Domed Roof Pressures (Table 3-57)

The roof pressure coefficients for a domed roof are taken from Figure 6-7 of the Standard. The height from the ground to the spring line of the dome, h_D = 20 ft. The height of the dome itself from the spring line to the top of the dome, f = 30 ft. Determine C_p for a rise to diameter ratio, f/D = 30/100 = 0.30; and a base height to diameter ratio, h_D/D = 20/100 = 0.20. Interpolation from Figure 6-7 of the Standard is required.

Two load cases are required for the MWFRS loads on domes: Cáses A and B. Case A is based on linear interpolation of C_p values from point A to B and from point B to C (see Figure 3-40 of this guide for the locations of points A, B, and C). Case B uses the pressure coefficient at A for the entire front area of the dome up to an angle θ = 25°, then interpolates the values for the rest of the dome as in Case A.

Case A

For design purposes, interpolate the pressure coefficients at points at 10-ft intervals along the dome. Values of pressure coefficients C_p are shown in Table 3-58.

Case B

Determine the point on the front of the dome at which θ = 25°. The point is 36.2 ft from the center of the dome, therefore 13.8 ft from point A. The pressure coefficient at A shall be used for the section from A to an arc 13.8 ft from A. The remainder of the dome pressures are based on linear interpolation between the 25° point and point B; and then from point B to C. Values of pressure coefficients C_p are shown in Table 3-59.

Internal Pressure Coefficient for Domed Roof

The building is not in a wind-borne debris region, so glazing protection is not required. The building is assumed to be an enclosed building.

Table 3-57 Domed Roof C_p (at f/D = 0.30)

Point on dome	$h_D/D = 0$	$h_D/D = 0.20$	$h_D/D = 0.25$	$h_D/D = 0.50$
A	+0.5	–0.04	–0.18	–
B	–0.78	–0.97	–	–1.26
C	0	–0.20	–	–0.50

Table 3-58 Interpolated Domed Roof C_p (Case A)

Segment	*Start point*	*+10 ft*	*+20 ft*	*+30 ft*	*+40 ft*	*End point*
A to B	–0.04	–0.23	–0.41	–0.60	–0.78	–0.97
B to C	–0.97	–0.82	–0.66	–0.51	–0.35	–0.20

Table 3-59 Interpolated Domed Roof C_p (Case B)

Segment	*Start point*	*+13.8 ft*	*+20 ft*	*+30 ft*	*+40 ft*	*End point*
A to B	–0.04	–0.04	–0.20	–0.46	–0.71	–0.97
Segment	*Start point*	*+10 ft*	*+20 ft*	*+30 ft*	*+40 ft*	*End point*
B to C	–0.97	–0.82	–0.66	–0.51	–0.35	–0.20

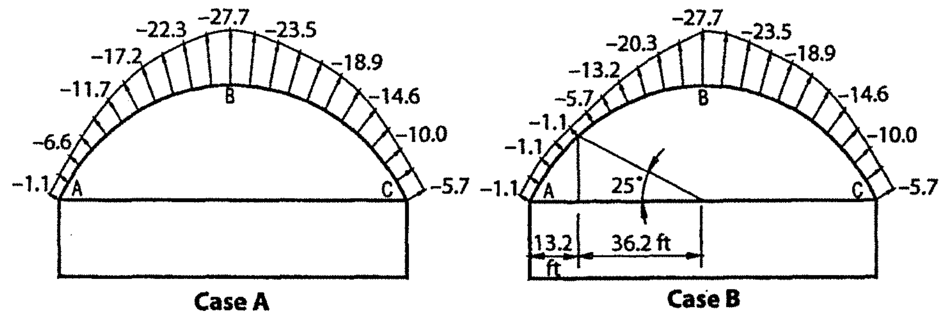


Figure 3-41 MWFRS External Pressures

Note: Internal pressure of ±6.1 psf to be added.

The net pressure on any surface is the difference in the external and internal pressures on the opposites sides of that surface:

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

For enclosed buildings:

$$GC_{pi} = \pm 0.18 \qquad \text{(Figure 6-5)}$$

q_i is taken as $q(_{hD+f}) = 33.6$ psf

Design internal pressure:

$$q_i(GC_{pi}) = 33.6\ (\pm 0.18) = \pm 6.1 \text{ psf}$$

Design Wind Pressures for Domed Roof (Figure 3-41)

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	=	$q_{(h_D+f)} = 33.6$ psf
G	=	0.85, the gust effect factor for rigid buildings and structures
C_p	=	External pressure coefficient
q_i	=	q_h for all surfaces since the building is enclosed
GC_{pi}	=	±0.18, the internal pressure coefficient for enclosed buildings

Design pressure:

$$p = 33.6(0.85)\,C_p - 33.6(\pm 0.18) = 28.6C_p \pm 6.1$$

Values of design pressures for MWFRS are shown in Table 3-60.

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. However, since the building is round, the cases as shown do not apply. There is a possibility of non-symmetrical action by the wind, causing some torsion. Load Case 2, with the reduced calculated horizontal load and moment using eccentricity of 15 ft, could be applied to the cylindrical wall portion of the building.

Table 3-60 Domed Roof Design Pressures for MWFRS (psf)

Surface	Location (ft)	C_p	External pressure (psf)	Design pressures (psf) $(+GC_{pi})$	Design pressures (psf) $(-GC_{pi})$
Domed roof:					
Case A	Point A – 0 ft	–0.04	–1.1	–7.2	5.0
	10	–0.23	–6.6	–12.7	–0.5
	20	–0.41	–11.7	–17.8	–5.6
	30	–0.60	–17.2	–23.3	–11.1
	40	–0.78	–22.3	–28.4	–16.2
	Point B – 50 ft	–0.97	–27.7	–33.8	–21.6
	60	–0.82	–23.5	–29.6	–17.4
	70	–0.66	–18.9	–25.0	–12.8
	80	–0.51	–14.6	–20.7	–8.5
	90	–0.35	–10.0	–16.1	–3.9
	Point C – 100 ft	–0.20	–5.7	–11.8	0.4
Domed roof:					
Case B	Point A – 0 ft	–0.04	–1.1	–7.2	5.0
	θ = 25°; 13.8 ft	–0.04	–1.1	–7.2	5.0
	20	–0.20	–5.7	–11.8	0.4
	30	–0.46	–13.2	–19.3	–7.1
	40	–0.71	–20.3	–26.4	–14.2
	Point B – 50 ft	–0.97	–27.7	–33.8	–21.6
	60	–0.82	–23.5	–29.6	–17.4
	70	–0.66	–18.9	–25.0	–12.8
	80	–0.51	–14.6	–20.7	–8.5
	90	–0.35	–10.0	–16.1	–3.9
	Point C – 100 ft	–0.20	–5.7	–11.8	0.4

Design Pressures for C&C (Figure 3-42)

Design pressure for components and cladding is obtained by the following equation:

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

where

q_h = $q_{(h_D+f)}$ = 33.6 psf for all domed roofs calculated at height $h_D + f$
q_i = $q_{(h_D+f)}$ = 33.6 psf for positive and negative internal pressure
(GC_p) = External pressure coefficient (see Figure 6-16 of the Standard)
(GC_{pi}) = ±0.18 for internal pressure coefficient (see Figure 6-5 of the Standard)

Wall Design Pressures

The Standard does not address component and cladding wall loads for round buildings.

Domed Roof Design Pressures (Table 3-61)

The C&C domed roof pressure coefficients are given in Figure 6-16 of the Standard. This figure is valid only for domes of certain geometric parameters. The base height to diameter ratio, h_D/D = 20/100 = 0.20, which is in the range of 0 to 0.5 for Figure 6-16. The rise to diameter ratio, f/D =

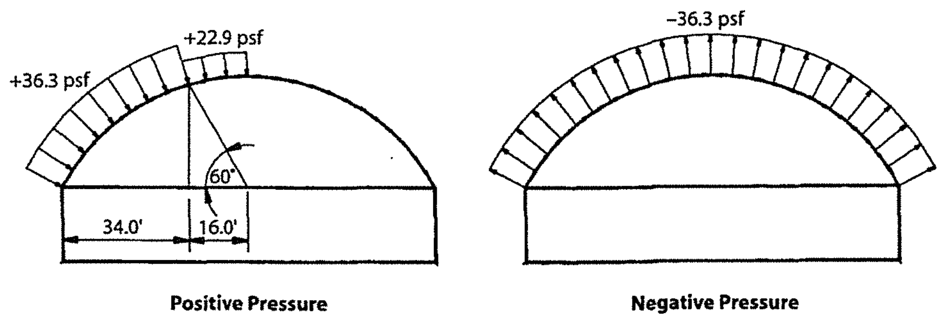


Figure 3-42 Component Design Pressures

Table 3-61 Roof External Pressure Coefficient (GC_p) from Figure 6-16

Zone	*External pressure coefficient (GC_p)* Positive	Negative
0° to 60°	+0.9	–0.9
60° to 90°	+0.5	–0.9

Table 3-62 Roof Design Pressures (psf)

Zone	*Design pressure (psf)* Positive	Negative
0° to 60°	+36.3	–36.3
60° to 90°	+22.9	–36.3

30/100 = 0.30 which is in the range of 0.2 to 0.5 for Figure 6-16. Therefore, it is valid to use Figure 6-16 for this dome.

The design pressures are the algebraic sum of external and internal pressures. Positive internal pressure provides controlling negative pressures, and negative internal pressure provides the controlling positive pressure. These design pressures act across the roof surface (interior to exterior).

$$p = qGC_p - q_i(GC_{pi})$$

$$p = 33.6GC_p - 33.6(\pm 0.18) = 33.6GC_p \pm 6.1$$

Design pressures are summarized in Table 3-62.

These pressures are for the front half of the dome. The back half would experience only the negative value of –36.3 psf. However, since all wind directions must be taken into account, and since each element would at some point be considered to be in the front half of the dome, each element must be designed for both positive and negative values.

3.12 Example 12 Unusually Shaped Building

This example demonstrates calculation of wind loads for an unusually shaped building, as shown in Figure 3-43. Building data are as follows:

Location:	San Francisco, California
Topography:	Homogeneous
Terrain:	Suburban
Dimensions:	100-ft × 100-ft overall in plan with a 70-ft × 70-ft wedge cut off Flat roof with eave height of 15 ft
Framing:	Steel joist, beam, column roof framing with X-bracing
Cladding:	Location is outside a wind-borne debris region, so no glazing protection is required.

Non-symmetrical buildings are outside the scope of Method 1, Simplified Procedure, of ASCE 7-02. Therefore, 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 unusually shaped 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, the "all heights method" of Section 6.5.12.2.1 is used.

Exposure

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

Building Classification

The building is an office building. 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.

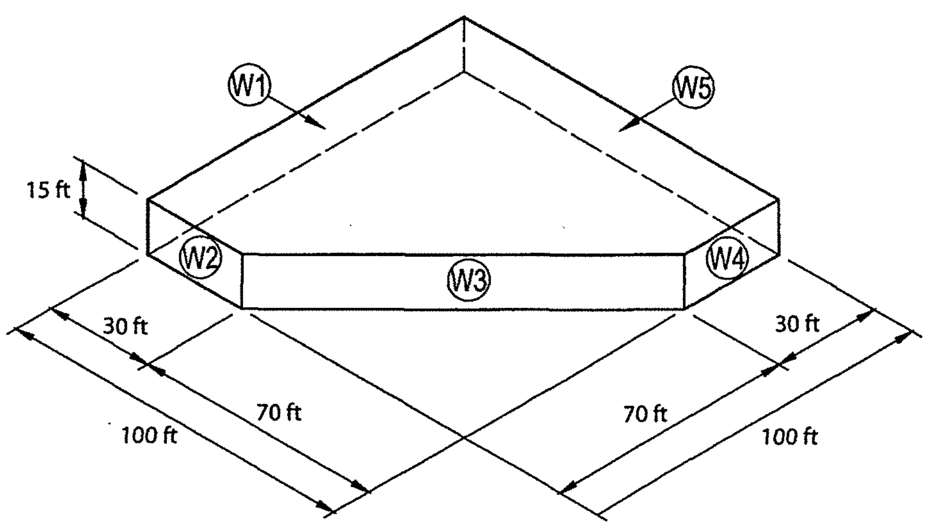


Figure 3-43 100-ft × 100-ft Unusually Shaped Building

Table 3-63 q_z Velocity Pressures

	MWFRS		C&C	
Height (ft)	K_z	q_z *(psf)*	K_z	q_z *(psf)*
0–15	0.57	9.0	0.70	11.0
Eave height = 15	0.57	9.0	0.70	11.0

Basic Wind Speed

Selection of the basic wind speed is addressed in Section 6.5.4 of the Standard. San Francisco, California, is located in the 85-mph zone; therefore, the basic wind speed $V = 85$ mph (see Figure 6-1 of the Standard).

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: 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 = 85 mph
I = 1.0 for Category II classification (see Table 6-1 of the Standard)

$$q_z = 0.00256 K_z (1.0)(0.85)(85)^2(1.0)$$
$$= 15.72 K_z \text{ psf}$$

The mean roof height for a flat roof is the eave height $h = 15$ ft. Values for K_z and the resulting velocity pressures for MWFRS and C&C are shown in Table 3-63.

External Pressure Coefficients (C_p)

The values for the external pressure coefficients for the various surfaces are obtained from Figure 6-6 of the Standard for each of the surfaces of the building shown in Figure 3-43 of this guide. The determination of certain pressure coefficients is based on aspect ratios. The overall dimensions for L and B are used.

$L/B = 100/100 = 1.00$

$h/L = 15/100 = 0.15$

$\theta = 0°$

The windward wall C_p is always 0.8, the side walls are always –0.7, and the leeward wall is –0.5 based on an aspect ratio $L/B = 1.0$.

The roof C_ps come from the "wind parallel to a ridge" portion of Figure 6-6 of the Standard. For these flat roofs, C_p varies with h/L and with distance from the leading edge of the roof. For $h/L = 0.15 \leq 0.5$, $C_p = -0.9, -0.5$, or -0.3, depending on the distance from the leading edge. Figure 6-6 also includes the –0.18 case for all roofs; however, this case causes critical loading when combined with transient loads such as snow load or live load. For brevity, the case is not shown.

External pressure coefficients are summarized in Tables 3-64 through 3-67.

Table 3-64 External Pressure Coefficients (C_p) for Wind Normal to Wall W1

Surface type	Surface designation	Surface	Distance from windward edge	L/B or h/L	C_p
Walls	W1	Windward		All	+0.80
	W3, W4	Leeward		1.0	–0.50
	W2, W5	Side		All	–0.70
Roof			0 to h	0.15	–0.90*
			h to $2h$	0.15	–0.50*
			$> 2h$	0.15	–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 3-65 External Pressure Coefficients (C_p) for Wind Normal to Wall W5

Surface type	Surface designation	Surface	Distance from windward edge	L/B or h/L	C_p
Walls	W5	Windward		All	+0.80
	W2, W3	Leeward		1.0	–0.50
	W1, W4	Side		All	–0.70
Roof			0 to h	0.15	–0.90
			h to $2h$	0.15	–0.50
			$> 2h$	0.15	–0.30

Table 3-66 External Pressure Coefficients (C_p) for Wind Normal to Wall W4

Surface type	Surface designation	Surface	Distance from windward edge	L/B or h/L	C_p
Walls	W4, W3	Windward		All	+0.80
	W1	Leeward		1.0	–0.50
	W2, W5	Side		All	–0.70
Roof			0 to h	0.15	–0.90
			h to $2h$	0.15	–0.50
			$> 2h$	0.15	–0.30

Table 3-67 External Pressure Coefficients (C_p) for Wind Normal to Wall W2

Surface type	Surface designation	Surface	Distance from windward edge	L/B or h/L	C_p
Walls	W2, W3	Windward		All	+0.80
	W5	Leeward		1.0	–0.50
	W1, W4	Side		All	–0.70
Roof			0 to h	0.15	–0.90
			h to $2h$	0.15	–0.50
			$> 2h$	0.15	–0.30

Design Wind Pressures for the MWFRS

The design pressures for this building are obtained by the equation

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

where

q = q_z = 9.0 for windward wall at height z = 15 ft and below
q = q_h = 9.0 psf for leeward wall, side walls, and roof
q_i = q_h = 9.0 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 Figure 3-43 of this guide
(GC_{pi}) = ±0.18, the internal pressure coefficient for enclosed buildings

For windward walls:

$$p = q_zGC_p - q_h(GC_{pi}) = 9.0(0.85)C_p - 9.0(\pm 0.18) = 7.7C_p \pm 1.6$$

For all other surfaces:

$$p = q_hGC_p - q_h(GC_{pi}) = 9.0(0.85)C_p - 9.0(\pm 0.18) = 7.7C_p \pm 1.6$$

Design pressures are summarized in Tables 3-68 through 3-71.

The external roof pressures and their prescribed zones are shown in Figure 3-44.

Table 3-68 Design Pressures for Wind Normal to Wall W1

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	W1	0 to 15	9.0	+0.80	+6.2	+4.6	+7.8
	W3, W4	0 to 15	9.0	–0.50	–3.9	–5.5	–2.3
	W2, W5	0 to 15	9.0	–0.70	–5.4	–7.0	–3.8
Roof		0 to 15	9.0	–0.90	–6.9	–8.5	–5.3
		15 to 30	9.0	–0.50	–3.9	–5.5	–2.3
		30 to 100	9.0	–0.30	–2.3	–3.9	–0.7

Note: q_h = 9.0 psf; G = 0.85.

Table 3-69 Design Pressures for Wind Normal to Wall W5

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	W5	0 to 15	9.0	+0.80	+6.2	+4.6	+7.8
	W2, W3	0 to 15	9.0	–0.50	–3.9	–5.5	–2.3
	W1, W4	0 to 15	9.0	–0.70	–5.4	–7.0	–3.8
Roof		0 to 15	9.0	–0.90	–6.9	–8.5	–5.3
		15 to 30	9.0	–0.50	–3.9	–5.5	–2.3
		30 to 100	9.0	–0.30	–2.3	–3.9	–0.7

Note: q_h = 9.0 psf; G = 0.85.

Table 3-70 Design 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, W3	0 to 15	9.0	+0.80	+6.2	+4.6	+7.8
	W1	0 to 15	9.0	–0.50	–3.9	–5.5	–2.3
	W2, W5	0 to 15	9.0	–0.70	–5.4	–7.0	–3.8
Roof		0 to 15	9.0	–0.90	–6.9	–8.5	–5.3
		15 to 30	9.0	–0.50	–3.9	–5.5	–2.3
		30 to 100	9.0	–0.30	–2.3	–3.9	–0.7

Note: q_h = 9.0 psf; G = 0.85.

Table 3-71 Design 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)* $(+GC_{pi})$	*Design pressures (psf)* $(-GC_{pi})$
Walls	W2, W3	0 to 15	9.0	+0.80	+6.2	+4.6	+7.8
	W5	0 to 15	9.0	–0.50	–3.9	–5.5	–2.3
	W1, W4	0 to 15	9.0	–0.70	–5.4	–7.0	–3.8
Roof		0 to 15	9.0	–0.90	–6.9	–8.5	–5.3
		15 to 30	9.0	–0.50	–3.9	–5.5	–2.3
		30 to 100	9.0	–0.30	–2.3	–3.9	–0.7

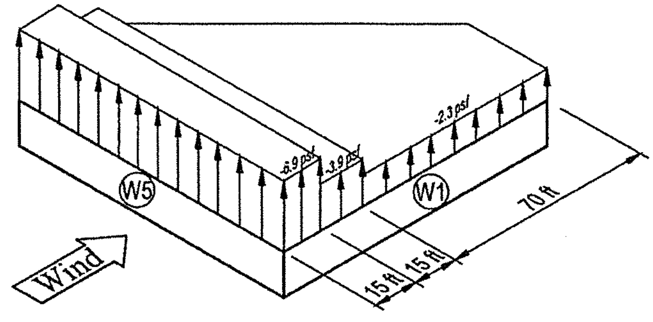
Note: q_h = 9.0 psf; G = 0.85.

Minimum Design Wind Pressures

Section 6.1.4.1 of the Standard requires that the MWFRS be designed for not less than 10 psf applied to the projection of the building in each orthogonal direction on a vertical plane. This is checked as a separate load case. The application of this load is shown in Figure 3-45.

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. There are several exceptions noted that require only the use of Load Case 1, the full orthogonal wind case, and Load Case 3, the diagonal wind case approximated by applying 75% of the loads to adjacent faces simultaneously. The exceptions are building types that are not sensitive to torsional wind effects, which are created by Load Cases 2 and 4. One of these exceptions is for one-story buildings less than 30 ft in height, so this example meets that exception and is required only to meet Load Cases 1 and 3. Load Case 1 is calculated above and shown applied in each orthogonal direction in Figure 3-46. Load Case 3 is the diagonal wind load case, applied in each of four directions as shown in Figure 3-47.



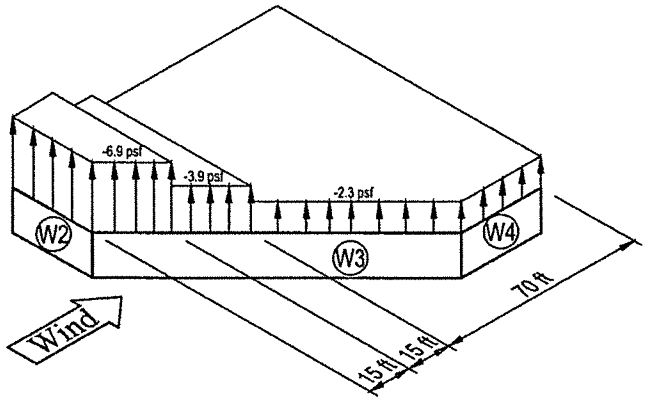
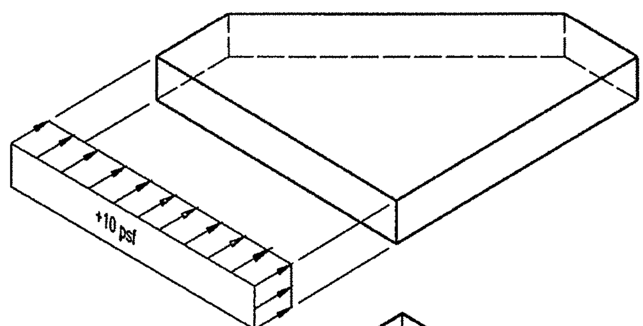


Figure 3-44 External Roof Pressure Zones for MWFRS



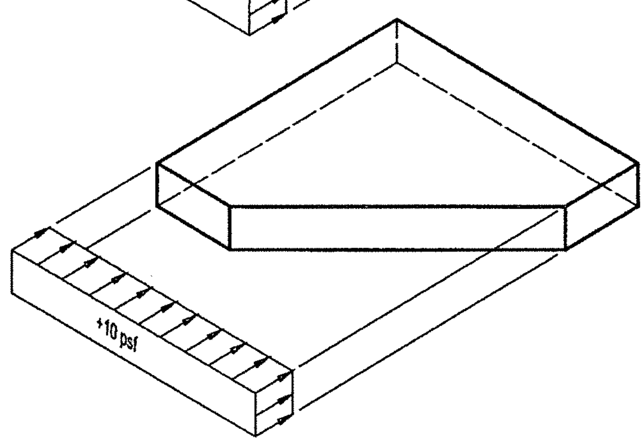


Figure 3-45 Application of 10-psf Minimum Load Case

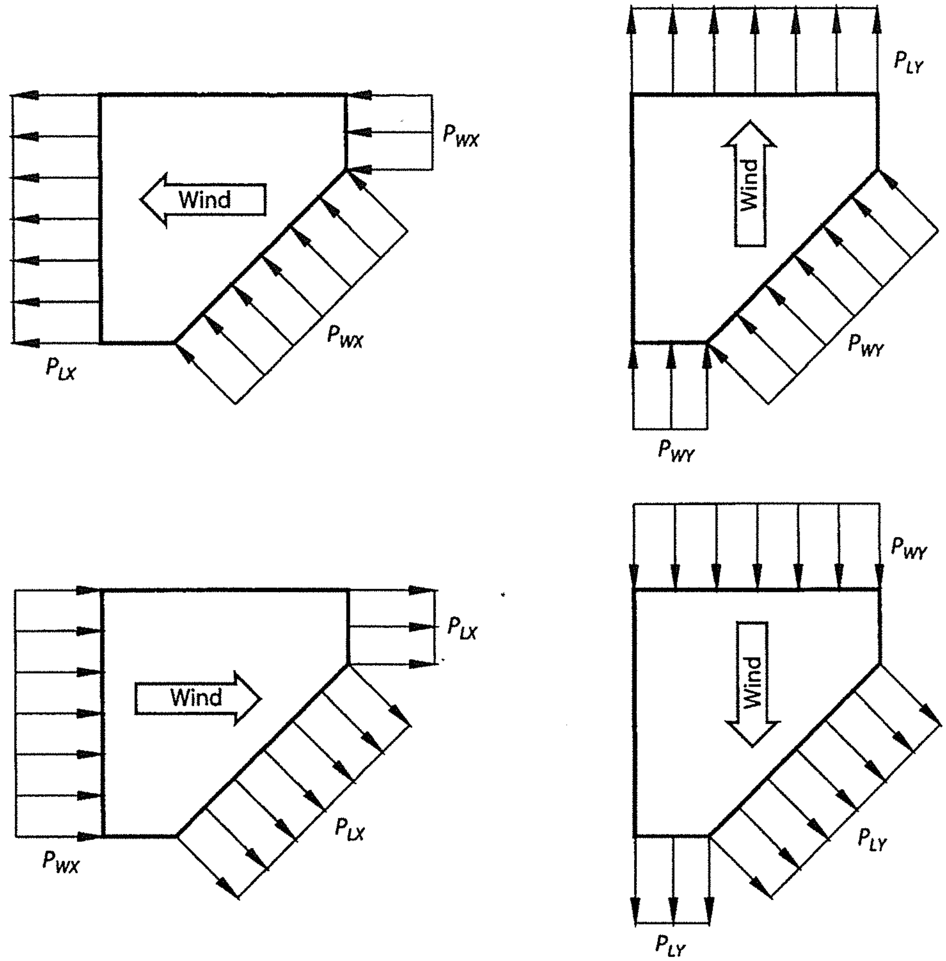


Figure 3-46 Application of Load Case 1 from Each Orthogonal Direction

Design Pressures for Components and Cladding

Design pressure for components and cladding is obtained by the following equation:

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

where

q_h = 11.0 psf for Case 1

(GC_p) = External pressure coefficient (see Figures 6-11A and 6-11B of the Standard)

(GC_{pi}) = ±0.18, the internal pressure coefficient for enclosed buildings

Wall Design Pressures

The pressure coefficients (GC_p) are a function of effective wind area. Since specific components of the walls are not identified, pressure coefficients are given for various effective wind areas in Table 3-72. These values have been reduced by 10% as allowed by Note 5 in Figure 6-11A for roof angle $\theta \leq 10°$.

Width edge zone:

smaller of

$a = 0.1(100) = 10$ ft

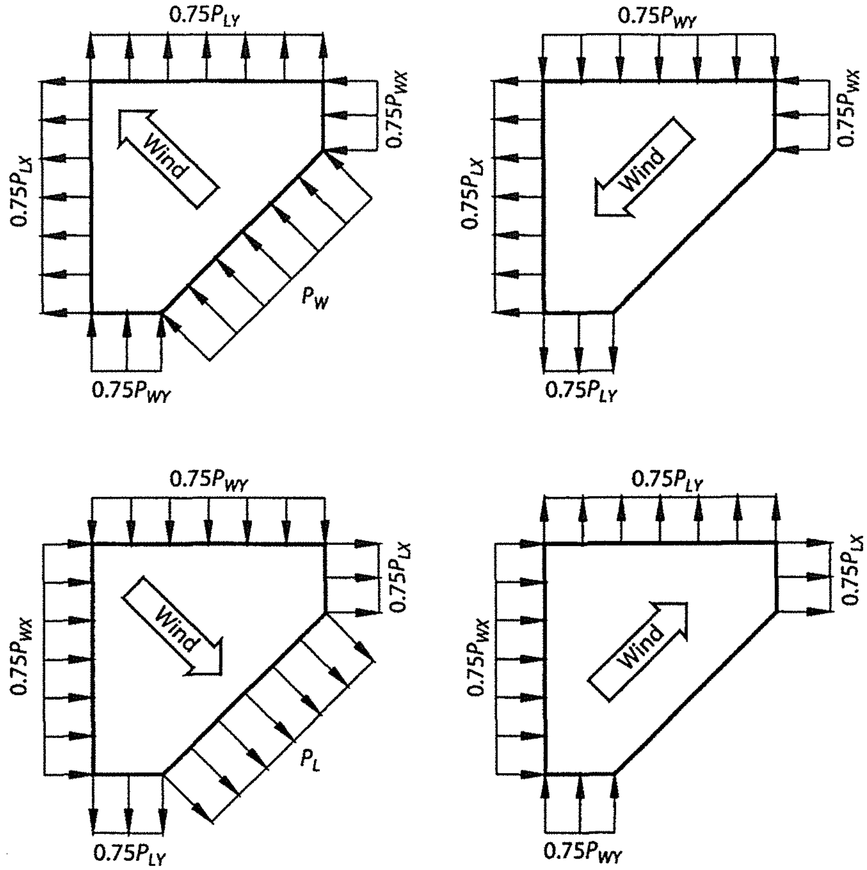


Figure 3-47 Application of Load Case 3 from Each Diagonal Direction

Table 3-72 Wall (GC_p) for Ex.12

	GC_p		
A (ft^2)	*Zones 4 and 5* ($+GC_p$)	*Zone 4* ($-GC_p$)	*Zone 5* ($-GC_p$)
≤10	+0.90	–0.99	–1.26
50	+0.79	–0.88	–1.04
100	+0.74	–0.83	–0.95
>500	+0.63	–0.72	–0.72

Note: GC_p values have been reduced by 10% since $\theta \leq 10°$.

or

$a = 0.4(15) = 6.0$ ft (controls)

but not less than

$a = 0.04(100) = 4.0$ ft

or

$a = 3$ ft

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 controlling design pressures are given in Table 3-73.

Table 3-73 Controlling Design Pressures for Wall Components (psf)

	Design pressure (psf)			
	Zone 4		*Zone 5*	
Area	*Positive*	*Negative*	*Positive*	*Negative*
≤ 10	+11.9	−12.9	+11.9	−15.9
50	+10.7	−11.6	+10.7	−13.4
100	+10.1	−11.1	+10.1	−12.4
> 500	+8.9*	−9.9*	+8.9*	−9.9*

*Section 6.1.4.2 of the Standard requires that C&C pressures be not less than ± 10 psf.

Table 3-74 Roof External Pressure Coefficients (GC_p)

	External pressure coefficient			
	Positive	*Negative*		
	Zones 1, 2, 3	*Zone 1*	*Zone 2*	*Zone 3*
A (ft^2)	GC_p	GC_p	$-GC_p$	$-GC_p$
10	+0.30	−1.00	−1.80	−2.80
50	+0.23	−0.93	−1.31	−1.61
100	+0.20	−0.90	−1.10	−1.10

Table 3-75 Roof Design Pressures (psf)

	Design pressure (psf)			
	Zones 1, 2, 3	*Zone 1*	*Zone 2*	*Zone 3*
Area	*Positive*	*Negative*	*Negative*	*Negative*
10	+5.3*	−13.0	−21.8	−32.8
50	+4.5*	−12.2	−16.4	−19.7
100	+4.2*	−11.9	−14.1	−14.1

*Section 6.1.4.2 of the Standard requires that C&C pressures be not less than ±10 psf.

The edge zones for the walls are arranged at exterior corners, as shown in Figure 3-48.

Roof Design Pressures

The pressure coefficients (GC_p) are a function of effective wind area. Since specific components of the roof are not identified, design pressures are given for various effective wind areas in Table 3-74.

The design pressures (Table 3-75) 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 roof are arranged as shown in Figure 3-49.

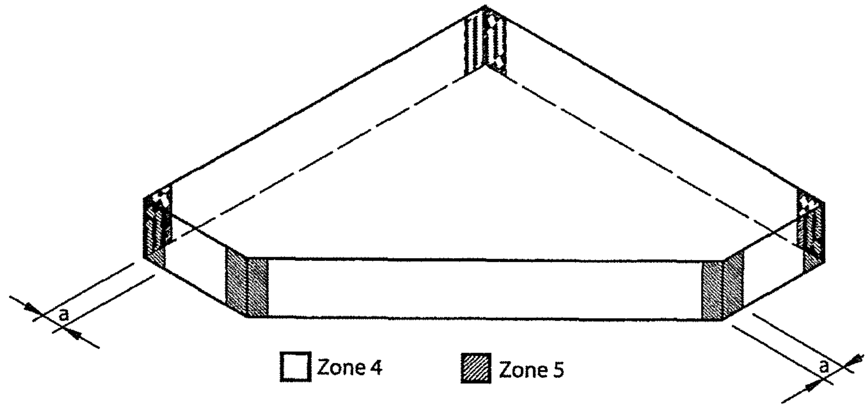


Figure 3-48 Component and Cladding Wall Pressure Zones

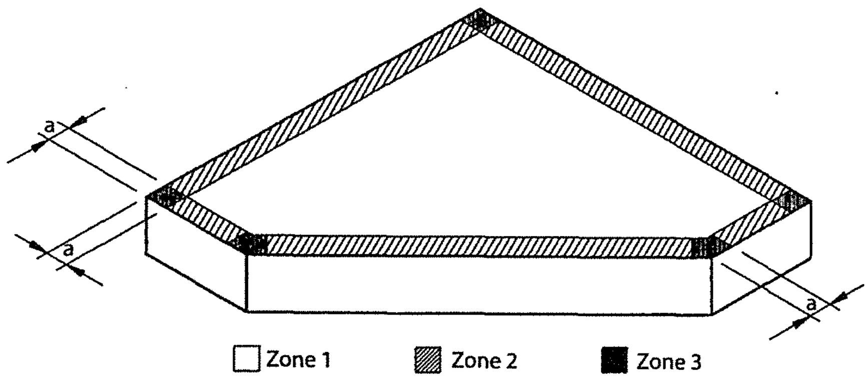
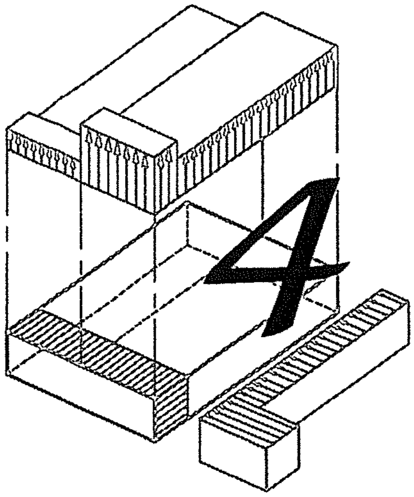


Figure 3-49 Component and Cladding Roof Pressure Zones



4 Frequently Asked Questions

Over the last several years, the authors have fielded hundreds of questions and inquiries from users of the ASCE 7 wind load provisions. The purpose of this chapter is to clarify provisions of the Standard about which questions frequently and repeatedly arise.

1. Is it possible to obtain larger scale maps of basic wind speeds (see Figures 6-1, 6-1a, 6-1b, and 6-1c) so that the locations of the wind speed contours can be determined with greater accuracy?

No. The wind speed contours in the hurricane-prone region of the United States are based on hurricane wind speeds from Monte Carlo simulations and on estimates of the rate at which hurricane wind speeds attenuate to 90 mph following landfall. Because the wind speed contours of these figures represent a consensus of the ASCE 7 Task Committee on Wind Loads, increasing the map scale would do nothing to improve their accuracy.

2. IBC Figure 1609 gives the 3-s wind speed at the project location. However, according to the Notes, Figure 1609 is for Exposure C. If the project location is Exposure B, what is the proper wind speed to use?

Basic wind speed in IBC Figure 1609 or ASCE 7-02 is defined as 3-s gust wind speed at 33 ft above ground for Exposure Category C, which is the standard measurement. The velocity pressure exposure coefficient, K_z, adjusts the wind speed for exposure and height above ground. However, for simplicity the coefficient is applied in the pressure equation, thus adjusting pressure rather than wind speed. Use of K_z adjusts the pressures from Exposure C to Exposure B.

3. If the design wind loads are to be determined for a building that is located in a special wind region (shaded areas) in Figures 6-1, 6-1a, 6-1b, and 6-1c, what basic wind speed should be used?

The purpose of the special wind regions in these figures is to alert the designer to the fact that there are regions in which wind speed anomalies are known to exist. Wind speeds in these regions may be substantially higher than the speeds indicated on the map, and the use of regional climatic data and consultations with a wind engineer or meteorologist are advised.

4. In the design of main wind force-resisting systems (MWFRS), the provisions of Figure 6-6 apply to enclosed or partially enclosed buildings of all heights. The provisions of Figure 6-10 apply to enclosed or partially enclosed buildings with mean roof height less than or equal to 60 ft. Does this mean that either figure may be used for the design of a low-rise MWFRS?

Figure 6-6 may be used for buildings of all heights, whereas Figure 6-10 applies only to low-rise buildings. Section 6.2 defines low-rise buildings to comply with mean roof height $h \leq 60$ ft and h not to exceed least horizontal dimensions. Pressure coefficients for low-rise buildings given in Figure 6-10 represent "pseudo" loading conditions enveloping internal structural reactions of total uplift, total horizontal shear, bending moment, etc. (see Section C6.5.11). Thus, they are not real wind-induced loads. These loads work adequately for buildings of the shapes shown in Figure 6-10; they become questionable when extrapolated to other shapes.

5. Do I consider a tilt-up wall system to be components and cladding (C&C) or MWFRS or both?

Both. Depending on the direction of the wind, a tilt-up wall system must resist either MWFRS forces or C&C forces. In the C&C scenario, the elements receive the wind pressure directly and transfer the forces to the MWFRS in the other direction. When a tilt-up wall acts as a shear wall, it is resisting forces of MWFRS. Because the wind is not expected to blow from both directions at the same time, the MWFRS forces and C&C forces are analyzed independently from each other in two different load cases. This is also true of masonry and reinforced-concrete walls.

6. Section 6.1.4.1 provides for a minimum wind pressure of 10 lb/ft^2 multiplied by the area of the building or structure projected onto a vertical plane normal to the assumed wind direction of MWFRS. Does this provision apply to low-rise buildings?

It should. There was some confusion in ASCE 7-98 provisions for low-rise buildings where it was difficult to interpret application of loads on building frames using the two cases of loads at each corner. In ASCE 7-02, application of loads on low-rise buildings is clarified with illustrative sketches, and only one table of pressure coefficients is provided (See Figure 6-10 of the standard). In addition, Note 6 is added to account for minimum total horizontal shear, although this provision does not guarantee minimum 10 psf on the projected area of the building.

7. A tower has a fundamental frequency of 2 Hz, but has a height-to-width ratio of 6. Should the tower be treated as a flexible structure to determine the gust effect factor?

No. The guideline of height-to-width ratio of 4 or more given in the Commentary is intended to save the user of the Standard the trouble of calculating the fundamental frequency in each and every case. The energy in the turbulence spectrum is very small for frequencies above 1 Hz. Hence, a tower with fundamental frequency of 2 Hz will not be dynamically excited.

8. When can I use the one-third stress increase specified in some material standards?

When using the loads or load combinations specified in ASCE 7-02, no increase in allowable stress is permitted except when the increase is justified by the rate of duration of load (such as duration factors used in wood design). Instead, load combination #6 from Section 2.4.1 of ASCE 7-02 was added for the case when wind load and another transient load are combined. This load combination applies a 0.75 factor to the transient loads ONLY (not to the dead load). The 0.75 factor applied to the transient loads accounts for the fact that it is extremely unlikely that two maximum events will happen at the same time.

9. Why can the wind directionality factor (K_d) only be used with the load combinations specified in Sections 2.3 and 2.4 of ASCE 7-02?

In the strength design load combinations provided in previous editions of ASCE 7 (ASCE 7-95 and earlier), the 1.3 factor for wind included a "wind directionality factor" of 0.85. In ASCE 7-98, the loading combinations used 1.6 instead of 1.3 (approximately equals 1.6×0.85), and the directionality factor is included in the equation for velocity pressure. Separating the directionality factor from the load combinations allows the designer to use specific directionality factors for each structure and allows the factor to be revised more readily when new research becomes available.

10. What exposure category should I use for the MWFRS if the terrain around my site is Exposure B, but there is a large parking lot directly next to one of the elevations?

Section 6.5.6 of ASCE 7-02 provides general definitions of Exposures B, C, and D; however, the designer must refer to the Commentary for a detailed explanation for each exposure. The exposure depends on the size of the parking lot, its size relative to the building, and the number and type of obstructions in the area. Section C6.5.6 of the Commentary includes a formula (Eq. C6-2) that will help the designer determine if the terrain roughness is sufficient to be categorized as Exposure B. Note that, for Exposure B, the fetch distance is 2,630 ft or 10 times the structure's height, whichever is greater. Also note that the Commentary provides suggestions for determining the "upwind fetch surface area."

For clearings such as parking lots, wide roads, road intersections, underdeveloped lots, and tree clearings, the Commentary provides a rational procedure and an example to interpolate between Exposure B and C; the designer is encouraged to use this procedure.

11. What pressure coefficients should be used to reflect contributions for the underside (bottom) of the roof overhangs and balconies?

Sections 6.5.11.4.1 and 6.5.11.4.2 specify pressure coefficients to be used for roof overhangs to determine loads for MWFRS and C&C, respectively. No specific guidance is given for balconies, but use of the loading criteria for roof overhangs should be adequate.

12. If the mean roof height, h, is greater than 60 ft with a roof geometry that is other than flat roof, what pressure coefficients are to be used for roof C&C design loads?

Section 6.5.12.4.3 permits use of pressure coefficients of Figures 6-11 through 6-16 provided the mean roof height $h < 90$ ft, the height-to-width ratio is 1 or less, and Eq. 6-22 is used.

Note 6 of Figure 6-17 permits use of coefficients of Figure 6-11 when the roof angle $\theta > 10°$.

13. Equation 6-15 for velocity pressure uses the subscript z while other equations use subscripts z and h. When is z used and when is h used?

Equation 6-15 is the general formula for the velocity pressure, q_z, at any height, z, above ground. There are many situations in the Standard where a specific value of z is called for, namely the height (or mean roof height) of a building or other structure. Whenever the subscript h is called for, it is understood that z becomes h in the appropriate equations.

14. Under what conditions is it necessary to consider speed-up due to topographic effects when calculating wind loads?

Section 6.5.7 of the Standard requires the calculation of the topographic factor, K_{zt}, for buildings and other structures sited on the upper half of isolated hills or escarpments located in Exposures B, C, or D where the upwind terrain is free of such topographic features for a distance of at least 100 H or 2 mi, whichever is smaller, as measured from the crest of the topographic feature. K_{zt} need not be calculated when the height, H, is less than 15 ft in Exposures D and C, or less than 60 ft in Exposure B. In addition, K_{zt} need not be calculated when H/L_h is less than 0.2. H and L_h are defined in Figure 6-4. The value of K_{zt} is never less than 1.0.

15. What constitutes an open building? If a process plant has a three-story frame with no walls but with a lot of equipment inside the framing, is this an open building?

An open building is a structure in which each wall is at least 80% open (see Section 6.2). Yes, this three-story frame would be classified as an open building, or as "other" structure. In calculating the wind force, F, appropriate values of C_f and A_f would have to be assigned to the frame and to the equipment inside.

16. When is a gable truss in a house part of the MWFRS? Should it also be designed as a C&C? What about individual members of a truss?

Roof trusses are considered to be components since they receive load directly from the cladding. However, since a gable truss receives wind loads from more than one surface, which is part of the definition for MWFRS, an argument can be made that the total load on the truss is more accurately defined by the MWFRS loads. A common approach is to design the members and internal connections of the gable truss for C&C loads, while using the MWFRS loads for the anchorage and reactions. When designing shear walls or cross-bracing, roof loads can be considered an MWFRS.

In the case where the tributary area on any member exceeds 700 ft^2, Section 6.5.12.1.3 permits it to be considered an MWFRS. Even when considered an MWFRS under this provision, the top chord members of a gable truss would have to follow rules of C&C if they receive load directly from the roof sheathing.

17. Flat roof trusses are 30 ft long and are spaced on 4-ft centers. What effective wind area should be used to determine the design pressures for the trusses?

Roof trusses are classified as C&C since they receive wind load directly from the cladding (roof sheathing). In this case, the effective wind area is the span length multiplied by an effective width that need not be less than one-third the span length or (30) (30/3) = 300 ft^2. This is the area on which the selection of GC_p should be based. Note, however, that the resulting wind pressure acts on the tributary area of each truss, which is (30) (4) = 120 ft^2.

18. Roof trusses have a clear span of 70 ft and are spaced 8 ft on center. What effective wind area should be used to determine the design pressures for the trusses?

Following the approach of question #17, above, the effective wind area is (70) (70/3) = 1,633 ft^2. The tributary area of the truss is (70) (8) – 560 ft^2, which is less than the 700-ft^2 area required by Section 6.5.12.1.3 to qualify for design of the truss using the rules for MWFRS. The truss is to be designed using the rules for C&C, and the wind pressure corresponding to an effective wind area of 1,633 ft^2 is to be applied to the tributary area of 560 ft^2.

19. Metal decking consisting of panels 20 ft long and 2 ft wide is supported on purlins spaced 5 ft apart. Will the effective wind area be 40 ft^2 for the determination of pressure coefficients?

Although the length of a decking panel is 20 ft, the basic span is 5 ft. According to the definition of effective wind area, this area is the span length multiplied by an effective width that need not be less than one-third the span length. This gives a minimum effective wind area of (5) (5/3) = 8.3 ft^2. However, the actual width of a panel is 2 ft, making the effective wind area equal to the tributary area of a single panel, or (5) (2) = 10 ft^2. Therefore, GC_p would be determined on the basis of 10 ft^2 of effective wind area, and the corresponding wind load would be applied to a tributary area of 10 ft^2. Note that GC_p is constant for effective wind areas less than 10 ft^2.

20. A masonry wall is 12 ft in height and 80 ft long. It is supported at the top and at the bottom. What effective wind area should be used in determining the design pressure for the wall?

For a given application, the magnitude of the pressure coefficient, GC_p, increases with decreasing effective wind area. Therefore, a very conservative approach would be to consider an effective wind area with a span of 12 ft and a width of 1 ft, and design the wall element as C&C. However, the definition of effective wind area states that this area is the span length multiplied by an effective width that need not be less than one-third the span length. Accordingly, the effective wind area would be (12) (12/3) = 48 ft^2.

21. If a monoslope roof over an open building is virtually flat, what force coefficients from Figure 6-18 should be used?

A requirement for the use of Figure 6-18 is that the wind shall be assumed to deviate plus or minus 10° from the horizontal. Accordingly, the values of C_f corresponding to a roof angle of 10° should be used. The wind forces may be directed either inward or outward, and both cases should be checked.

22. A trussed tower of 10- × 10-ft^2 cross section consists of structural angles forming basic tower panels 10 ft high. The solid area of the face of one tower panel projected on a plane of that face is 22 ft^2. What force coefficient, C_f, should be used to calculate the wind force? What would the force coefficient be for the same tower fabricated of rounded members and having the same projected solid area? What area should be used to obtain the wind force per foot of tower height acting (1) normal to a tower face, and (2) along a tower diagonal?

The gross area of one panel face is (10)(10) = 100 ft^2, and the solidity ratio is $\epsilon = 22/100 = 0.22$. For a tower of square cross section, the force coefficient from Figure 6-22 is as follows:

$$C_f = (4)(0.22)^2 - (5.9)(0.22) + 4.0 = 2.90$$

For rounded members, the force coefficient may be reduced by the factor

$$(0.51)(0.22)^2 + 0.57 = 0.59$$

Thus, the force coefficient for the same tower constructed of rounded members with the same projected area would be

$$C_f = (0.59)(2.90) = 1.71$$

The area, A_f, used to calculate the wind force per foot of tower height is 22/10 = 2.2 ft^2 for all wind directions.

23. In calculating the wind forces acting on a trussed tower of square cross section (see Figure 6-22), should the force coefficient, C_f, be applied to both the front and the back (windward and leeward) faces of the tower?

No. The calculated wind forces are the total forces acting on the tower. The force coefficients given in Figure 6-22 include the contributions of both front and back faces of the tower, as well as the shielding effect of the front face on the back face.

24. If the pressure or force coefficients for various roof shapes (e.g., a canopy) are not given in ASCE 7-02, how can the appropriate wind forces be determined for these shapes?

With the exception of pressure or force coefficients for certain shapes, parameters such as V, I, K_z, K_{zt}, and G are given in ASCE 7-02. It is possible to use pressure or force coefficients from the published literature (see Section 1.4 of this guide) provided these coefficients are used with care. Mean pressure or force coefficients from other sources can be used to determine wind loads for MWFRS. However, it should be recognized that these coefficients might have been obtained in wind tunnels that have smooth, uniform flows as opposed to more proper turbulent boundary-layer flows. Pressure coefficients for components and cladding obtained from the literature should be adjusted to the 3-s gust speed, which is the basic wind speed adopted by ASCE 7-02.

25. Section 6.2 of the Standard provides definitions of glazing, impact resistant; impact-resistant covering; and wind-borne debris regions. To be impact resistant, the Standard specifies that the glazing of the building envelope must be shown by an approved test method to withstand the impact of wind-borne missiles likely to be generated during design winds. Where does one find information on appropriate test methods?

Section 6.7 of the Standard refers to two ASTM standards. These standards give test method and performance criteria of glazing, doors, and shutters when impacted by wind-borne debris.

26. The Standard does not provide for across-wind excitation caused by vortex shedding. How can one determine when vortex shedding might become a problem?

Vortex shedding is almost always present with bluff-shaped cylindrical bodies. Vortex shedding can become a problem when the frequency of shedding is close to or equal to the frequency of the first or second transverse of the structure. The intensity of excitation increases with aspect ratio (height-to-width or length-to-breadth) and decreases with increasing structural damping. Structures with low damping and with aspect ratios of 8 or more may be prone to damaging vortex excitation. If across-wind or torsional excitation appears to be a possibility, expert advice should be obtained.

27. If high winds are accompanied by rain, will the presence of raindrops increase the mean density of the air to the point where the wind loads are affected?

No. Although raindrops will increase the mean density of the air, the increase is small and may be neglected. For example, if the average rate of rainfall is 5 in./h, the average density of raindrops will increase the mean air density by less than 1%.

28. How do I design for a Category 3 hurricane?

The Saffir/Simpson Hurricane Scale classifies hurricanes based on intensity and damage potential using five categories (1 through 5, with 5 being the most intense). No direct correlation can be made between the Saffir/Simpson Hurricane Scale and design wind speeds. The Saffir/Simpson Hurricane Scale categorizes hurricanes based on 1-min sustained wind speeds, which conform to the National Weather Service requirements. These classifications are intended to be used by emergency management personnel. ASCE 7-93 and previous editions are based on the fastest-mile speeds and ASCE 7-95 and subsequent editions use 3-s gust wind speeds. Figure C6-2 of the Standard may be used to convert 1-min wind speed to 3-s gust wind speed; design using Category 3 hurricane wind speed is likely to be higher than basic wind speed in most areas.

29. What wind loads do I use during construction?

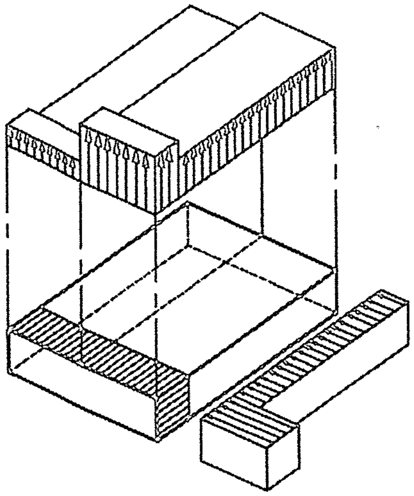
ASCE 7 does not address wind loads during construction. Construction loads are specifically addressed in the standard SEI/ASCE 37-01, *Design Loads on Structures during Construction.*

30. Can the pressure/force coefficients given in ASCE 7-02 be used with the provisions of ASCE 7-88, 7-93, 7-95, or 7-98?

Yes, in a limited way. ASCE 7-88 (and 7-93) used the fastest-mile wind speed as the basic wind speed. With the adoption of the 3-s gust speed starting with ASCE 7-95, the values of certain parameters used in the determination of wind forces have been changed accordingly. The provisions of ASCE 7-88 and 7-02 should not be interchanged. Coefficients in ASCE 7-95, 7-98, and 7-02 are consistent; they are related to 3-s gust speed.

31. Is it possible to determine the wind loads for the design of interior walls?

The Standard does not address the wind loads to be used in the design of interior walls or partitions. A conservative approach would be to apply the internal pressure coefficients $GC_{pi} = \pm 0.18$ for enclosed buildings and $GC_{pi} = \pm 0.55$ for partially enclosed buildings. Post-disaster surveys have revealed the failure of interior walls when the building envelope has been breached.



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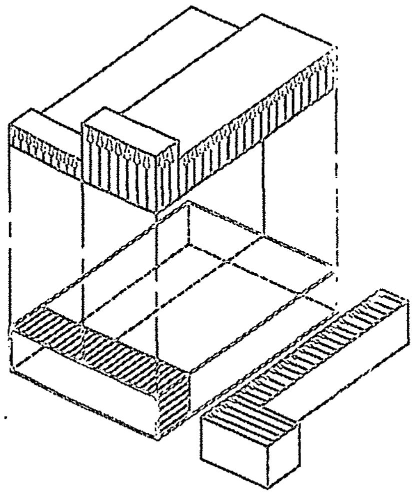

Index