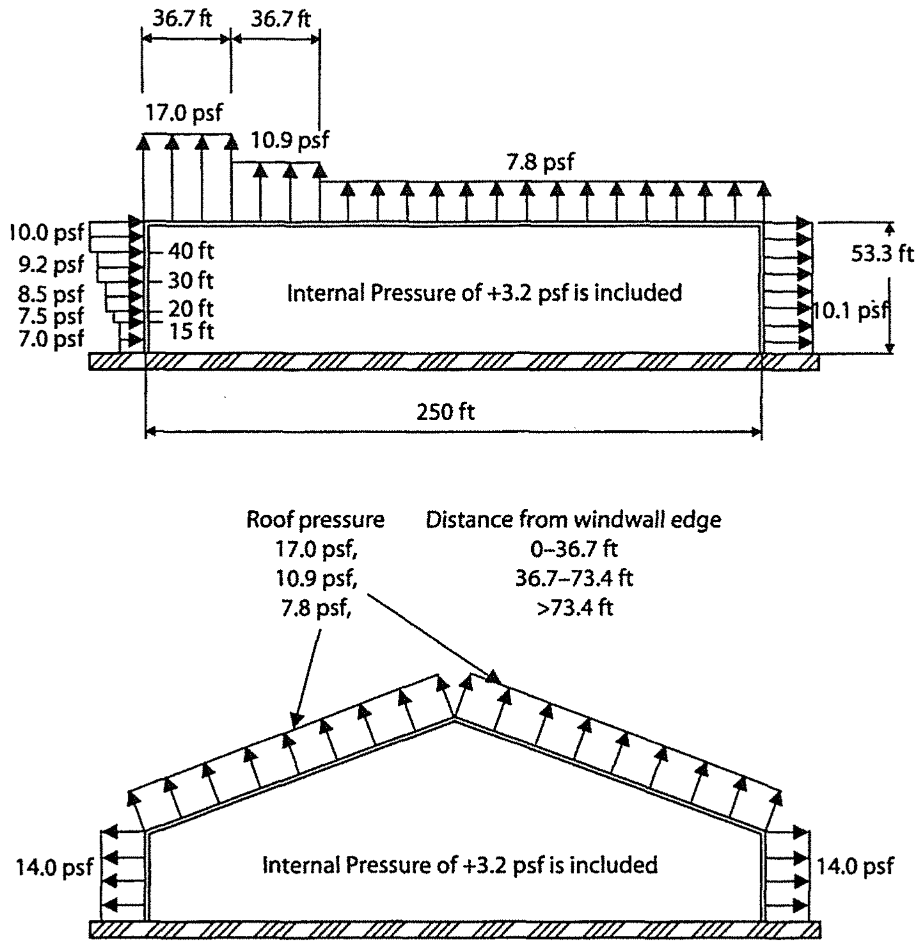


Figure 3-16 Net Design Wind Pressures for MWFRS when Wind is Parallel to Ridge with Positive Internal Pressure

Table 3-23 Wall Coefficients (GC_p) in Figure 6-11A

		External (GC_p)		
C&C	$A(ft^2)$	*Zones 4 and 5*	*Zone 4*	*Zone 5*
Girt	208	0.77*	–0.87	–0.93
Panel	14.8	0.97	–1.07	–1.34
Fastener	6.7	1.00	–1.10	–1.40
Other**	≤ 10	1.00	–1.10	–1.40
Other**	≥ 500	0.70	–0.80	–0.80

*(GC_p) values are obtained using equations in Chapter 2 of this guide.

**Other C&C can be doors, windows, etc.

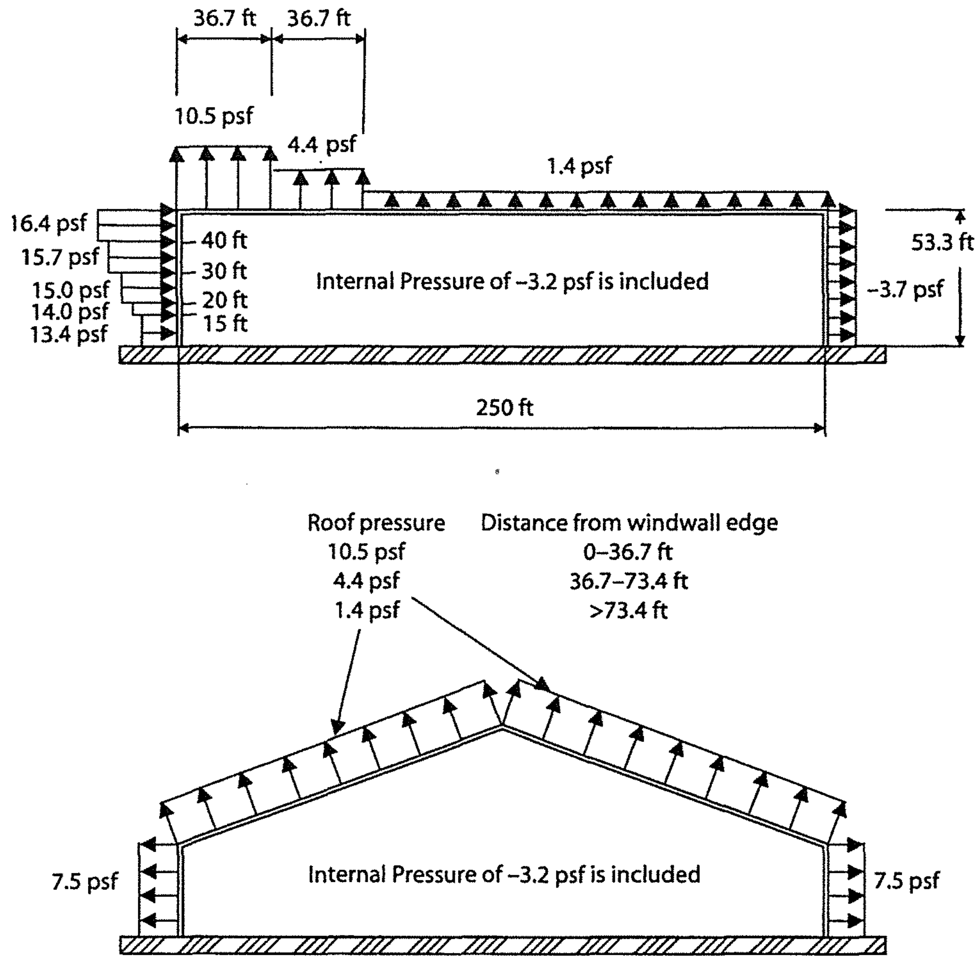


Figure 3-17 Net Design Wind Pressures for MWFRS when Wind is Parallel to Ridge with Negative Internal Pressure

Table 3-24 Net Controlling Wall Component Pressures (psf)

	Controlling design pressures (psf)			
	Zone 4		*Zone 5*	
C&C	*Positive*	*Negative*	*Positive*	*Negative*
Girt	17.1	–18.9	17.1	–20.0
Panel	20.7	–22.5	20.7	–27.4
Fastener	21.2	–23.0	21.2	–28.4
$A \leq 10$ ft^2	21.2	–23.0	21.2	–28.4
$A \geq 500$ ft^2	15.8	–17.6	15.8	–17.6

For maximum negative pressure:

$$p = 18.0[(-0.87) - (\pm 0.18)]$$

$p = -18.9$ psf with positive internal pressure (controls)

$p = -12.4$ psf with negative internal pressure

For maximum positive pressure:

$$p = 18.0[(0.77) - (\pm 0.18)]$$

$p = 10.6$ psf with positive internal pressure

$p = 17.1$ psf with negative internal pressure (controls)

Roof C&C Pressures

Effective wind areas of roof C&C (Table 3-25):

Purlin:
larger of
$A = 25(5) = 125 \text{ ft}^2$
or
$A = 25(25/3) = 208 \text{ ft}^2$ (controls)

Panel:
larger of
$A = 5(2) = 10 \text{ ft}^2$ (controls)
or
$A = 5(5/3) = 8.3 \text{ ft}^2$

Fastener:
$A = 5(1) = 5 \text{ ft}^2$

Typical calculations of design pressures for purlin in Zone 1 are as follows and roof C&C pressures are summarized in Table 3-26:

For maximum negative pressure:

$$p = 18.0[(-0.8) - (\pm 0.18)]$$

$p = -17.6$ psf with positive internal pressure (controls)

$p = -11.2$ psf with negative internal pressure

Table 3-25 Roof Coefficients (GC_p) in Figure 6-11C; $7° < \theta \leq 27°$

Component	*A* (ft^2)	*External* (GC_p) *Zones 1, 2, and 3*	*Zone 1*	*Zone 2*	*Zone 3*
Purlin	208	0.3	–0.8	–1.2	–2.0
Panel	10	0.5	–0.9	–1.7	–2.6
Fastener	5	0.5	–0.9	–1.7	–2.6
Other*	≤10	0.5	–0.9	–1.7	–2.6
Other*	≥100	0.3	–0.8	–1.2	–2.0

*Other C&C can be skylight, etc.

Table 3-26 Net Controlling Roof Component Pressures (psf)

	Controlling design pressures (psf)			
	Positive	*Negative*		
Component	*Zones 1, 2, and 3*	*Zone 1*	*Zone 2*	*Zone 3*
Purlin	10.0*	–17.6	–24.8	–39.2
Panel	12.2	–19.4	–33.8	–50.0
Fastener	12.2	–19.4	–33.8	–50.0
$A \leq 10\ \text{ft}^2$	12.2	–19.4	–33.8	–50.0
$A \geq 500\ \text{ft}^2$	10.0*	–17.6	–24.8	–39.2

*Minimum net pressure controls (Section 6.1.4.2 of the Standard).

For maximum positive pressure:

$p = 18.0[(0.3) - (\pm 0.18)]$

$p = 2.1$ psf with positive internal pressure

$p = 8.6$ psf with negative internal pressure

$p = 10$ psf minimum net pressure (controls)
(Section 6.1.4.2 of the Standard)

Special case of girt that transverses Zones 4 and 5:

Width of Zone 5:

smaller of

$a = 0.1(200) = 20$ ft

or

$a = 0.4(36.7) = 14.7$ ft (controls)

but not less than

$0.04(200) = 8$ ft

or 3 ft

Weighted average design pressure:

$$P = \frac{14.7(-20.0) + 10.3(-18.9)}{25} = -19.6 \text{ psf}$$

This procedure of using a weighted average may be used for other components and cladding.

Special Case of Strut Purlin (interior)

Strut purlins in the end bay experience combined uplift pressure as a roof component (C&C) and axial load as part of the MWFRS

Component Pressure

End bay purlin located in Zones 1 and 2

Width of Zone 2, $a = 14.7$ ft

Weighted average design pressure:

$$= \frac{14.7(-24.8) + 10.3(-17.6)}{25} = -21.9 \text{ psf}$$

(Purlin in Zones 2 and 3 will have higher pressure)

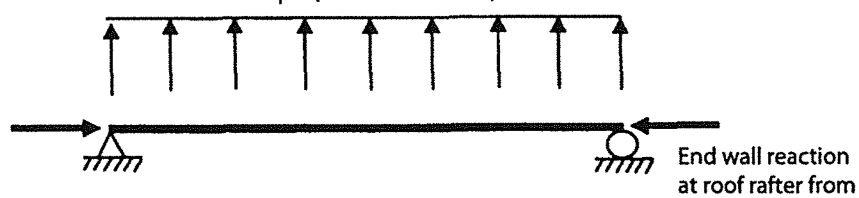


Figure 3-18A Combined Uplift and Axial Design Loads on Interior Strut Purlin

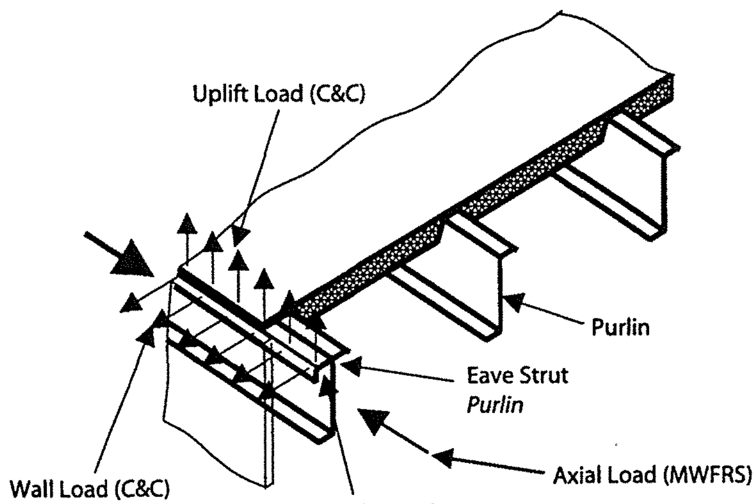


Figure 3-18B Eave Strut Purlin Supports Roof and Wall Panels

MWFRS Load

Figure 3-16 shows design pressure on end wall with wind parallel to ridge with positive internal pressure (consistent with high uplift on the purlin). Assuming that the end wall is supported at the bottom and at the roof line, the effective axial load on an end bay purlin can be determined.

Combined Design Loads on Interior Strut Purlin

Figure 3-18A shows combined design load on interior strut purlin. Note that many metal building manufacturers support the top of the wall panels with the eave strut purlin (see Figure 3-18B). For this case, the eave purlin also serves as a girt, and the negative wall pressures of Zones 5 and 4 would occur for the same wind direction as the maximum negative uplift pressures on the purlin (refer to Zones 3 and 2). Thus, in this instance, the correct load combination would involve biaxial bending loads based on C&C pressures combined with the MWFRS axial load.

3.7 Example 7 Building of Ex. 6 Using Low-Rise Building Provisions

This example illustrates the use of the low-rise building provisions to determine design pressures for the MWFRS. For this purpose, the building used has the same dimensions as the building in Ex. 6 (Section 3.6 of this guide). The design pressures on C&C will be the same as Ex. 6. The building is shown in Figure 3-13. The building data are as follows:

Location:	Memphis, Tennessee
Terrain:	Flat farmland
Dimensions:	200 ft × 250 ft in plan Eave height of 20 ft Roof slope 4:12 (18.4°)
Framing:	Rigid frame spans the 200-ft direction Rigid frame bay spacing is 25 ft Lateral bracing in the 250-ft direction is provided by a "wind truss" spanning the 200 ft to side walls and cable/rod bracing in the planes of the walls Openings uniformly distributed

Low-Rise Building

Section 6.2 of the Standard specifies two requirements for a building to qualify as a low-rise building: (1) mean roof height has to be less than or equal to 60 ft, and (2) mean roof height does not exceed least horizontal dimension. A building with these dimensions qualifies as a low-rise building and the alternate provisions of Section 6.5.12.2.2 may be used.

Exposure, Building Classification, and Basic Wind Speed

Same as Ex. 6:

Exposure C

Category II

Enclosed building (openings uniformly distributed)

$V = 90$ mph

Velocity Pressure

The low-rise building provisions for MWFRS in the Standard use the velocity pressure at mean roof height, h, for calculation of all external and internal pressures, including the windward wall. All pressures for a given zone are assumed to be uniformly distributed with respect to height above ground.

Mean roof height $h = 36.7$ ft

The velocity pressures are computed using:

$$q_h = 0.00256 K_h K_{zt} K_d V^2 I \text{ (psf)} \qquad \text{(Eq. 6-15)}$$

where

q_h = Velocity pressure at mean roof height, h
K_h = 1.02 for Exposure C (see Table 6-3 of the Standard)
K_{zt} = 1.0 topographic factor (see Section 6.5.7.1)
K_d = 0.85 (see Table 6-4)

V = 90 mph basic wind speed (see Figure 6-1)
I = 1.0 for Category II (50-yr mean return interval)

Therefore:

$$q_h = 0.00256(1.02)(1.0)(0.85)(90)^2(1.0) = 18.0 \text{ psf}$$

Design Pressures for the MWFRS

The equation for the determination of design wind pressures for MWFRS for low-rise buildings is given by Eq. 6-18 in Section 6.5.12.2.2 of the Standard:

$$p = q_h[(GC_{pf}) - (GC_{pi})] \qquad \text{(Eq. 6-18)}$$

where

q_h = The velocity pressure at mean roof height associated with Exposure C
(GC_{pf}) = The external pressure coefficients from Figure 6-10 of the Standard
(GC_{pi}) = The internal pressure coefficients from Figure 6-5 of the Standard

The building must be designed for all wind directions using the eight loading patterns shown in Figure 6-10 of the Standard. For each of these patterns, both positive and negative internal pressures must be considered, resulting in a total of 16 separate loading conditions. However, if the building is symmetrical, the number of separate loading conditions will be reduced to eight (two directions of MWFRS being designed for normal load and torsional load cases—a total of four load cases, one windward corner, and two internal pressures). The load patterns are applied to each building corner in turn as the reference corner.

External Pressure Coefficients (GC_{pf})

The roof and wall coefficients are functions of the roof slope, θ (see Tables 3-27 and 3-28).

Width of end zone surface:
smaller of
$2a = 2(0.1)(200) = 40$ ft
or
$2(0.4)(36.7) = 29.4$ ft (controls)
but not less than
$2(0.04)(200) = 16$ ft
or $2(3) = 6$ ft

Internal Pressure Coefficients (GC_{pi})

Openings are assumed to be evenly distributed in the walls, and since Memphis, Tennessee, is not located in a hurricane-prone region, the building qualifies as an enclosed building (see Section 6.2 of the Standard). The internal pressure coefficients are given from Figure 6-5 as $(GC_{pi}) = \pm 0.18$.

Design Wind Pressures (psf)

Design wind pressures in the transverse and longitudinal directions are shown in Tables 3-29 and 3-30.

Table 3-27 Transverse Direction ($\theta = 18.4°$)

	Building surface									
GC_{pf}*	*1*	*2*	*3*	*4*	*5*	*6*	*1E*	*2E*	*3E*	*4E*
	0.52	−0.69	−0.47	−0.42	−0.45	−0.45	0.78	−1.07	−0.67	−0.62

*By linear interpolation.

Table 3-28 Longitudinal Direction ($\theta = 0°$)

	Building surface									
GC_{pf}	*1*	*2*	*3*	*4*	*5*	*6*	*1E*	*2E*	*3E*	*4E*
	0.40	−0.69	−0.37	−0.29	−0.45	−0.45	0.61	−1.07	−0.53	−0.43

Table 3-29 Design Wind Pressures, Transverse Direction

		Design pressure (psf)	
Building surface	(GC_{pf})	$(+GC_{pi})$	$(-GC_{pi})$
1	0.52	6.1	12.6
2	−0.69	−15.6	−9.2
3	−0.47	−11.7	−5.2
4	−0.42	−10.8	−4.3
5	−0.45	−11.3	−4.9
6	−0.45	−11.3	−4.9
1E	−0.78	10.8	17.3
2E	−1.07	−22.5	−16.0
3E	−0.67	−15.3	−8.8
4E	−0.62	−14.4	−7.9

Table 3-30 Design Wind Pressures, Longitudinal Direction

		Design pressure (psf)	
Building surface	(GC_{pf})	$(+GC_{pi})$	$(-GC_{pi})$
1	0.40	4.0	10.5
2	−0.69	−15.6	−9.2
3	−0.37	−9.9	−3.4
4	−0.29	−8.5	−2.0
5	−0.45	−11.3	−4.9
6	−0.45	−11.3	−4.9
1E	0.61	7.7	14.2
2E	−1.07	−22.5	−16.0
3E	−0.53	−12.8	−6.3
4E	−0.43	−11.0	−4.5

Calculation for Surface 1:

$$p = 18.0\ [0.52 - (\pm 0.18)] = +6.1 \text{ or } +12.6$$

Application of Pressures on Building Surfaces 2 and 3

Note 8 of Figure 6-10 of the Standard states that when the roof pressure coefficient, GC_{pf}, is negative in Zone 2, it shall be applied in Zone 2 for a distance from the edge of the roof equal to 0.5 times the horizontal dimension of the building measured parallel to the direction of the MWFRS being designed or 2.5h, whichever is less. The remainder of Zone 2 that extends to the ridge line shall use the pressure coefficient GC_{pf} for Zone 3. Thus, the distance from the edge of the roof is the smaller of:

$$0.5(200) = 100 \text{ ft for transverse direction}$$
$$0.5(250) = 125 \text{ ft for longitudinal direction}$$

or

$$(2.5)(36.7) = 92 \text{ ft for both directions (controls)}$$

Therefore, Zone 3 applies over a distance of 105 – 92 = 13 ft in what is normally considered to be Zone 2 (adjacent to ridge line) for transverse direction and 125 – 92 = 33 ft for longitudinal direction.

Loading Cases

Because the building is symmetrical, the four loading cases provide all the required combinations provided the design is accomplished by applying loads for each of the four corners. The load combinations illustrated in Figures 3-19 through 3-22 are to be used to design the rigid frames, the "wind truss" spanning across the building in the 200-ft direction, and the rod/cable bracing in the planes of the walls (see Figure 3-13) (Section 3.6 of this guide).

Torsional Load Cases

Since the mean roof height, h = 36.7 ft, is greater than 30 ft and if the roof diaphragm is assumed to be rigid, torsional load cases need to be considered (see exception in Note 5 in Figure 6-10 of the Standard if building is designed with flexible diaphragm). Pressures in "T" zones are 25% of the

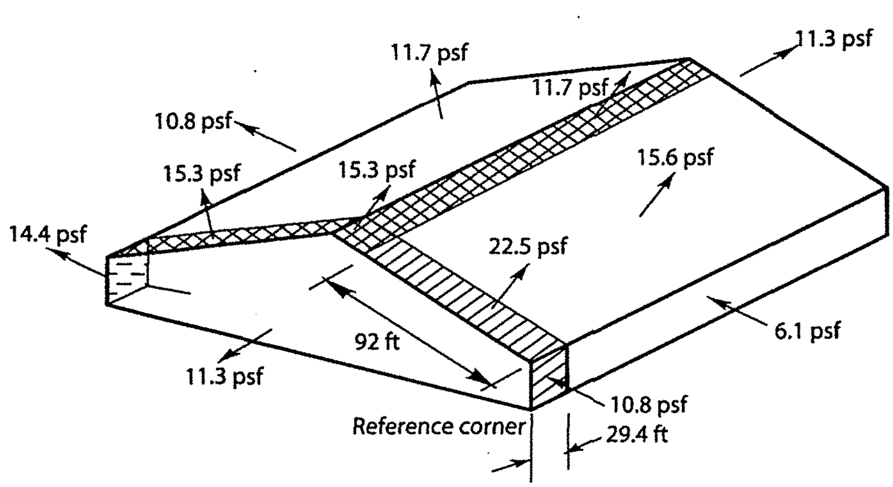


Figure 3-19 Design Pressures for Transverse Direction with Positive Internal Pressure

Note: The pressures are assumed to be uniformly distributed over each of the surfaces shown

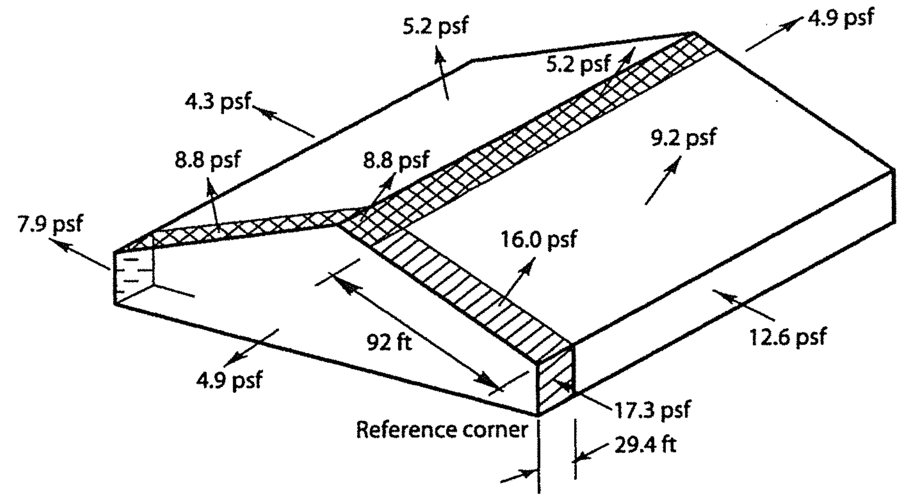


Figure 3-20 Design Pressures for Transverse Direction with Negative Internal Pressure
Note: The pressures are assumed to be uniformly distributed over each of the surfaces shown

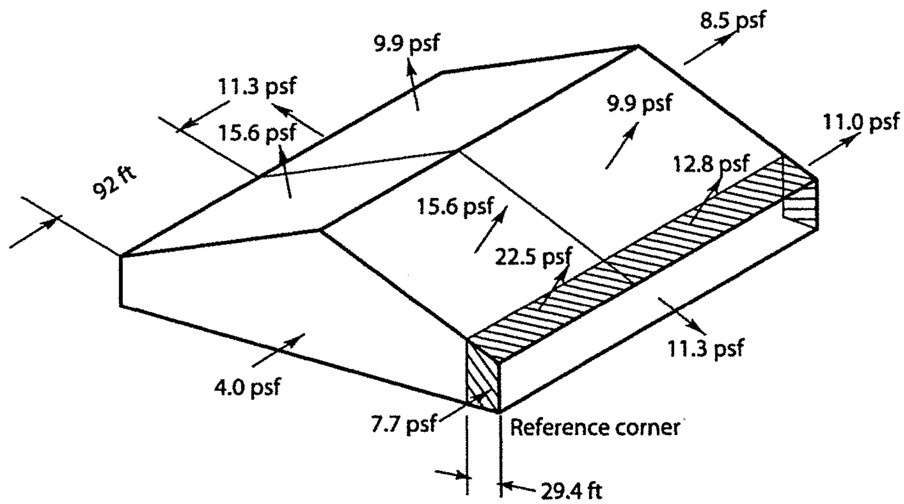


Figure 3-21 Design Pressures for Longitudinal Direction with Positive Internal Pressure
Note: The pressures are assumed to be uniformly distributed over each of the surfaces shown

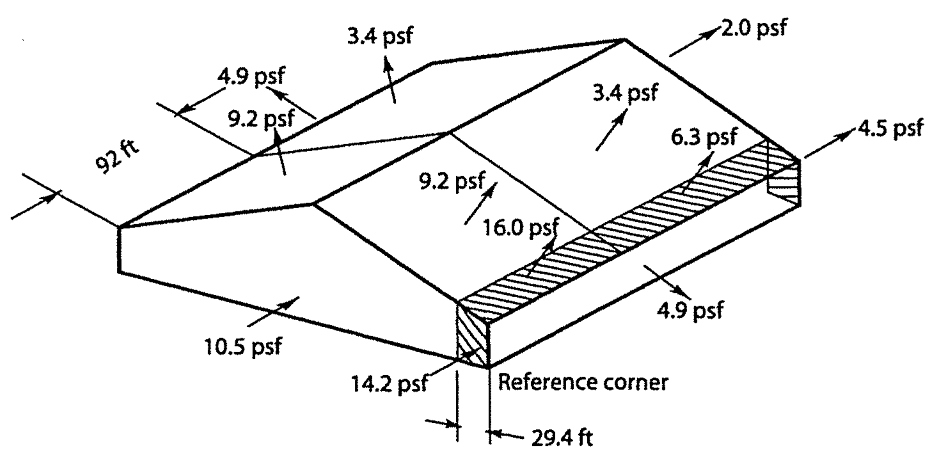


Figure 3-22 Design Pressures for Longitudinal Direction with Negative Internal Pressure
Note: The pressures are assumed to be uniformly distributed over each of the surfaces shown

Table 3-31 Design Wind Pressure for Zone "T," Transverse Direction

Building surface	Design pressures (psf)	
	$(+GC_{pi})$	$(-GC_{pi})$
1T	1.5	3.2
2T	−3.9	−2.3
3T	−2.9	−1.3
4T	−2.7	−1.1

Table 3-32 Design Wind Pressure for Zone "T," Longitudinal Direction

Building surface	Design pressures (psf)	
	$(+GC_{pi})$	$(-GC_{pi})$
1T	1.0	2.6
2T	−3.9	−2.3
3T	−2.5	−0.9
4T	−2.1	−0.5

full design pressures; the "T" zones are shown in Figure 6-10 of the Standard. Other surfaces will have the full design pressures. The "T" zone pressures with positive and negative internal pressures for transverse and longitudinal directions are shown in Tables 3-31 and 3-32, respectively.

Figures 3-19 through 3-26 show design pressure cases for one reference corner; these cases are to be considered for each corner.

Design Wind Pressures for C&C

The design pressures for C&C are the same as shown for Ex. 6 (Section 3.6 of this guide).

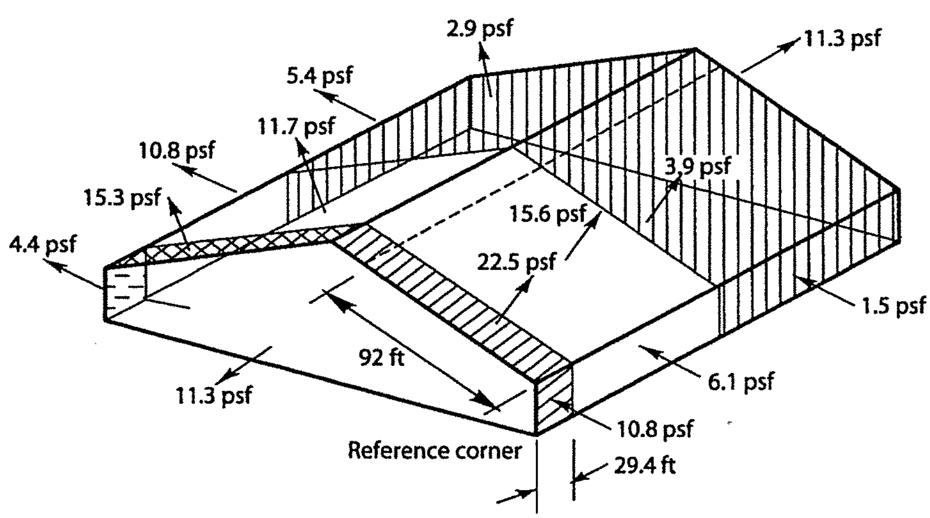


Figure 3-23 Torsional Load Case for Transverse Direction with Positive Internal Pressure

Notes: (1) The pressures are assumed to be uniformly distributed over each of the surfaces shown (2) Roof pressures of 22.5, 15.6, and 3.9 psf apply up to 92 ft; the remaining 13 ft up to the ridge line will have pressure of 15.3, 11.7, and 2.9 psf

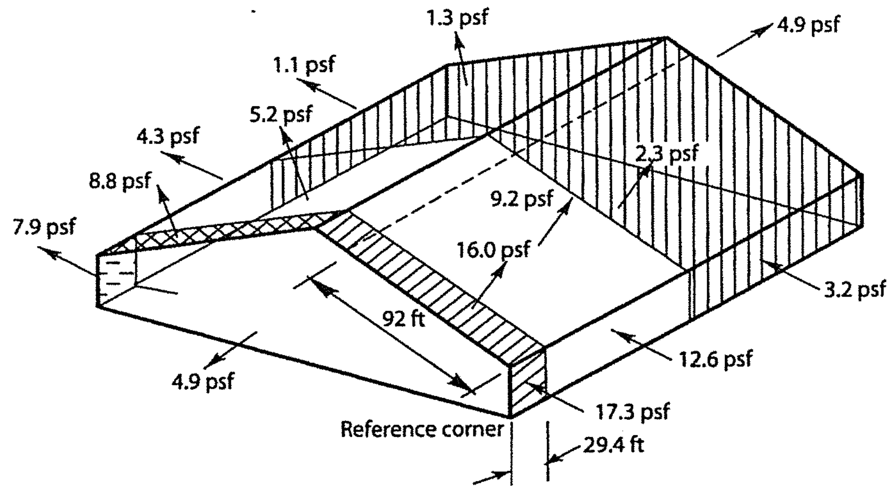


Figure 3-24 Torsional Load Case for Transverse Direction with Negative Internal Pressure

Notes: (1) The pressures are assumed to be uniformly distributed over each of the surfaces shown (2) Roof pressures of 16.0, 9.2, and 2.3 psf apply up to 92 ft; the remaining 13 ft up to the ridge line will have pressures of 8.8, 5.2, and 1.3 psf

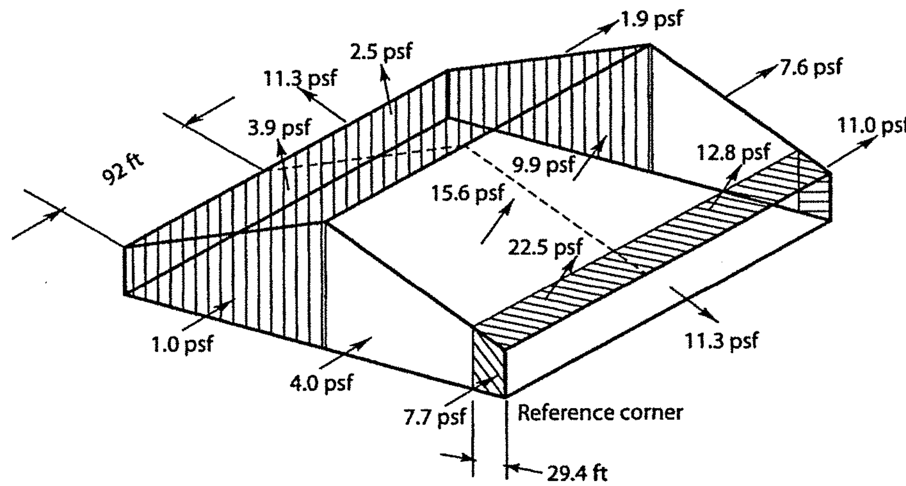


Figure 3-25 Torsional Load Case for Longitudinal Direction with Positive Internal Pressure

Note: The pressures are assumed to be uniformly distributed over each of the surfaces shown

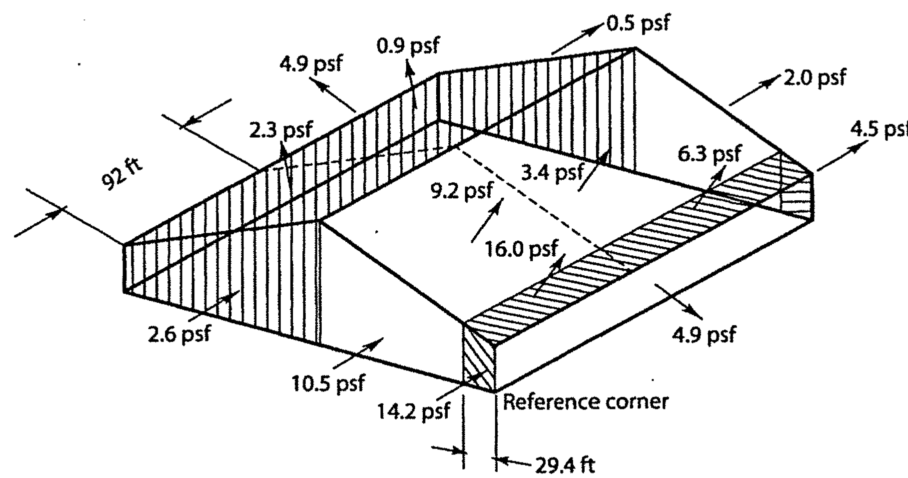


Figure 3-26 Torsional Load Case for Longitudinal Direction with Negative Internal Pressure

Note: The pressures are assumed to be uniformly distributed over each of the surfaces shown

3.8 Example 8 40-ft × 80-ft Commercial Building with Monoslope Roof with Overhang

In this example, design pressures for a typical retail store in a strip-mall are determined. The building's dimensions are shown in Figure 3-27. The building data are as follows:

Location:	Boston, Massachusetts, within 1 mi of the coastal mean high watermark
Topography:	Homogeneous
Terrain:	Suburban
Dimensions:	40 ft × 80 ft in plan Monoslope roof with slope of 14° and overhang of 7 ft in plan Wall heights are 15 ft in front and 25 ft in rear
Framing:	Walls of CMU on all sides supported at top and bottom; steel framing in front (80-ft width) to support window glass and doors. Roof joists span 41.2 ft with 7.2-ft overhang spaced at 5 ft on center
Cladding:	Glass and door sizes vary; glazing is not debris-impact–resistant and occupies 50% of front wall (80 ft in width) Roof panels are 2 ft wide and 20 ft long

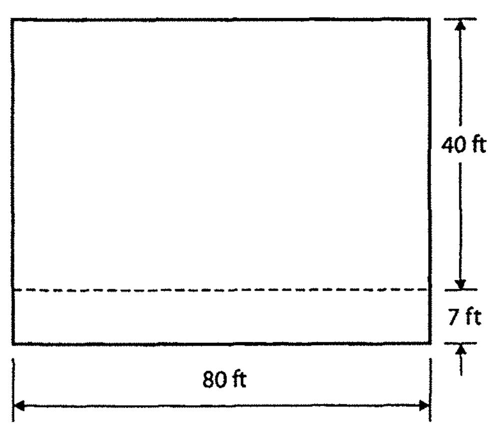
Building Classification, Enclosure Classification, and Exposure Category

The building is not an essential facility, nor is it likely to be occupied by more than 300 persons at any one time. Use Category II (see Table 1-1 of the Standard). Importance Factor $I = 1.00$ (see Table 6-1 of the Standard).

The building is sited in a suburban area and satisfies the criteria for Exposure B (see Section 6.5.6 of the Standard).

The building is sited in a wind-borne debris region. It has glazing (not impact resistant) occupying 50% of a wall that receives positive pressure. The building must be classified as partially enclosed (see Sections 6.5.9.3 and 6.2 of the Standard).

The building does not meet the requirements of Method 1, Simplified Procedure (Section 6.4 of the Standard), because the roof slope is greater than 10°. Therefore, Method 2, Analytical Procedure, is used (see Section 6.5.3



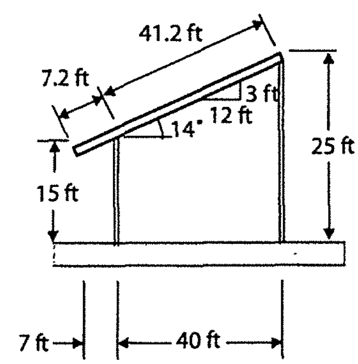


Figure 3-27 Dimensions of the Retail Store Strip-Mall

of the Standard). The roof is not gabled; hence, the low-rise building provisions may not be used.

The provisions of the Standard do not permit use of external pressure coefficients (GC_{pf}) given in Figure 6-10. The values in Figure 6-10 were obtained from wind tunnel studies of rigid, gable-framed buildings. Their use for a monoslope roof requires considerable judgment. The design examples presented in Ex. 7 (Section 3.7 of this guide) illustrate use of the pressure coefficients of Figure 6-10, and the Commentary in the Standard gives the background for (GC_{pf}) values.

Basic Wind Speed

The wind speed contour of 110 mph traverses over Boston, Massachusetts (Figure 6-1c of the Standard); use a basic wind speed of 110 mph.

Velocity Pressures

The velocity pressures (Table 3-33) are calculated using the following equation (see Section 6.5.10 of the Standard):

$$q = 0.00256 K_z K_{zt} K_d V^2 I \text{ (psf)} \qquad \text{(Eq. 6-15)}$$
$$= 0.00256 K_z (1.0)(0.85)(110)^2(1.0)$$
$$= 26.33 K_z \text{ (psf)}$$

where

K_z = Value obtained from Table 6-3 of the Standard
K_{zt} = 1.0 homogeneous terrain
I = 1.0 for Category II building (see Table 6-1)
K_d = 0.85, see Table 6-4

The provisions of the Standard require the use of the external pressure coefficients, C_p, from Figure 6-6; hence, the exposure coefficients, K_z are based on Exposure B, Case 2, for MWFRS and Exposure B, Case 1, for C&C (see Table 6-3).

Design Pressures for MWFRS

The equation for rigid buildings of all heights is given in Section 6.5.12.2 of the Standard as follows:

$$p = qGC_p - q_i(GC_{pi}) \qquad \text{(Eq. 6-17)}$$

where
q = q_z for windward wall
q_i = q_h for windward and leeward walls, side walls and roof

Table 3-33 Velocity Pressures, q_z, q_i, and q_h (psf)

	MWFRS		*C&C*	
Height (ft)	*Exposure B, Case 2*	q_z, q_i	*Exposure B, Case 1*	q_h
0–15	0.57	15.01		
h = 20	0.62	16.32*	0.70	18.43
25	0.66	17.38		

*q_h = 16.32 psf for MWFRS

G = Value determined from Section 6.5.8 of the Standard
C_p = Value obtained from Figure 6-6 of the Standard
(GC_{pi}) = Value obtained from Figure 6-5 of the Standard

For positive internal pressure evaluation, the Standard permits q_i to be conservatively evaluated at height h ($q_i = q_h$). Section 6.5.11.1.1 of the Standard permits the reduction of (GC_{pi}) for a partially enclosed building containing a single, unpartitioned large volume by the following factor:

$$R_i = 0.5\left(1+\frac{1}{\sqrt{1+\frac{V_i}{22,800A_{og}}}}\right)$$

$$= 0.5\left(1+\frac{1}{\sqrt{1+\frac{80\times 40\times 20}{22,800\times 50\%(15\times 80)}}}\right) \cong 1 \qquad \text{(no reduction)}$$

where

V_i = Unpartitioned internal volume
A_{og} = Total area of openings in building envelope (50% of front wall)

Gust Effect Factor, G

The gust effect factor for non-flexible (rigid) buildings is given in Section 6.5.8 of the Standard as $G = 0.85$.

The size of the building would not permit a reduction in G based on Eq. 6-4 of the Standard.

Wall External Pressure Coefficients (C_p) (Table 3-34)

The coefficients for the windward and side walls are given in Figure 6-6 of the Standard as $C_p = +0.8$ and -0.7, respectively. The values for the leeward wall depend on L/B; they are different for the two directions: (1) wind parallel to roof slope (normal to ridge), and (2) wind normal to roof slope (parallel to ridge).

Roof External Pressure Coefficients (C_p) (Table 3-35)

Since the building has a monoslope roof, the roof surface for wind directed parallel to the slope (normal to ridge) may be a windward or a leeward surface. The value of $h/L = 0.5$ in this case, and the proper coefficients are obtained from linear interpolation for $\theta = 14°$.

When wind is normal to the roof slope (parallel to ridge), angle $\theta = 0$ and $h/L = 0.25$.

For the overhang, Section 6.5.11.4.1 of the Standard requires $C_p = 0.8$ for wind directed normal to 15-ft wall. The Standard does not address the leeward overhang for the case of wind directed toward 25-ft wall and perpendicular to roof slope (parallel to ridge). A $C_p = -0.5$ could be used (coefficient for leeward wall), but the coefficient has been conservatively taken as 0.

The building is sited in a hurricane-prone region less than 1 mi from the coastal mean high-water level. The basic wind speed is 110 mph and the glazing is not designed to resist wind-borne debris impact. Thus, as noted earlier, the building must be classified as partially enclosed, despite the

Table 3-34 Wall Pressure Coefficients (C_p)

Surface	*Wind direction*	*L/B*	C_P
Leeward wall	‖ to roof slope	0.5	–0.5
Leeward wall	⊥ to roof slope	2.0	–0.3
Windward wall	–	–	0.8
Side walls	–	–	–0.7

Table 3-35 Roof Pressure Coefficients (C_p)

Wind direction	*h/L*	θ°	C_p
‖ to roof slope	0.5	14	–0.74, –0.18** as windward slope
‖ to roof slope	0.5	14	–0.50 as leeward slope
⊥ to roof slope	0.25	0	–0.90 (0–20 ft)* –0.18** (0–80 ft)* –0.50 (20–40 ft) –0.30 (40–80 ft)

*Distance from the windward edge of the roof.

**The values of smaller uplift pressures on the roof can become critical when wind load is combined with roof live load or snow load; load combinations are given in Sections 2.3 and 2.4 of the Standard. For brevity, loading for this value is not shown here.

openings in the walls and the roof. The internal pressure coefficients, from Figure 6-5 of the Standard are as follows:

$$(GC_{pi}) = +0.55$$

and

$$(GC_{pi}) = -0.55$$

Typical Calculations of Design Pressures for MWFRS (Wind Parallel to Slope with 15-ft Windward Wall) (Table 3-36)

Pressure on Leeward Wall

$$\begin{aligned} p &= q_h GC_p - q_h(\pm GC_{pi}) \\ &= 16.32(0.85)(-0.5) - (16.32)(+0.55) \\ &= -15.9 \text{ psf with positive internal pressure} \end{aligned}$$

and

$$\begin{aligned} &= 16.32(0.85)(-0.5) - (16.32)(-0.55) \\ &= 2.0 \text{ psf with negative internal pressure} \end{aligned}$$

Pressure on Overhang Top Surface

$$\begin{aligned} p &= q_h GC_p \\ &= 16.32(0.85)(-0.74) \\ &= -10.3 \text{ psf} \end{aligned}$$

Table 3-36 Design Pressures for MWFRS: Wind Parallel to Roof Slope (normal to ridge line)

Wind direction	*Surface*	*Z (ft)*	*q_z (psf)*	*Gust effect*	*External C_p**	*Design pressure (psf)* ($+GC_{pi}$)	($-GC_{pi}$)
Windward wall (15 ft)	Windward wall	0–15	15.01	0.85	0.80	1.2	19.2
	Leeward wall	0–25	16.32	0.85	–0.50	–15.9	2.0
	Side wall	All	16.32	0.85	–0.70	–18.7	–0.7
	Roof	–	16.32	0.85	–0.74	–19.2	–1.3
	Overhang top	–	16.32	0.85	–0.74	–10.3**	–10.3**
	Overhang bottom	–	15.01	0.85	0.80	10.2**	10.2**
Windward wall (25 ft)	Windward wall	0–15	15.01	0.85	0.80	1.2	19.2
		15–20	16.32	0.85	0.80	2.1	20.1
		20–25	17.38	0.85	0.80	2.8	20.8
	Leeward wall	All	16.32	0.85	–0.50	–15.9	2.0
	Side wall	All	16.32	0.85	–0.70	–18.7	–0.7
	Roof	–	16.32	0.85	–0.50	–15.9	2.0
	Overhang top	–	16.32	0.85	–0.50	–6.9**	–6.9**
	Overhang bottom	–	–		–	0.0**	0.0**

*External pressure calculations include $G = 0.85$.

**Overhang pressures are not affected by internal pressures. The Standard does not address bottom surface pressures for leeward overhang. It could be argued that leeward wall pressure coefficients can be applied, but note that neglecting the bottom overhang pressures would be conservative in this application.

Pressure on Overhang Bottom Surface (same as windward wall external pressure)

$$
\begin{aligned}
p &= q_z G C_p \\
&= 15.01(0.85)(0.8) \\
&= 10.2 \text{ psf}
\end{aligned}
$$

Note that q_z was evaluated for $z = 15$ ft for bottom surface of overhang as C_p coefficient is based on induced pressures at top of wall.

Figures 3-28 and 3-29 illustrate the external, internal, and combined pressure for wind directed normal to the 15-ft wall. Figures 3-30 and 3-31 illustrate combined pressure for wind directed normal to the 25-ft wall and perpendicular to slope (parallel to ridge line), respectively (Table 3-37).

Design Wind Load Cases

Section 6.5.12.3 of the Standard requires that any building whose wind loads have been determined under the provisions of Sections 6.5.12.2.1 and 6.5.12.2.3 shall be designed for wind load cases as defined in Figure 6-9 of the Standard. Case 1 includes the loadings shown in Figure 3-28 through Figure 3-31. The exception in Section 6.5.12.3 of the standard indicates that a combination of windward (P_W) and leeward (P_L) loads is applied for Load Cases 3 only since mean roof height h of the building is less than 30 ft.

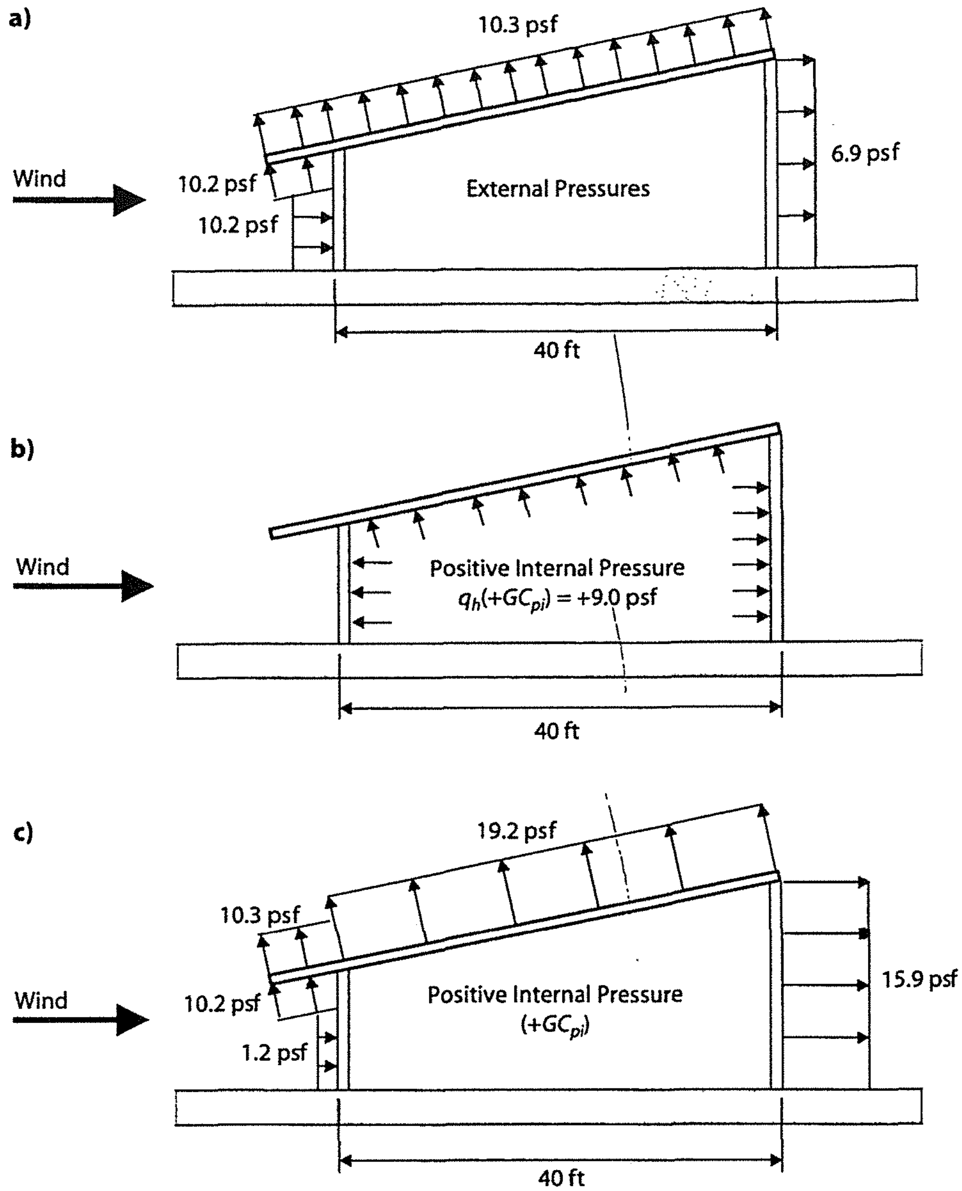


Figure 3-28 Design Pressures for MWRFS; Wind Parallel to Roof Slope, Normal to 15-ft Wall, and Positive Internal Pressure a., External Pressures; b., Positive Internal Pressure; c., Combined External and Positive Internal Pressure

Design Pressures for C&C

The design pressure equation for C&C for building with mean roof height $h \leq 60$ ft is given in Section 6.5.12.4.1 of the Standard:

$$P = q_h[(GC_p) - (GC_{pi})] \qquad \text{(Eq. 6-22)}$$

where

q_h = Velocity pressure at mean roof height associated with Exposure B, Case 1 ($q_h = 18.43$ psf, previously determined)

(GC_p) = External pressure coefficients from Figures 6-11A, 6-11C, and 6-14B of the Standard

(GC_{pi}) = +0.55 and −0.55, previously determined from Figure 6-5 of the Standard

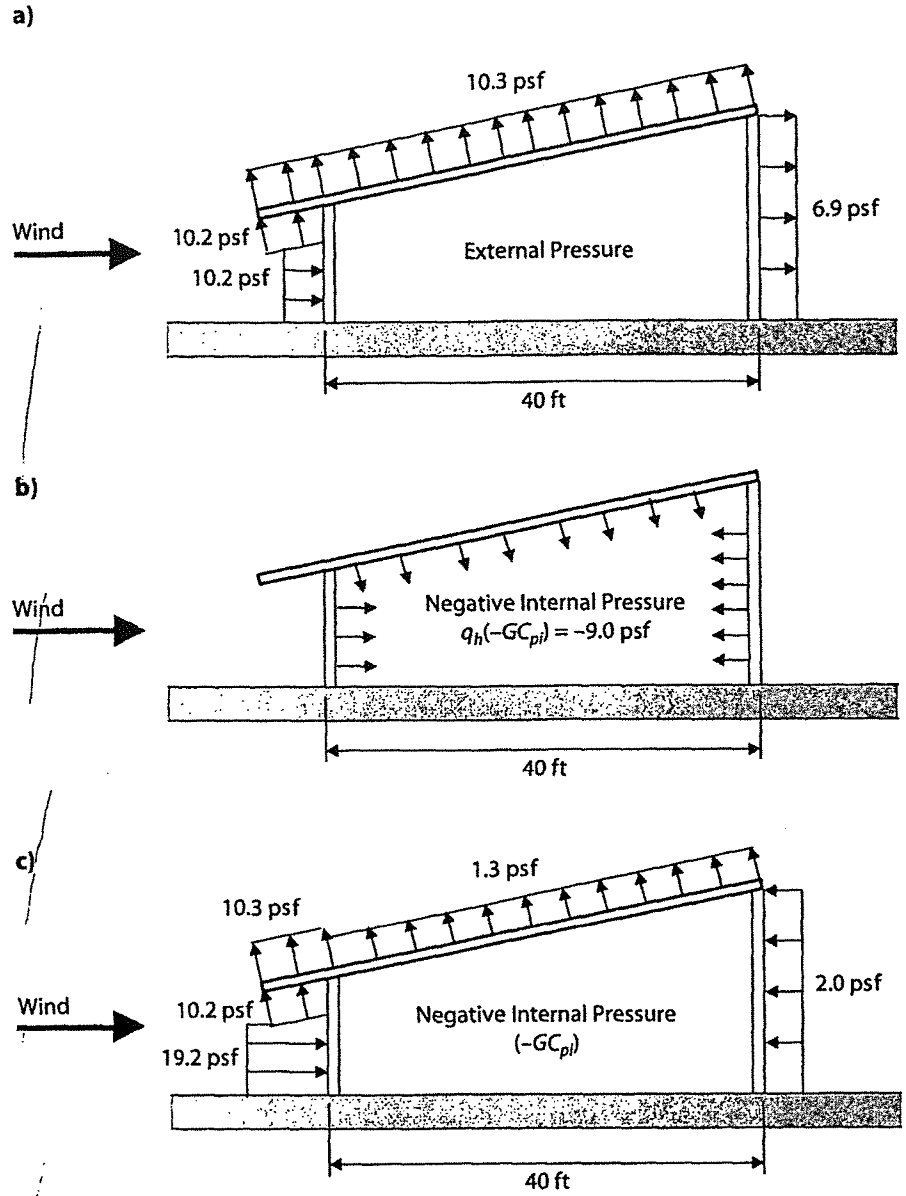


Figure 3-29 Design Pressures for MWRFS; Wind Parallel to Roof Slope, Normal to 15-ft Wall and Negative Internal Pressure a., External Pressures; b., Negative Internal Pressure; c., Combined External and Negative Internal Pressure

Wall Design Pressures (Table 3-38)

Since the CMU walls are supported at the top and bottom, the effective wind area will depend on the span length.

Effective wind area:

For span of 15 ft, $A = 15(15/3) = 75\ \text{ft}^2$
For span of 20 ft, $A = 20(20/3) = 133\ \text{ft}^2$
For span of 25 ft, $A = 25(25/3) = 208\ \text{ft}^2$

Width of Zone 5 (Figure 6-11A):

$$a \le \begin{cases} 0.1(40) = 4 \text{ ft (controls)} \\ 0.4(20) \ = 8 \text{ ft} \end{cases}$$

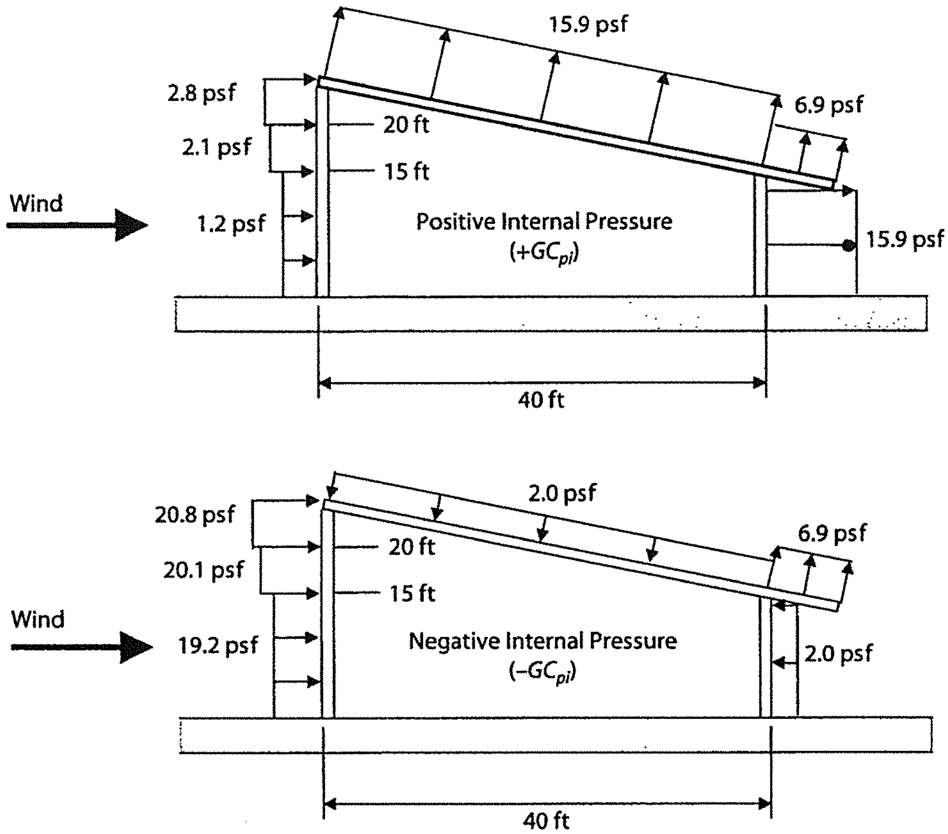


Figure 3-30 Combined Design Pressures for MWRFS; Wind Parallel to Roof Slope (Normal to 25-ft Wall)

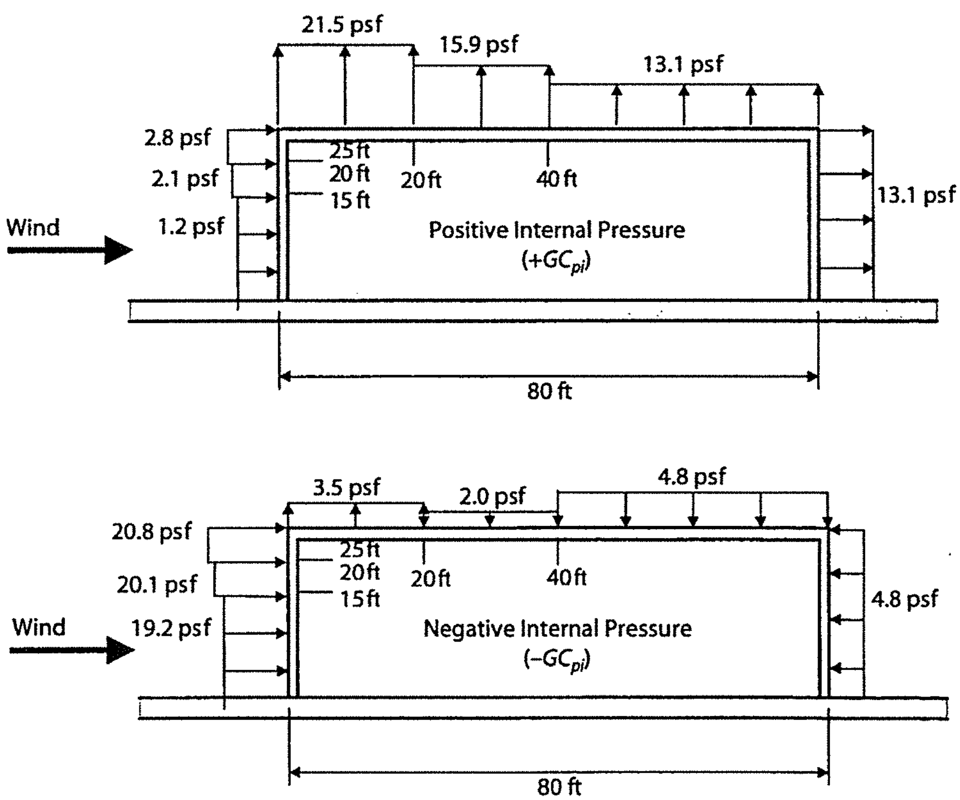


Figure 3-31 Combined Design Pressures for MWRFS; Wind Perpendicular to Roof Slope (Parallel to Ridge Line)

Table 3-37 Design Pressures for MWFRS: Wind Normal to Roof Slope (parallel to ridge line)

Surface	z or Distance (ft)†	q_z* (psf)	Gust effect, G	C_p	Design pressure (psf) +(GC_{pi})**	Design pressure (psf) −(GC_{pi})**
Windward wall	0–15	15.01	0.85	0.8	1.2	19.2
	15–20	16.32	0.85	0.8	2.1	20.1
	20–25	17.38	0.85	0.8	2.8	20.8
Leeward wall	All	16.32	0.85	−0.3	−13.1	4.8
Side wall	All	16.32	0.85	−0.7	−18.7	−0.7
Roof‡	0–20	16.32	0.85	−0.9	−21.5	−3.5
	20–40	16.32	0.85	−0.5	−15.9	2.0
	40–80	16.32	0.85	−0.3	−13.1	4.8

*External pressure calculations include $G = 0.85$.

**Internal pressure is associated with $q_h = 16.32$ psf.

†Distance along roof is from leading windward edge.

‡Pressure on overhang is only external pressure (contribution on underside is conservatively neglected).

Table 3-38 Wall External Pressure Coefficients (GC_p)

A (ft²)	Pressure coefficients: Zones 4 and 5 (+GC_p)	Zone 4 (−GC_p)	Zone 5 (−GC_p)
75	0.85	−0.95	−1.09
133	0.80	−0.90	−1.00
208	0.77	−0.87	−0.93

but not less than

$$a \geq \begin{cases} 0.4(40) = 1.6 \text{ ft} \\ 3 \text{ ft} \end{cases}$$

Design pressures are the critical combinations when the algebraic sum of the external and internal pressures is a maximum.

Typical Calculations for Design Pressures for 15-ft Wall, Zone 4 (Table 3-39)

$$\begin{aligned} p &= q_h[(GC_p) - (\pm GC_{pi})] \\ &= 18.43[(0.85) - (-0.55)] \\ &= 25.8 \text{ psf} \end{aligned}$$

and

$$\begin{aligned} &= 18.43[(-0.95) - (0.55)] \\ &= -27.6 \text{ psf} \end{aligned}$$

The CMU walls are designed for pressures determined for Zones 4 and 5 using appropriate tributary areas.

The design pressures for doors and glazing can be assessed by using appropriate pressure coefficients associated with their effective wind areas.

Table 3-39 Wall Design Pressures (psf)

	Design pressures (psf)		
Wall height (ft)	*Zones 4 and 5 Positive*	*Zone 4 Negative*	*Zone 5 Negative*
15	25.8	–27.6	–30.2
20	24.9	–26.7	–28.6
25	24.3	–26.2	–27.3

Note: q_h = 18.43 psf.

Roof Design Pressures (Tables 3-40 and 3-41)

Effective wind area:

Roof joist:

$$A = (41.2)(5) = 206 \text{ ft}^2$$

or

$$= (41.2)(41.2/3) = 566 \text{ ft}^2 \text{ (controls)}$$

Roof panel:

$$A = (5)(2) = 10 \text{ ft}^2 \text{ (controls)}$$

or

$$= (5)(5/3) = 8.3 \text{ ft}^2$$

Had the effective wind area of the roof joist been greater than 700 ft^2, its external pressure coefficients (GC_p) would still have been determined on the basis of components and cladding. The statement in Section 6.5.12.1.3 of the Standard, in which provisions for MWFRS may be used for a major component, is valid only when the tributary area is greater than 700 ft^2. The tributary area for the roof joist is 242 ft^2.

Section 6.5.11.4.2 of the Standard requires that pressure coefficients for components and cladding of roof overhangs be obtained from Figure 6-11C. Note that the zones for roof overhangs in Figure 6-11C are different from the zones for a monoslope roof in Figure 6-14B.

Width of zone distance a:

$$a \le \begin{cases} 0.1(40) = 4 \text{ ft (controls)} \\ 0.4(20) \;=\; 8 \text{ ft} \end{cases}$$

but not less than

$$a \ge \begin{cases} 0.4(40) = 1.6 \text{ ft} \\ 3 \text{ ft} \end{cases}$$

The widths and lengths of Zones 2 and 3 for a monoslope roof are shown in Figure 6-14B of the Standard (they vary from a to $4a$); for overhangs, widths and lengths are shown in Figure 6-11C.

Similar to the determination of design pressures for walls, the critical design pressures for roofs are the algebraic sum of the external and internal pressures. The design pressures for overhang areas are based on pressure coefficients obtained from Figure 6-11C of the Standard.

Table 3-40 Roof External Pressure Coefficients (GC_p), $\theta = 14°$

		Pressure coefficient, Figure 6-14B			
Component	A (ft^2)	*Zones 1, 2, and 3* ($+GC_p$)	*Zone 1* ($-GC_p$)	*Zone 2* ($-GC_p$)	*Zone 3* ($-GC_p$)
Joist panel	566	0.3	–1.1	–1.2	–2.0
	10	0.4	–1.3	–1.6	–2.9
		Pressure coefficient, Figure 6-11C			
		Zones 1, 2, and 3 ($+GC_p$)	*Zone 1* ($-GC_p$)	*Zone 2** ($-GC_p$)	*Zone 3** ($-GC_p$)
Joist panel	566	0.3	–0.8	–2.2	–2.5
	10	0.5	–0.9	–2.2	–3.7

* Values are from overhang chart in Figure 6-11C

Table 3-41 Roof Design Pressures (psf)

	Design pressures (psf)			
Component	*Zones 1, 2, and 3** *Positive*	*Zone 1* *Negative*	*Zone 2* *Negative*	*Zone 3* *Negative*
Joist	15.7	–30.4	–32.2	–47.0
Joist overhang	10.0**	–14.7	–40.6	–46.1
Panel	17.5	–34.1	–39.6	–63.6
Panel in overhang	10.0**	–16.6	–40.6	–68.2

Notes: q_h = 18.43 psf

*Zones for overhang are in accordance with Figure 6-11C of the Standard.

**Section 6.1.4.2 of the Standard requires minimum of 10 psf.

Typical Calculations for Joist Pressures

Zone 2:

$$
\begin{aligned}
p &= q_h[(GC_p) - (\pm GC_{pi})] \\
&= 18.43[(0.3) - (-0.55)] \\
&= 15.7 \text{ psf}
\end{aligned}
$$

and

$$
\begin{aligned}
&= 18.43[(-1.2) - (0.55)] \\
&= -32.2 \text{ psf}
\end{aligned}
$$

Zones for the monoslope roof and for overhang are shown in Figure 3-32. The panels are designed for the pressures indicated.

Roof joist design pressures need careful interpretation. The high pressures in corner or eave areas do not occur simultaneously at both ends. Two loading cases: wind loadings 1, 2 for joist 1 and wind loadings 3, 4 for joist 2, are shown in Figure 3-32 based on the following zones:

- Joist 1, loading 1: Zones 2 and 3 for roof and Zone 2 for overhang
- Joist 1, loading 2: Zone 2 for roof and Zones 2 and 3 for overhang
- Joist 2, loading 3: Zones 1 and 2 for roof and Zone 1 for overhang
- Joist 2, loading 4: Zone 1 for roof and Zones 1 and 2 for overhang

For simplicity, only one zone is used for overhang pressures in Figure 3-32.

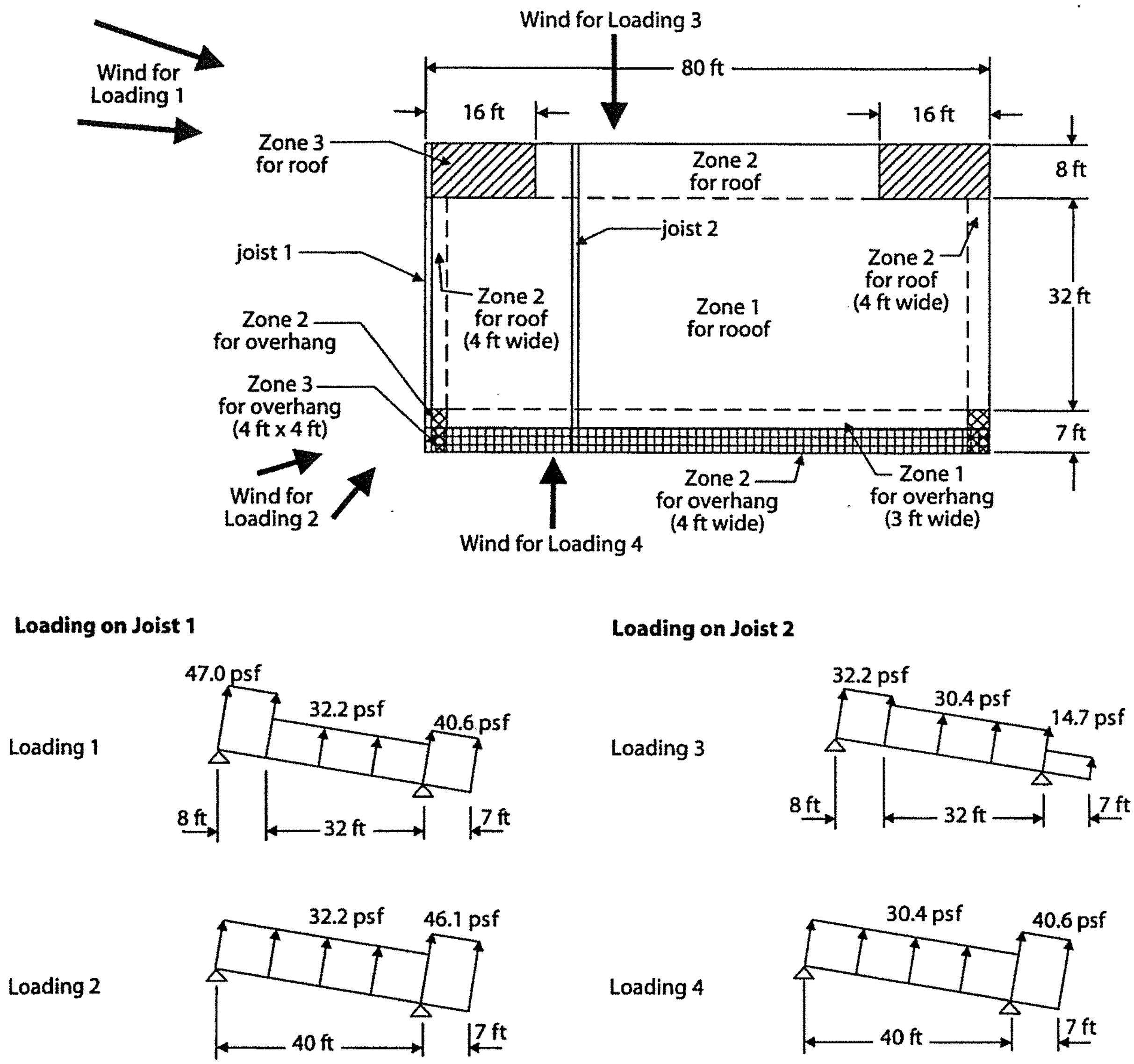


Figure 3-32 Design Pressures for Typical Joists and Pressure Zones for Roof Components and Cladding

3.9 Example 9 U-Shaped Apartment Building

This example demonstrates calculation of wind loads for a U-shaped apartment building, shown in Figure 3-33. Data for the building are provided below:

Location:	Birmingham, Alabama
Topography:	Homogeneous
Terrain:	Suburban
Dimensions:	170 ft × 240 ft overall in plan Roof eave height of 30 ft Hip roof with 5 on 12 pitch
Framing:	Typical timber construction Wall studs are spaced at 16 in. on center, 10 ft tall Roof rafters are spaced at 16 in. on center, spanning 15 ft between interior or exterior bearing walls Floor and roof slabs provide diaphragm action
Cladding:	Location is outside a wind-borne debris region, so no glazing protection is required. Window units are 3 ft × 4 ft

The building is non-symmetrical, and therefore is ineligible for design by Method 1, Simplified Procedure, of ASCE 7-02. Method 2, Analytical Procedure is used. The building is less than 60 ft tall, so it is possible to use low-rise provisions of Section 6.5.12.2.2. However, because U-, T-, and L-shaped

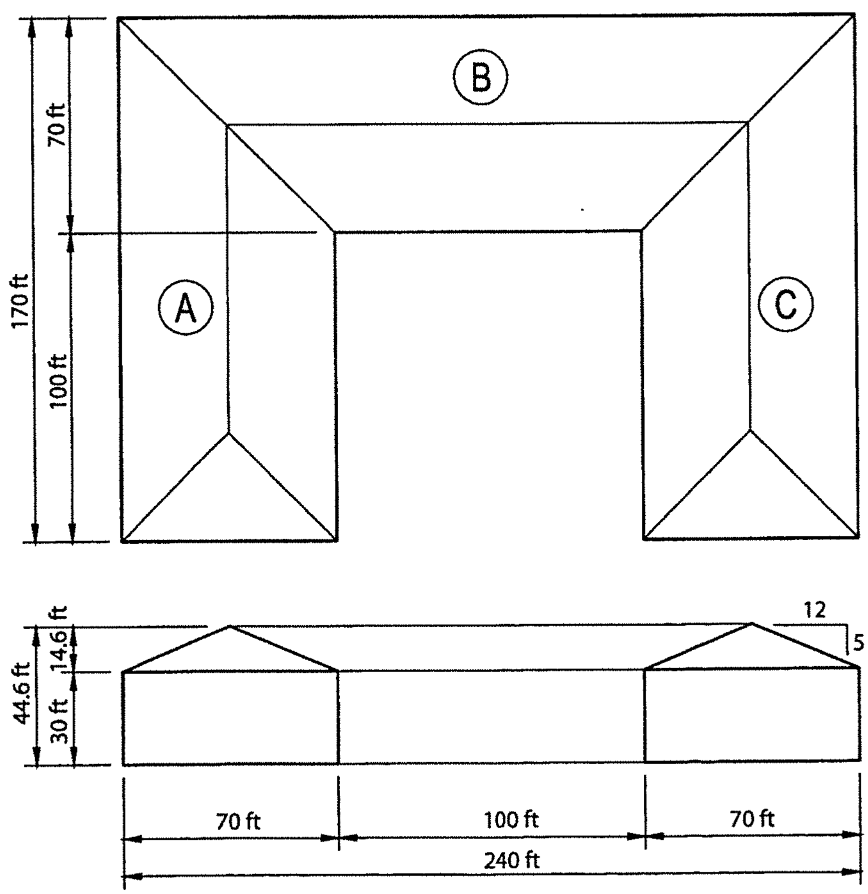


Figure 3-33 240-ft × 170-ft U-shaped Apartment Building

buildings are not specifically covered, the adaptation of the low-rise "pseudo pressure" coefficients to buildings outside the scope of the research is not recommended. Therefore, use the "all heights method" of Section 6.5.12.2.1 of the Standard.

Exposure

The building is located in a suburban area; according to Section 6.5.6.3 of the Standard, Exposure B is used.

Building Classification

The building function is residential. It is not considered an essential facility, nor is it likely to be occupied by 300 persons in a single area at one time. Therefore, building Category II is appropriate (see Table 1-1 of the Standard).

Enclosure

The building is designed to be enclosed. It is not located within a wind-borne debris region, so glazing protection is not required.

Basic Wind Speed

Selection of the basic wind speed is addressed in Section 6.5.4 of the Standard. Birmingham, Alabama, is located just inside the 90-mph contour; therefore, the basic wind speed $V = 90$ mph (see Figure 6-1b of the Standard).

Velocity Pressures (Table 3-42)

The velocity pressures are computed using the following equation:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I \text{ psf} \qquad \text{(Eq. 6-15)}$$

where

K_z = Value obtained from Table 6-3 of the Standard: Case 1 for C&C and Case 2 for MWFRS
K_{zt} = 1.0 for homogeneous topography
K_d = 0.85 for buildings (see Table 6-4 of the Standard)
V = 90 mph
I = 1.0 for Category II classification (see Table 6-1 of the Standard)

$$q_z = 0.00256 K_z (1.0)(0.85)(90)^2 (1.0)$$
$$= 17.63 K_z \text{ psf}$$

Values for K_z and the resulting velocity pressures are given in Table 3-42 below. The mean roof height is the average of the eave and the peak:

$$h = (30 + 44.6) / 2 = 37.3 \text{ ft}$$

At the mean roof height, $h = 37.3$ ft; the velocity pressure is $q_h = 13.2$ psf.

Table 3-42 q_z Velocity Pressures

	MWFRS		*C&C*	
Height (ft)	K_z	q_z *(psf)*	K_z	q_z *(psf)*
0–15	0.57	10.1	0.70	12.3
20	0.62	10.9	0.70	12.3
30	0.70	12.3	0.70	12.3
Mean roof ht = 37.3	0.75	13.2	0.75	13.2

External Pressure Coefficients (C_p)

The values for the external pressure coefficients for the various surfaces (Tables 3-43 through 3-46) are obtained from Figure 6-6 of the Standard for each of the surfaces in Figure 3-34. The determination of certain pressure coefficients is based on aspect ratios. Even though this U-shaped building will be broken into pieces for the application of pressures, the overall dimensions have greater influence on the MWFRS pressure coefficients than the dimensions of the individual pieces. Therefore, the overall dimensions L and B are used.

When the wind is normal to wall W2, the wind blows over the "A" wing, crosses the courtyard in the middle of the U, and strikes the "C" wing. Although some reduction in the pressures on the "C" wind may occur due to the shielding offered by "A," it is impossible to predict without a wind tunnel study. Therefore, the pressures on the "C" wind are taken as the same as on the "A" wing.

For wind normal to surface W2 or W4:

$$L/B = 240/170 = 1.41$$

$$h/L = 37.3/240 = 0.16$$

$$\theta = 22.6° \text{ for a 5-in-12 slope}$$

For wind normal to surface W3 or W1-W7-W5:

$$L/B = 170/240 = 0.71$$

$$h/L = 37.3/170 = 0.22$$

$$\theta = 22.6° \text{ for a 5-in-12 slope}$$

The windward wall C_p is always 0.8, the side walls are –0.7, and the leeward wall varies with the aspect ratio L/B.

Table 3-43 External Pressure Coefficients (C_p) for Wind Normal to Wall W2

Surface type	*Surface designation*	*Surface*	*Case*	*L/B or h/L*	C_p
Walls	W2, W6	Windward		All	+0.80
	W4, W8	Leeward		1.41	–0.42
	W1, W3, W5, W7	Side		All	–0.70
Roofs (⊥ to ridge)	A1, C2	Windward	Negative	0.16	–0.25
			Positive	0.16	+0.25
	A2, C1	Leeward		0.16	–0.60
Roofs (‖ to ridge)	A3, C3	Side	0 to h	0.16	–0.90*
			h to $2h$	0.16	–0.50*
	B1, B2	Side	0 to h	0.16	–0.90*
			h to $2h$	0.16	–0.50*
			$> 2h$	0.16	–0.30*

*The values of smaller uplift pressures (C_p = –0.18) on the roof can become critical when wind load is combined with roof live load or snow load; load combinations are given in Sections 2.3 and 2.4 of the Standard. For brevity, loading for this value is not shown here.

Table 3-44 External Pressure Coefficients (C_p) for Wind Normal to Wall W4

Surface type	*Surface designation*	*Surface*	*Case*	*L/B or h/L*	C_p
Walls	W4, W8	Windward		All	+0.80
	W6, W2	Leeward		1.41	–0.42
	W1, W3, W5, W7	Side		All	–0.70
Roofs (⊥ to ridge)	C1, A2	Windward	Negative	0.16	–0.25
			Positive	0.16	+0.25
	C2, A1	Leeward		0.16	–0.60
Roofs (|| to ridge)	A3, C3	Side	0 to h	0.16	–0.90
			h to $2h$	0.16	–0.50
	B1, B2	Side	0 to h	0.16	–0.90
			h to $2h$	0.16	–0.50
			$> 2h$	0.16	–0.30

Table 3-45 External Pressure Coefficients (C_p) for Wind Normal to Wall W3

Surface type	*Surface designation*	*Surface*	*Case*	*L/B or h/L*	C_p
Walls	W3	Windward		All	+0.80
	W1, W7, W5	Leeward		0.71	–0.50
	W2, W4, W6, W8	Side		All	–0.70
Roofs (⊥ to ridge)	B1	Windward	Negative	0.22	–0.25
			Positive	0.22	+0.25
	A3, B2, C3	Leeward		0.22	–0.60
Roofs (|| to ridge)	A1, A2,C1,C2	Side	0 to h	0.22	–0.90
			h to $2h$	0.22	–0.50
			$> 2h$	0.22	–0.30

Table 3-46 External Pressure Coefficients (C_p) for Wind Normal to Wall W1-W7-W5

Surface type	*Surface designation*	*Surface*	*Case*	*L/B or h/L*	C_p
Walls	W1, W7, W5	Windward		All	+0.80
	W3	Leeward		0.71	–0.50
	W2, W4, W6, W8	Side		All	–0.70
Roofs (⊥ to ridge)	A3, B2, C3	Windward	Negative	0.22	–0.25
			Positive	0.22	+0.25
	B1	Leeward		0.22	–0.60
Roofs (|| to ridge)	A1, A2,C1,C2	Side	0 to h	0.22	–0.90
			h to $2h$	0.22	–0.50
			$> 2h$	0.22	–0.30

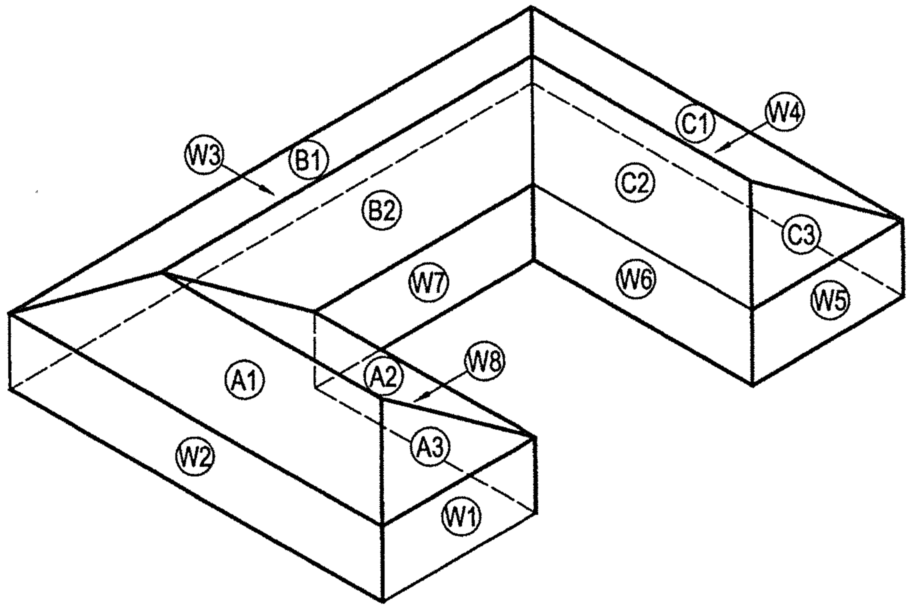


Figure 3-34 Surface Designations

The roof C_ps for wind normal to a ridge vary with roof angle and aspect ratio, h/L. $h/L \leq 0.25$ for all wind directions. The roof angle θ is always 22.6°, so interpolate between 20° and 25°. The C_p for wind parallel to a ridge varies with h/L and with distance from the leading edge of the roof.

Design Wind Pressures for the MWFRS

The design pressures for this building are obtained by the equation

$$p = qGC_p - q_i(GC_{pi}) \qquad \text{(Eq. 6-17)}$$

where

q = q_z for windward wall at height z above ground
q = q_h = 13.2 psf for leeward wall, side walls, and roof
q_i = q_h = 13.2 psf for all surfaces since the building is enclosed
G = 0.85, the gust effect factor for rigid buildings and structures
C_p = External pressure coefficient for each surface as shown in Tables 3-43 through 3-46
(GC_{pi}) = ±0.18, the internal pressure coefficient for enclosed buildings

For windward walls:

$$\begin{aligned} p &= q_zGC_p - q_h(GC_{pi}) \\ &= q_z(0.85)\,C_p - 13.2(\pm 0.18) \\ &= 0.85q_zC_p \pm 2.4 \end{aligned}$$

For all other surfaces:

$$\begin{aligned} p &= q_hGC_p - q_h(GC_{pi}) \\ &= 13.2(0.85)\,C_p - 13.2(\pm 0.18) \\ &= 11.2C_p \pm 2.4 \end{aligned}$$

Design Wind Load Cases

Section 6.5.12.3 of the Standard requires that any building whose wind loads have been determined under the provisions of Sections 6.5.12.2.1 and 6.5.12.2.3 shall be designed for wind load cases as defined in Figure 6-9. Case 1 includes the loadings determined in this example and shown in Tables 3-47 and 3-50. A combination of windward (P_W) and leeward (P_L) loads are applied for Load Cases 2, 3, and 4 as shown in Figure 3-35.

Design Pressures for C&C

Design pressure for C&C is obtained by the following equation:

$$p = q_h[(GC_p) - (GC_{pi})] \qquad \text{(Eq. 6-22)}$$

where

q_h = 13.2 psf for Case 1
(GC_p) = External pressure coefficient (see Figure 6-11 of the Standard)
(GC_{pi}) = ±0.18, the internal pressure coefficient for enclosed buildings

Wall Design Pressures (Table 3-51)

The pressure coefficients (GC_p) are a function of effective wind area. The definition of effective wind area for a C&C panel is the span length multiplied by an effective width that need not be less than one-third the span length (see Section 6.2 of the Standard). The effective wind areas, A, for wall components are as follows:

Window Unit:

$$A = 3(4) = 12 \text{ ft}^2 \text{ (controls)}$$

Table 3-47 External Pressures for Wind Normal to Wall W2

Surface type	*Surface designation*	*z or x (ft)*	*q (psf)*	C_p	*External pressure (psf)*	*Design pressures (psf)* ($+GC_{pi}$)	($-GC_{pi}$)
Walls	W2, W6	0 to 15	10.1	+0.80	+6.9	+4.5	+9.3
		20	10.9	+0.80	+7.4	+5.0	+9.8
		30	12.3	+0.80	+8.4	+6.0	+10.8
	W4, W8	0 to 30	13.2	−0.42	−4.7	−7.1	−2.3
	W1, W3, W5, W7	0 to 30	13.2	−0.70	−7.9	−10.3	−5.5
Roofs (⊥ to ridge)	A1, C2		13.2	−0.25	−2.8	−5.2	−0.4
			13.2	+0.25	+2.8	+0.4	+5.2
	A2, C1		13.2	−0.60	−6.7	−9.1	−4.3
Roofs (‖ to ridge)	A3, C3	0 to 37.3	13.2	−0.90	−10.1	−12.5	−7.7
		37.3 to 70	13.2	−0.50	−5.6	−8.0	−3.2
	B1 & B2	0 to 37.3	13.2	−0.90	−10.1	−12.5	−7.7
		37.3 to 74.6	13.2	−0.50	−5.6	−8.0	−3.2
		74.6 to 240	13.2	−0.30	−3.4	−5.8	−1.0

Note: q_h = 13.2 psf; G = 0.85.

Table 3-48 External Pressures for Wind Normal to Wall W4

Surface type	*Surface designation*	*z or x (ft)*	*q (psf)*	C_p	*External pressure (psf)*	*Design pressures (psf)* $(+GC_{pi})$	*Design pressures (psf)* $(-GC_{pi})$
Walls	W4, W8	0 to 15	10.1	+0.80	+6.9	+4.5	+9.3
		20	10.9	+0.80	+7.4	+5.0	+9.8
		30	12.3	+0.80	+8.4	+6.0	+10.8
	W6, W2	0 to 30	13.2	−0.42	−4.7	−7.1	−2.3
	W1, W3, W5, W7	0 to 30	13.2	−0.70	−7.9	−10.3	−5.5
Roofs (⊥ to ridge)	C1, A2		13.2	−0.25	−2.8	−5.2	−0.4
			13.2	+0.25	+2.8	+0.4	+5.2
	C2, A1		13.2	−0.60	−6.7	−9.1	−4.3
Roofs (|| to ridge)	A3, C3	0 to 37.3	13.2	−0.90	−10.1	−12.5	−7.7
		37.3 to 70	13.2	−0.50	−5.6	−8.0	−3.2
	B1, B2	0 to 37.3	13.2	−0.90	−10.1	−12.5	−7.7
		37.3 to 74.6	13.2	−0.50	−5.6	−8.0	−3.2
		74.6 to 240	13.2	−0.30	−3.4	−5.8	−1.0

Note: $q_h = 13.2$ psf; $G = 0.85$.

Table 3-49 External Pressures for Wind Normal to Wall W3

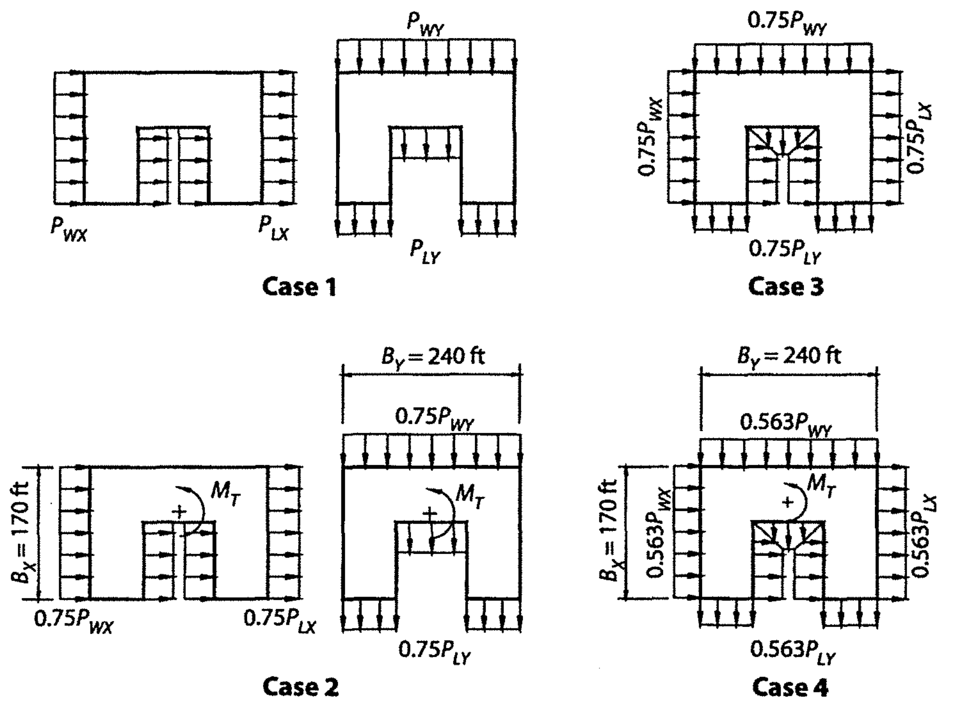
Surface type	*Surface designation*	*z (ft)*	*q (psf)*	C_p	*External pressure (psf)*	*Design pressures (psf)* $(+GC_{pi})$	*Design pressures (psf)* $(-GC_{pi})$
Walls	W3	0 to 15	10.1	+0.80	+6.9	+4.5	+9.3
		20	10.9	+0.80	+7.4	+5.0	+9.8
		30	12.3	+0.80	+8.4	+6.0	+10.8
	W1, W7, W5	0 to 30	13.2	−0.50	−5.6	−8.0	−3.2
	W2, W4, W6, W8	0 to 30	13.2	−0.70	−7.9	−10.3	−5.5
Roofs (⊥ to ridge)	B1		13.2	−0.25	−2.8	−5.2	−0.4
			13.2	+0.25	+2.8	+0.4	+5.2
	A3, B2, C3		13.2	−0.60	−6.7	−9.1	−4.3
Roofs (|| to ridge)	A1, A2, C1, C2	0 to 37.3	13.2	−0.90	−10.1	−12.5	−7.7
		37.3 to 74.6	13.2	−0.50	−5.6	−8.0	−3.2
		74.6 to 170	13.2	−0.30	−3.4	−5.8	−1.0

Note: $q_h = 13.2$ psf; $G = 0.85$.

Table 3-50 External Pressures for Wind Normal to Wall W1-W7-W5

Surface type	*Surface designation*	*z (ft)*	*q (psf)*	C_p	*External pressure (psf)*	*Design pressures (psf)* $(+GC_{pi})$	$(-GC_{pi})$
Walls	W1, W7, W5	0 to 15	10.1	+0.80	+6.9	+4.5	+9.3
		20	10.9	+0.80	+7.4	+5.0	+9.8
		30	12.3	+0.80	+8.4	+6.0	+10.8
	W3	0 to 30	13.2	–0.50	–5.6	–8.0	–3.2
	W2, W4, W6, W8	0 to 30	13.2	–0.70	–7.9	–10.3	–5.5
Roofs (⊥ to ridge)	A3, B2, C3		13.2	–0.25	–2.8	–5.2	–0.4
			13.2	+0.25	+2.8	+0.4	+5.2
	B1		13.2	–0.60	–6.7	–9.1	–4.3
Roofs (|| to ridge)	A1, A2, C1, C2	0 to 37.3	13.2	–0.90	–10.1	–12.5	–7.7
		37.3 to 74.6	13.2	–0.50	–5.6	–8.0	–3.2
		74.6 to 170	13.2	–0.30	–3.4	–5.8	–1.0

Note: $q_h = 13.2$ psf; $G = 0.85$.



$M_T = 0.75(P_{WX} + P_{LX}) \times 170 \times (\pm 25.5\ \text{ft})$
$e_X = \pm 25.5$ ft

$M_T = 0.75(P_{WY} + P_{LY}) \times 240 \times (\pm 36\ \text{ft})$
$e_Y = \pm 36$ ft

$M_T = 0.563(P_{WX} + P_{LX}) \times 170 \times (\pm 25.5\ \text{ft}) + 0.563(P_{WY} + P_{LY}) \times 240 \times (\pm 36\ \text{ft})$
$e_X = \pm 25.5$ ft, $e_Y = \pm 36$ ft

Figure 3-35 Design Wind Load Cases for Wind Normal to Wall W2 and W3

Table 3-51 Wall (GC_p) for Ex. 9

		GC_p		
Component	*A* (ft^2)	*Zones 4 and 5* ($+GC_p$)	*Zone 4* ($-GC_p$)	*Zone 5* ($-GC_p$)
Window	12	+0.99	–1.09	–1.37
Wall Stud	33.3	+0.91	–1.01	–1.22

Table 3-52 Controlling Design Pressures for Wall Components (psf)

	Design pressure (psf)			
	Zone 4		*Zone 5*	
Component	*Positive*	*Negative*	*Positive*	*Negative*
Window unit	+15.4	–16.8	+15.4	–20.5
Mullion	+14.4	–15.7	+14.4	–18.5

Wall Stud:

larger of

$A = 10(1.33) = 13.3 \text{ ft}^2$

or

$A = 10(10/3) = 33.3 \text{ ft}^2$ (controls)

Width of Corner Zone 5:

smaller of

$a = 0.1(170) = 17$ ft

or

$a = 0.1(240) = 24$ ft

or

$a = 0.4(37.3) = 14.9$ ft (controls)

but not less than the smaller of

$a = 0.04(170) = 6.8$ ft

$a = 0.04(240) = 9.6$ ft

and not less than

$a = 3$ ft

Typical Design Pressure Calculations (Table 3-52)

Controlling negative design pressure for window unit in Zone 4 of walls:

$= 13.2[(-1.09) - (\pm 0.18)]$

$= -16.8$ psf (positive internal pressure controls)

Controlling positive design pressure for window unit in Zone 4 of walls:

$= 13.2[(+0.99) - (\pm 0.18)]$

$= 15.4$ psf (negative internal pressure controls)

The design pressures are the algebraic sum of external and internal pressures. Controlling negative pressure is obtained with positive internal pressure, and controlling positive pressure is obtained with negative internal pressure.

The edge zones for the walls are arranged at exterior corners, as shown in Figure 3-36.

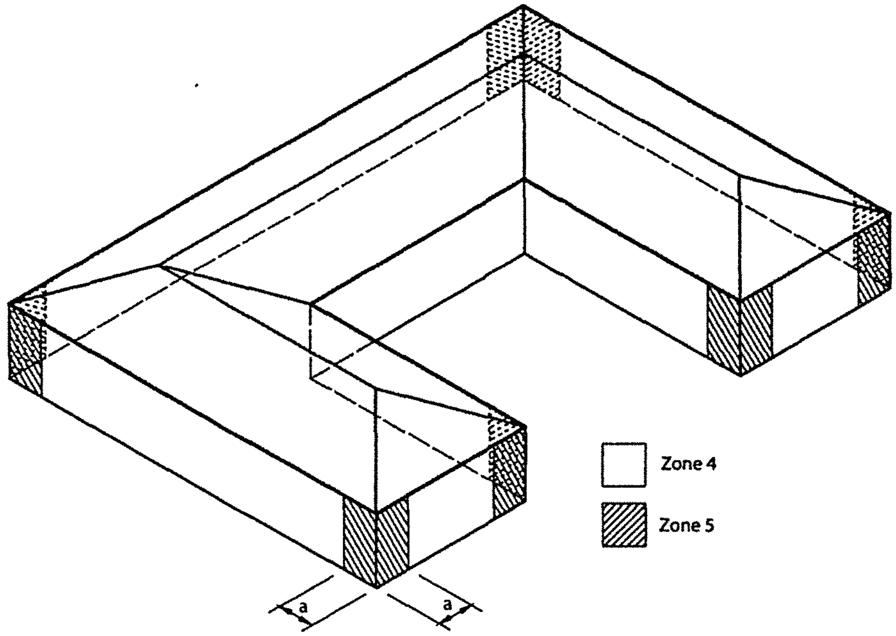


Figure 3-36 Component and Cladding Wall Pressure Zones

Roof Design Pressures (Tables 3-53 and 3-54)

The C&C roof pressure coefficients are given in Figure 6-11 of the Standard. The pressure coefficients are a function of the effective wind area. The definition of effective wind area for a component or cladding panel is the span length multiplied by an effective width that need not be less than one-third the span length (see Section 6.2 of the Standard). The effective wind areas, A, for the roof rafters are as follows:

Roof Rafter:

larger of

$$A = 15(1.33) = 20 \text{ ft}^2$$

or

$$A = 15(15/3) = 75 \text{ ft}^2 \text{ (controls)}$$

Note 7 of Figure 6-11C of the Standard says that for hip roofs with $\theta \leq 25°$, Zone 3 may be treated as Zone 2.

The design pressures are the algebraic sum of external and internal pressures. Controlling negative pressure is obtained with positive internal pressure, and controlling positive pressure is obtained with negative internal pressure.

The edge zones for the hip roof are arranged as shown in Figure 3-37.

Table 3-53 Roof External Pressure Coefficients (GC_p)

	External pressure coefficient		
	Positive	*Negative*	
	Zones 1, 2	*Zone 1*	*Zone 2*
A (ft^2)	GC_p	GC_p	$-GC_p$
75	+0.32	−0.81	−1.26

Table 3-54 Roof Design Pressures (psf)

	Design pressure (psf)		
Component	*Zones 1, 2 Positive*	*Zone 1 Negative*	*Zone 2 Negative*
Roof rafter	+6.6	−13.1	−19.0

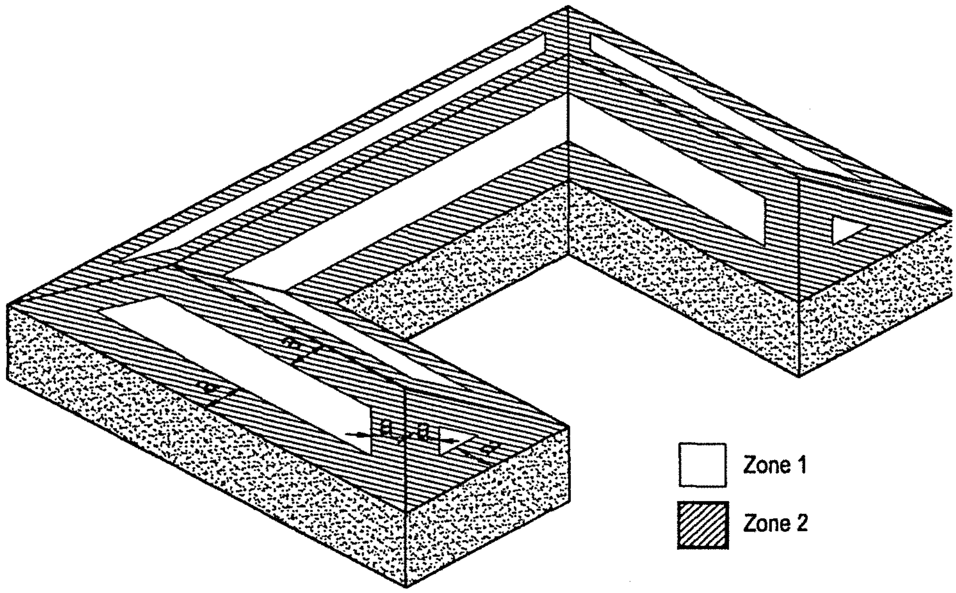


Figure 3-37 Component and Cladding Roof Pressure Zones

3.10 Example 10 50-ft × 20-ft Billboard Sign on Poles (Flexible) 60 ft Above Ground

In this example, design wind-forces for a tall billboard solid sign are determined. The example illustrates two items: (1) determination of G_f for a flexible structure, and (2) use of force coefficient for other structures. The dimensions of the billboard sign are shown in Figure 3-38. The billboard sign data are as follows:

Location:	Interstate highway in Iowa
Terrain:	Flat and open terrain
Dimensions:	50-ft × 20-ft sign mounted on two 16-in.-diameter steel pipe supports; bottom of the sign is 60 ft above ground
Structural characteristics:	Tall flexible structure; estimated fundamental frequency is 0.7 Hz and critical damping ratio is 0.01 *(The natural frequency of a structure can be calculated in different ways. It has been predetermined for this example.)*

Exposure and Building Classification

The sign is located in an open area. It does not fit Exposures B or D; therefore, Exposure C is used (see Sections 6.5.6.2 and 6.5.6.3 of the Standard).

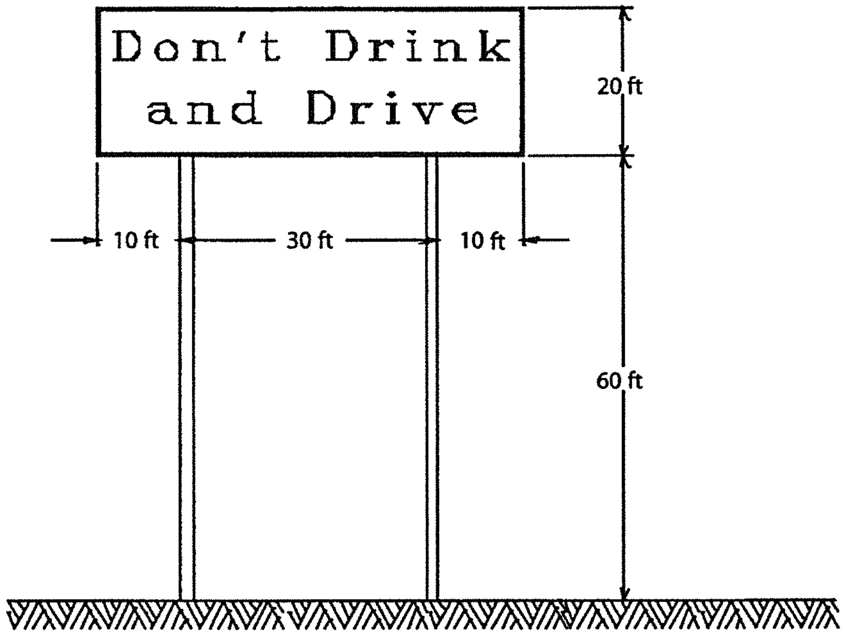


Figure 3-38 Dimensions of a Billboard Sign on an Interstate Highway

Failure of the sign represents low hazard to human life since it is located away from the highway and is not in a populated area. The structure can be classified as Category I (see Table 1-1).

Basic Wind Speed

The wind speed map (Figure 6-1 of the Standard) has only one value of wind speed in the middle of the country. Exact location of the sign in Iowa is not important. The basic wind speed $V = 90$ mph.

Velocity Pressures (Table 3-55)

The velocity pressures are computed using the following equation:

$$q_z = 0.00256\, K_z\, K_{zt}\, K_d\, V^2\, I \text{ psf} \qquad \text{(Eq. 6-15)}$$

where

V = 90 mph
I = 0.87 for Category I (see Table 6-1 of the Standard)
K_{zt} = 1.0 because of flat terrain
K_d = 0.85 for solid sign (see Table 6-4 of the Standard)
K_z = Values from Table 6-3 of the Standard for z of 30, 60, and 80 ft. More divisions of z are not justified because loads on pipe supports are small compared to the ones on the sign.

Table 3-55 Velocity Pressures (psf)

Height (ft)	K_z	q_z *(psf)*
30	0.98	15.0
60	1.13	17.3
80	1.21	18.6

Design Force for MWFRS

The design force for the MWFRS is given by

$$F = q_z G C_f A_f \qquad \text{(Eq. 6-25)}$$

where

q_z = Value as determined previously
G = Gust effect factor to be calculated by Eq. 6-4 because f < 1 Hz.
A_f = $50 \times 20 = 1{,}000$ ft^2; for normal and oblique wind, see Note 4 in Figure 6-20 of the Standard
C_f = Force coefficient values from Figures 6-19 and 6-20 of the Standard

Force Coefficient (C_f)

This sign qualifies as an above ground-level sign (see Figure 6-20 of the Standard):

$$M/N = 2.5$$

$$C_f = 1.2$$

The supports are round. From Figure 6-19 of the Standard:

$$D\sqrt{q_z} = 1.33\sqrt{15.0} = 5.2 > 2.5$$

and

$$h/D = 60/1.33 = 45$$

For moderately smooth surface:

$$C_f = 0.7$$

Gust Effect Factor (G)

The gust effect factor, G, is determined from Eq. 6-8:

$$G = 0.925\left[\frac{1+1.7I_{\bar{z}}\sqrt{g_Q^2Q^2+g_R^2R^2}}{1+1.7g_V I_{\bar{z}}}\right]$$

where

$I_{\bar{z}}$ = Value from Eq. 6-5 of the Standard
g_Q, g_V = Value taken as 3.4 (see Section 6.5.8.2 of the Standard)
g_R = Value from Eq. 6-9 of the Standard
Q = Value determined from Eq. 6-6 of the Standard
R = Value determined from Eq. 6-1 of the Standard
$\bar{z}$ = Equivalent height of the structure, it is used to determine nominal value of $I_{\bar{z}}$; for buildings, the recommended value is $0.6h$, but for the sign, it is the middle of the billboard area or 70 ft
c, l, $\bar{\epsilon}$ = Value given in Table 6-2 of the Standard

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.2\left(\frac{33}{70}\right)^{1/6} = 0.176 \qquad \text{(Eq. 6-5)}$$

$$L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500\left(\frac{70}{33}\right)^{1/5} = 581 \text{ ft} \qquad \text{(Eq. 6-7)}$$

$$Q^2 = \frac{1}{1+0.63\left[\dfrac{B+h}{L_{\bar{z}}}\right]^{0.63}} \qquad \text{(Eq. 6-6)}$$

$$= \frac{1}{1+0.63\left[\dfrac{50+20}{581}\right]^{0.63}} = 0.858$$

Note: In Eq. 6-6, B and h are the dimensions of the sign.

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 0.65\left(\frac{70}{33}\right)^{1/6.5}(90)\left(\frac{88}{60}\right) = 96.3 \qquad \text{(Eq. 6-14)}$$

Note: V is the basic (3-s gust) wind speed in mph.

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(0.7)(581)}{96.3} = 4.22 \qquad \text{(Eq. 6-12)}$$

Note: n_1 is the fundamental frequency of the structure.

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = 0.0564 \qquad \text{(Eq. 6-11)}$$

For R_h,

$$\eta = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{(4.6)(0.7)(80)}{96.3} = 2.675$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) = 0.3043 \qquad \text{(Eq. 6-13a)}$$

Note: h is taken as 80 ft because resonance response depends on full height.

For R_B (assuming $B = 50$ ft),

$$\eta = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}} = \frac{(4.6)(0.7)(50)}{96.3} = 1.672 \qquad \text{(Eq. 6-13a)}$$

$$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) = 0.4255$$

For R_L (assuming depth $L = 2$ ft),

$$\eta = \frac{15.4\, n_1 L}{\bar{V}_{\bar{z}}} = \frac{(15.4)(0.7)(2)}{96.3} = 0.2239 \qquad \text{(Eq. 6-13a)}$$

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) = 0.8661$$

$$g_R = \sqrt{2\ln(3{,}600n_1)} + \frac{0.577}{\sqrt{2\ln(3{,}600n_1)}} \qquad \text{(Eq. 6-9)}$$

$$g_R = 4.1$$

$$R^2 = \frac{1}{\beta}R_n R_h R_B(0.53+0.47R_L) \qquad \text{(Eq. 6-10)}$$

$$= \frac{1}{0.01}(0.0564)(0.3043)(0.4255)\left[0.53+(0.47)(0.8661)\right]$$

$$R^2 = 0.684$$

$$G_f = 0.925\left[\frac{1+1.7I_{\bar{z}}\sqrt{g_Q^2Q^2+g_R^2R^2}}{1+1.7g_v I_{\bar{z}}}\right] \qquad \text{(Eq. 6-8)}$$

$$= 0.925\left[\frac{1+1.7(0.176)\sqrt{(3.4)^2(0.858)+(4.1)^2(0.684)}}{1+1.7(3.4)(0.176)}\right]$$

$$= 1.093$$

Design Force

Force, $F = q_z G_f C_f A_f$

For one support:

0 to 30 ft	$F = 15.0(1.093)(0.7)(1.33) = 15.3$ plf
30 to 60 ft	$F = 17.3(1.093)(0.7)(1.33) = 17.7$ plf

For two supports:

0 to 30 ft	$F = 30.6$ plf
30 to 60 ft	$F = 35.4$ plf

For a 1-ft horizontal strip of the sign:

$F = 18.6(1.093)(1.2)(50) = 1{,}220$ plf

The force on the sign follows two cases (see Figure 3-39):

1. Force at geometric center
2. Force at 0.2(50) = 10 ft on either side of geometric center (see Note 4, Figure 6-20, of the Standard)

Limitation

In certain circumstances for circular members, across-wind response due to vortex shedding can be critical. The Standard does not provide a procedure to assess across-wind response, but suggests obtaining guidance from recognized literature (see Section 6.5.2 of the Standard).

Force on Components and Cladding

Eq. 6-25 of the Standard is

$$F = q_z G C_f A_f$$

The values of q_z are the same as MWFRS, except the value of $G = 0.85$. The design forces can be determined using appropriate C_f and A_f for each component or cladding panel.

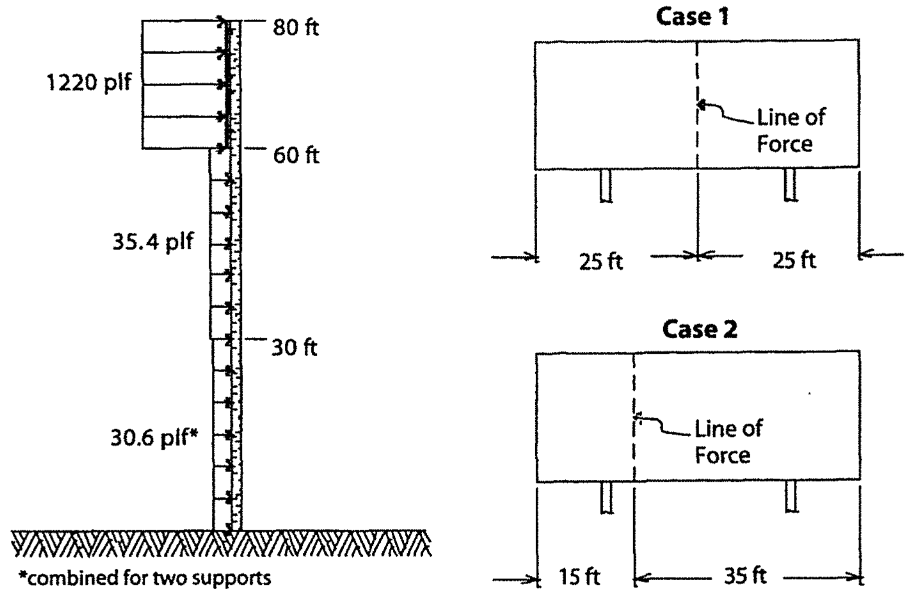


Figure 3-39 Design Forces for the Billboard Sign

3.11 Example 11 Domed Roof Building

Figure 3-40 illustrates the domed roof building used in this example. Building data are as follows:

Location:	Baton Rouge, Louisiana
Topography:	Homogeneous
Terrain:	Open
Dimensions:	100 ft diameter in plan Eave height of 20 ft Dome roof height of 50 ft
Framing:	Steel framed dome roof Metal deck roofing
Cladding:	Location is outside a wind-borne debris region, so no glazing protection is required

Domed roofs are outside the scope of Method 1, Simplified Procedure, of ASCE 7-02. Method 2, Analytical Procedure, is used. The building is less than 60 feet tall, so it is possible to use low-rise provisions of Section 6.5.12.2.2. However, because dome-shaped roofs are not specifically covered, the adaptation of the low-rise "pseudo pressure" coefficients to buildings outside the scope of the research is not recommended. Therefore, the "all heights method" of Section 6.5.12.2.1 of the Standard is used.

Exposure

The building is located in an open terrain area; according to Section 6.5.6.3 of the Standard, Exposure C is used.

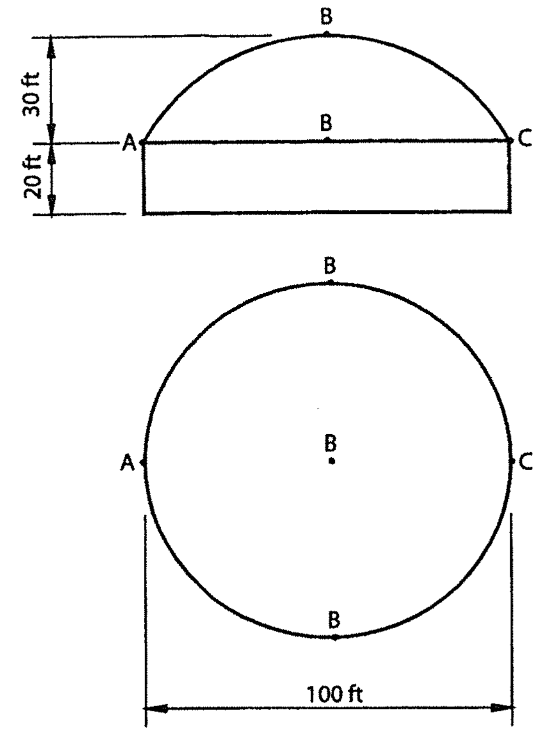


Figure 3-40 100-ft Diameter Domed Roof Building

Building Classification

The building is a church, so it will have more than 300 people congregating in one area. Therefore, building Category III is appropriate (see Table 1-1 of the Standard).

Enclosure

The building is designed to be enclosed. It is not located within a wind-borne debris region, so glazing protection is not required.

Basic Wind Speed

Selection of the basic wind speed is addressed in Section 6.5.4 of the Standard. Baton Rouge, Louisiana, is located halfway between the 100-mph and 110-mph contours; therefore, the basic wind speed V= 105 mph (see Figure 6-1b of the Standards).

Velocity Pressures

The velocity pressures are computed using the following equation:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I \text{ psf} \qquad \text{(Eq. 6-15)}$$

where

K_z = Value obtained from Table 6-3 of the Standard: Case 1 for C&C and Case 2 for MWFRS
K_{zt} = 1.0 for homogeneous topography
K_d = 0.95 for round tanks and similar structures (see Table 6-4 of the Standard)
V = 105 mph
I = 1.15 for Category III classification (see Table 6-1 of the Standard)

Table 3-56 q_z Velocity Pressures

	MWFRS		*C&C*	
Height (ft)	K_z	q_z *(psf)*	K_z	q_z *(psf)*
0–15	0.85	26.2	0.85	26.2
Eave height = 20	0.90	27.7	0.90	27.7
Top of dome = 50	1.09	33.6	1.09	33.6

$$q_z = 0.00256K_z(1.0)(0.95)(105)^2(1.15)$$
$$= 30.8\ K_z \text{ psf}$$

Values for K_z and the resulting velocity pressures are given in Table 3-56. Wall pressures will be evaluated at mid-height = 20 ft/2 = 10 ft / 15 ft; use q_z at 15 ft.

Design Wind Pressures for the MWFRS

Wall Pressures

The walls of a round building are not specifically covered by the Standard. The values for the force coefficients for round tanks and chimneys from Figure 6-19 are used to determine the effect of the wall pressures on the MWFRS. The values of the force coefficients for round tanks vary with the aspect ratio of height to diameter and with the surface roughness.

The value of q_z varies from 26.2 psf at the ground to 27.7 psf at the eave line. Therefore, the ratio $D\sqrt{q_z}$ varies from $100\sqrt{26.2} = 512$ to $100\sqrt{27.7} = 527$, both of which are much greater than 2.5; therefore, the first set of values for C_f for round tanks in Figure 6-19 of the Standard is used. Any projections on the exterior skin of the building are assumed to be less than 2 ft; therefore, D'/D would be less than 2 ft/100 ft = 0.02, so the building is considered moderately smooth. The height of the entire structure (h = 50 ft) is used for the aspect ratio, since the wind has to travel over the dome. Therefore, h/D = 50 ft/100 ft = 0.5, which is less than 1, resulting in C_f = 0.5.

The force on the walls represents the total drag of the wind on the walls of the building, both windward and leeward. Since it is not the typical pressures applied normal to the wall surfaces, ignore internal pressures, as they cancel out in the net drag calculation.

Total drag force on walls = $F = q_z G C_f A_f$ (Eq. 6-25)

where

q_z = q at the centroid of A_f – centroid of A_f is at wall mid-height = 20 ft/2 = 10 ft
q = 26.2 psf (at 10 ft)
G = 0.85, the gust effect factor for rigid structures
C_f = 0.5
A_f = 100 ft × 20 ft = 2,000 sf

Total drag force on walls = F = 26.2(0.85)(0.5)(2,000) = 22,270 lb