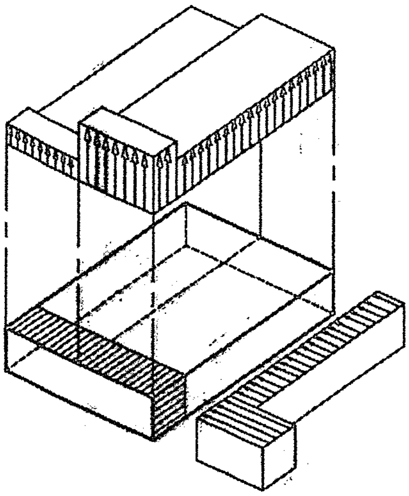
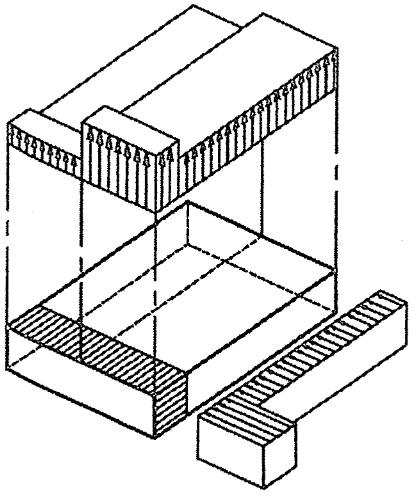


Guide to the Use of the Wind Load Provisions



Guide to the Use of the Wind Load Provisions of ASCE 7-02

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ASCE PRESS

Cataloging-in-publication data on file with the Library of Congress.

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Published by the American Society of Civil Engineers
1801 Alexander Bell Drive
Reston, VA 20191
www.asce.org

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ISBN 0-7844-0703-7
Manufactured in the United States of America.

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Preface

This guide is designed to assist professionals in the use of the wind load provisions of SEI/ASCE Standard 7-02, *Minimum Design Loads for Buildings and Other Structures*, published by the American Society of Civil Engineers (ASCE). The guide is a revision of the *Guide to the Use of Wind Load Provisions of ASCE 7-98*, reflecting the significant changes made to wind load provisions when the previous version of the Standard, ASCE 7-98, was updated. The guide contains 12 example problems worked out in detail, which can provide direction to practicing professionals in assessing wind loads on a variety of buildings and other structures. Every effort has been made to make these illustrative example problems correct and accurate. The authors would welcome comments regarding inaccuracies, errors, or different interpretations. The views expressed and interpretation of the wind load provisions made in the guide are those of the authors and not of the ASCE 7 Standards Committee or the ASCE organization.

Acknowledgments

The authors wish to acknowledge the members of the ASCE 7 Minimum Design Loads for Buildings and Other Structures Standards Committee that was chaired by Dr. Jim Harris during the consensus process of ASCE 7-02. The members of the Task Committee on Wind Loads and the ASCE 7 Standards Committee contributed significantly to the final wind load provisions of the ASCE 7-02 through their questions, comments, and discussions. The authors are indebted to these 100-plus members.

In a document of this type, there are individuals in the background who helped in layout, word-processing, and checking calculations. These tasks were handled by the staff of the Wind Science and Engineering Research Center, Texas Tech University, and the authors acknowledge and appreciate the contributions of these individuals. In particular, we would like to thank Hua He, doctoral degree candidate at Texas Tech University, who performed and checked calculations for example problems and made a significant contribution to this guide; and Kevin Brown, project engineer at LBYD, Inc., who back-checked the calculations for all the example problems. The authors acknowledge and appreciate the contributions of all these individuals.

since it had the potential to establish a single wind load criterion for design of all buildings and structures for the entire United States. A *Guide to the Use of the Wind Load Provisions of ASCE 7-98* (Mehta and Perry, 2000) was published soon after publication of ASCE 7-98.

In 2003, the new standard, ASCE 7-02, was published. This guide is designed to assist practicing professionals in the use of wind load criteria of ASCE 7-02.

1.1 Objective of the Guide

The objective of this guide is to provide direction in the use of wind load provisions of ASCE 7-02 (referred to as "the Standard"). The Commentary of ASCE 7-02 (Section C6.0) contains a good background and discussion of the wind load criteria; that information is not repeated in this document. Rather, this guide contains two important items to assist the users of ASCE 7-02: (1) examples, and (2) Frequently Asked Questions.

The guide contains 12 worked examples. Sufficient details of calculation of wind loads are provided to help the reader properly interpret the wind load provisions of the Standard. Section 6.0 of the Standard, as well as the figures and tables of the Standard, are cited liberally in the examples. **It is necessary to have a copy of ASCE 7-02 to follow the examples and work with this Guide.** A copy of ASCE 7-02 can be ordered by calling 1-800-548-ASCE or ordered on the web at www.pubs.asce.org.

1.2 Significant Changes

The wind load provisions of Section 6.0 were revised in ASCE 7-02 using recent research and development achievements. The major changes involve expansion of the simplified procedure, load cases for main wind force-resisting systems (MWFRS), and introduction of surface roughness length to define exposure coefficients.

The basic approach to assessing wind loading has not been changed, but new parameters, such as surface roughness length, are added to provide more flexibility to designers. In addition, wind-borne debris provisions in hurricane-prone areas are specifically spelled out. Significant changes affecting the design process are listed below.

- Method 1, Simplified Procedure, is significantly revised and expanded. The requirements for using this method have been set separately for MWFRS and components and cladding (C&C). The procedure is expanded to include flat, gable, and hip roofs with roof slope up to 45° MWFRS. For C&C, the hip roof is restricted to $\theta = 27°$. Eq. 6-1 and 6-2 permit modification for terrain and height from tabulated values, as well as for importance factor.
- For design of MWFRS, wind load cases (Figure 6-9) are applied to buildings of all heights.
- Exposure category is based on surface roughness length in each wind direction sector. The surface roughness length parameter permits interpolation of the exposure category using a rational proce-

dure. An assessment of exposure categories using word description, as previously done, is permitted.

- Distances of ground surface roughness condition in the prevailing direction are revised.
- Exposure A is deleted from the tabulated values in the Standard.
- Lower value of roof pressure coefficient is added for all roof angles to obtain minimum design pressures for MWFRS.
- Low-rise building provisions for MWFRS have been significantly revised. The load cases are clearly delineated.
- New pressure coefficients are provided to determine wind loads for domed roof buildings (see Example 11 in Section 3.11 of this guide).
- Provisions for calculating wind load for parapets (MWFRS and C&C) are added to the Standard (see Example 3 in Section 3.3 of this guide).

ASCE 7-02 incorporates the latest available technical information. As noted above, the basic methodology of the Standard remains the same as in ASCE 7-98. Additional information on the changes can be found in the Commentary of the Standard and from references.

1.3 Limitations of Standard

The possible shortcomings or limitations of the Standard are directly dependent on accurate knowledge of parameters and factors used in the algorithms that define the wind loads for design applications. Limitations of some of the significant parameters are discussed below.

1.3.1 Assessment of Wind Climate

The current Standard provides a more realistic description of wind speeds than did the previous editions of the 1970s and 1980s. Perhaps the most serious limitations are that design speeds are not referenced to direction, and potential wind speed anomalies are defined only in terms of special wind regions. These special wind regions include mountain ranges, gorges, or river valleys. Unusual winds may be encountered in these regions because of orographic effects or because of the channeling of wind. The Standard permits climatological studies using regional climatic data and consultation with a wind engineer and/or a meteorologist.

Tornado winds are not included in development of the basic wind speed map (Figure 6-1 of the Standard) because of their relatively rare occurrence at a given location. Intense tornadoes can have ground level wind speeds in the range of 150 to 200 mph; however, the annual probability of exceedance of this range of wind speeds may be less than 1×10^{-5} (mean recurrence interval exceeding 100,000 years). Special structures and storm shelters can be designed to resist tornado winds if required.

1.3.2 Limitations in Evaluating Structural Response

Given that the majority of buildings and other structures can be treated as rigid structures, the gust effect factor specified in the Standard is adequate. For dynamically sensitive buildings and other structures, a gust effect factor, G_f, is given. The formulation of gust effect factor, G_f, is primarily for buildings; it is not always applicable to other structures. It should be noted that the gust effect factor, G_f, is based on along-wind buffeting response.

Vortex shedding is almost always present with bluff-shaped cylindrical bodies. It can become a problem when the frequency of shedding is close to, or equal to, the frequency of the first or second transverse modes 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 an aspect ratio 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.

Another limitation with respect to evaluating structural response is that the Standard does not define acceptable design wind speeds for serviceability states (e.g., deflection, dynamic sway). Table C6-3 in the Commentary provides conversion factors for determining appropriate wind speeds for mean recurrence intervals of 5 to 500 yr.

1.3.3 Limitations in Shapes of Buildings and Other Structures

The pressure and force coefficients given in the Standard are limited. Many of the structural shapes (e.g., "Y," "T," and "L" shapes) or buildings with stepped elevations are not included. Fortunately, this information may be found in other sources (see Table 1-1 of this guide).

When coefficients for a specific shape are not given in the Standard, the designer is encouraged to use values that are available in the literature. However, the use of prudent judgment is advised, and the following caveats must be addressed:

1. Were the coefficients obtained from proper turbulent boundary layer wind tunnel tests (BLWT), or were they generated under conditions of relatively smooth flow?
2. The averaging time used must necessarily be considered in order to determine whether the coefficients are directly applicable to the evaluation of design loads or whether they need to be modified.
3. The reference wind speed (fastest-mile, hourly mean, 10-min mean, 3-s gust, etc.) and exposure category under which the data are generated must be established in order to properly compute the velocity pressure, q.
4. If an envelope approach is used, the coefficients should be appropriate for all wind directions. If, however, a directional approach is indicated, then the applicability of the coefficients as a function of wind direction needs to be ascertained. A major limitation in the use of directional coefficients is that their adequacy for other than normal wind directions may not have been verified.

1.4 Technical Literature

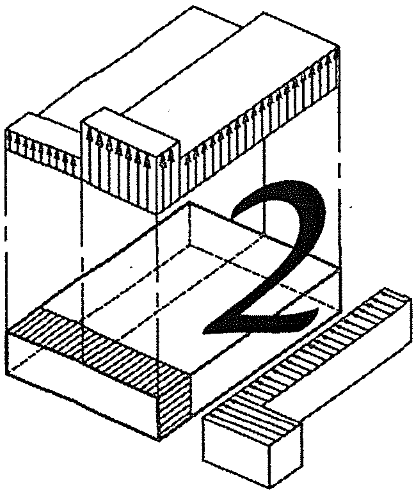
There has been a vast amount of literature published on wind engineering during the past three decades. Most of it is in the form of research papers in the *Journal of Wind Engineering and Industrial Aerodynamics*, *Journal of Structural Engineering*, *Proceedings of the International Conferences on Wind Engineering* (a total of eleven), *Proceedings of the U.S. National Conferences on Wind*

Engineering (a total of nine), *Proceedings of the Asia-Pacific Conferences on Wind Engineering* (a total of five), and *Proceedings of the European-African Conferences on Wind Engineering* (two). The literature is extensive and scholarly; however, it is not always in a format that can be used by practicing professionals.

Several textbooks, handbooks, standards and codes, reports, and papers contain material that can be used to determine wind loads. Selected items are identified in Table 1-1 of this guide. The items are listed by subject matter for easy identification. Detailed references for these items are given in the citations in Chapter 4, References, of this guide.

Table 1-1 Technical Literature

Subject	*Selected reference material (see Chapter 4 of this guide)*
Wind effects on buildings and structures	Newberry and Eaton (1974); Lawson, vols. 1 and 2 (1980); Cook, parts 1 and 2 (1985); Holmes, Melbourne, and Walker (1990); Liu (1991); Simiu and Scanlan (1996); Holmes (2001)
Foreign codes and standards	NRCC (1995a, 1995b); British Standard BS 6399 (1995); Eurocode 1(1994); ISO (1997); Australian/New Zealand Standard AS/NZS 1170.2 (2002)
Wind tunnel testing	Reinhold (1982); ASCE (1999)
General wind research	ASCE (1961); Cermak (1977); Davenport, Surry, and Stathopoulos (1977, 1978); Simiu (1981)
Pressure and force coefficients	ASCE (1961, 1997); Hoerner (1965)
Tornadoes, shelter design	Minor, McDonald, and Mehta (1993); FEMA TR83-A (1980); Minor (1982); McDonald (1983); FEMA 320 (1999); FEMA 361 (2000)
Impact resistance protocol	SBCCI (1999); ASTM E1886-97, ASTM E1996-01; Miami/Dade County Building Code Compliance Office Protocol PA 201-94 and PA 203-94



Wind Load Provisions

2.1 Format

The designer is given three options for evaluating the design wind loads for buildings and other structures:

1. Method 1, Simplified Procedure, as specified in Section 6.4 of the Standard, for buildings meeting certain specific requirements. The requirements are set for main wind force-resisting system (MWFRS) and components and cladding (C&C), respectively.
2. Method 2, Analytical Procedure, of Section 6.5 of the Standard, applicable to buildings and other structures.
3. Method 3, Wind Tunnel Procedure, which meets certain test conditions as specified in Section 6.6 of the Standard.

The simplified and analytical procedures (see Sections 6.4.2 and 6.5.3, respectively) provide specific steps to be followed in the determination of wind loads on MWFRS and C&C separately. MWFRS is defined in Section 6.2 as the overall structure receiving wind loading from more than one surface. Cladding receives wind loads directly and generally transfers the load to other components or to the MWFRS. Equations for the determination of wind loads using the analytical procedures are given in the body of the text of the Standard. Some of the important notes that were given with figures and tables of the previous editions of the Standard have been moved into the body of the text.

Equations for the graphs of Figures 6-11A through 6-17 in the Standard are given in Section 2.4 of this guide because interpolation using these graphs, as presented in the Standard, is difficult.

2.2 Design Procedures

2.2.1 Velocity Pressure

The first step in using Method 2, Analytical Procedure, is to determine the appropriate parameters for evaluating the velocity pressure, q.

Velocity pressure, q, at any height above ground and at mean roof height is obtained by the following equation:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I \text{ (lb/ft}^2\text{)} \quad \text{(Eq. 6-15)}$$

where

- q = Effective velocity pressure to be used in the appropriate equations to evaluate wind pressures for MWFRS and C&C; q_z at any height, z, above ground; q_h is based on K_h at mean roof height, h
- K_z = Exposure velocity pressure coefficient, which reflects change in wind speed with height and terrain roughness (see Section 6.5.5 and Table 6-3 of the Standard)
- K_{zt} = Topographic factor which accounts for wind speed-up over hills and escarpments (see Section 6.5.7 and Figure 6-4 of the Standard)
- K_d = Directionality factor (see Section 6.5.4.4 and Table 6-4 of the Standard)
- V = Basic wind speed, which is the 3-s gust speed at 33 ft above ground for Exposure Category C and is associated with an annual probability of 0.02 (50-yr mean recurrence interval) (see Section 6.5.4 and Figure 6-1 of the Standard)
- I = Importance factor, which adjusts wind speed associated with annual probability of 0.02 (50-yr mean recurrence interval) to other probabilities (25- or 100-yr MRI) (see Section 6.5.5 and Table 6-1 of the Standard)

2.2.2 Method 1, Simplified Procedure

Method 1 was introduced in ASCE 7-98 for simplifying evaluation of design loads for common regular-shaped buildings. In ASCE 7-02, provisions of this method are revised significantly. The restrictions for using the simplified procedure are set for MWFRS and C&C in Sections 6.4.1.1 and 6.4.1.2, respectively.

Tabulated wind pressure values are provided in Figure 6-2 for MWFRS and Figure 6-3 for C&C. For MWFRS, Method 1 combines the windward and leeward pressures into a net horizontal wind pressure on the walls (internal pressures cancel). The maximum uplift on the roof for MWFRS is based on a positive internal pressure as the controlling case and is applied on horizontal projection of the roof surface. For C&C, values are provided only for enclosed buildings and represent the net pressure (sum of external and internal pressures) applied normal to surfaces. The following values have been assumed in the preparation of the tabulated values:

$h = 30$ ft
Exposure B, $K_z = 0.70$
$K_d = 0.85$
$G = 0.85$
$K_{zt} = 1.0$
$I = 1.0$
$GCp_i = \pm 0.18$ (enclosed building)
MWFRS pressure coefficients from Figure 6-10
C&C pressure coefficients from Figure 6-11A

Multiplying factor "λ" is given for different mean roof heights and exposure classifications in Figures 6-2 and 6-3 of the Standard. For importance factors other than $I = 1.0$, tabulated pressure values should be multiplied by the appropriate value of I.

2.2.3 Method 2, Analytical Procedure

The analytical procedure for this method is applicable to

1. Buildings of all heights
2. Alternate low-rise buildings with mean roof height less than or equal to 60 ft and as defined in Section 6.2 of the Standard
3. Open buildings and other structures

The design procedure for each building type is delineated in Section 6.5.3 of the Standard. Velocity pressures, q_z or q_h, are determined in each case using Eq. 6-15 (see Section 6.5.10).

Design pressures for MWFRS and for C&C are determined separately. Generally, C&C design pressures will be higher because of localized high pressures acting over small areas. MWFRS receive wind pressures from several surfaces; hence, with spatial averaging and correlation, the pressures are likely to be smaller than those for C&C.

Calculation of design pressures requires selection of appropriate gust effect factors and pressure or force coefficients. The equation for the evaluation of wind loads guides the user in the selection of appropriate factors and coefficients. Various gust effect factors and pressure and force coefficients specified in the Standard are as follows:

- G — Gust effect factor for MWFRS of buildings (all heights) and for other structures (Section 6.5.8.1)
- G_f — Gust effect factor for MWFRS of flexible buildings and dynamically sensitive other structures obtained using the procedure given in Section 6.5.8.2 or using a rational analysis (see Section 6.5.8.3)
- C_p — External pressure coefficients for MWFRS of buildings: all heights (Figure 6-6); domed roof (Figure 6-7); and arched roof (Figure 6-8)
- C_f — Force coefficients for open buildings and other structures (Figures 6-18 through 6-22)
- (GC_{pf}) — External pressure coefficients for MWFRS of low-rise buildings (Figure 6-10)
- (GC_p) — External pressure coefficients for C&C of buildings (Figures 6-11A through 6-17)
- (GC_{pi}) — Internal pressure coefficients for MWFRS and C&C of buildings (Figure 6-5)

Sign convention in the Standard is as follows:

+ (plus sign) means pressure acting toward the surface
– (minus sign) means pressure acting away from the surface

Whenever the sign of "±" is specified, both positive and negative values should be used to obtain design loads. Values of external and internal pressures are to be combined algebraically to obtain the most critical load.

2.2.3.1 Design Pressures for Buildings

Design wind pressures for the MWFRS of rigid buildings of all (any) heights are determined using the following equation:

$$p = qGC_p - q_i(GC_{pi}) \text{ (lb/ft}^2\text{)} \quad \text{(Eq. 6-17)}$$

The terms in Eq. 6-17 are defined above. The effective velocity pressure related to internal pressure, q_I, is generally used as q_h (see Section 6.5.12.2.1 of the Standard). Only for high-rise building may it be advantageous to use q_i as defined in Section 6.5.12.2.1 related to positive internal pressure. Use of this term is illustrated in Ex. 3 (Section 3.3 of this guide).

Alternatively, design pressures for MWFRS of low-rise buildings can be determined using the following equation:

$$p = q_h[(GC_{pf}) - (GC_{pi})] \text{ (lb/ft}^2\text{)} \quad \text{(Eq. 6-18)}$$

The terms in Eq. 6-18 are defined above. A low-rise building is defined in Section 6.2 of the Standard as a building with mean roof height $h = 60$ ft and with mean roof height not exceeding the least horizontal dimension. The design pressures are applied for transverse and longitudinal directions as shown in Figure 6-10. This alternate procedure is appropriate for gable and rectangular buildings, though use of it for any building is permitted. Use of this procedure is illustrated in Ex. 7 (Section 3.7 of this guide).

Design wind pressures for the MWFRS of flexible buildings shall be determined from the following equation:

$$p = qG_fC_p - q_i(GC_{pi}) \text{ (lb/ft}^2\text{)} \quad \text{(Eq. 6-19)}$$

where the terms are as defined above, and G_f = gust effect factor as defined in Section 6.5.8.2 of the Standard. The procedure is the same as that for rigid buildings except for determination of gust effect factor, G_f. A flexible building (or structure) is defined in Section 6.2 as the one that has fundamental natural frequency less than 1 Hz (period of vibration greater than 1 s). Flexible buildings or structures are affected by the gustiness of the wind and have potential of resonance response. This response results in a large gust effect factor. Calculation of gust effect factor, G_f, for a flexible structure using Eq. 6-8 of Section 6.5.8.2 is illustrated in Ex. 10 (Section 3.10 of this guide).

Design wind pressures on C&C elements of buildings with $h \leq 60$ ft are determined from the following equation:

$$p = q_h[(GC_p) - (GC_{pi})] \text{ (lb/ft}^2\text{)} \quad \text{(Eq. 6-22)}$$

The terms in Eq. 6-22 are defined above.

Design wind pressures on C&C for buildings with $h > 60$ ft are determined from the following equation:

$$p = q(GC_p) - q_i(GC_{pi}) \text{ (lb/ft}^2\text{)} \quad \text{(Eq. 6-23)}$$

The terms in Eq. 6-23 are defined above.

An alternate procedure (see Section 6.5.12.4.3 of the Standard) to calculate design pressures on C&C for buildings with mean roof height of $60 < h = 90$ ft is to use Eq. 6-22 and associated pressure coefficients. Since this equation, which is for buildings with $h = 60$ ft, uses q_h for positive and negative external pressures, the resulting pressures may be higher in some cases.

If a component or cladding element has tributary area (not effective area) greater than 700 ft^2 (see Section 6.5.12.1.3 of the Standard), it is permitted to be designed using the provisions of MWFRS, Eq. 6-17, and associated pressure coefficients.

2.2.3.2 Design Wind Loads on Open Buildings and Other Structures

The design wind-force for open buildings and other structures is determined by the following equation:

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

The terms of Eq. 6-25 are defined above. The area, A_f, is the exposed area projected on a plane normal to the wind direction unless it is specified with the value of force coefficient, C_f. The force, F, is in the direction of wind except when it is specified with the value of C_f.

2.3 Method 3, Wind Tunnel Procedure

For those situations where the analytical procedure is considered uncertain or inadequate, or where more accurate wind pressures are desired, consideration should be given to wind tunnel tests. The Standard lists a set of conditions in Section 6.6 that must be satisfied for the proper conduct of such tests. The wind tunnel is particularly useful for obtaining detailed information about pressure distributions on complex shapes and the dynamic response of structures. Model scales for structural applications can range from 1:50 for a single-family dwelling to 1:400 for tall buildings. Even smaller scales may be used to model long-span bridges. Of equal importance is the ability to model complex topography at scales of the order of 1:10,000 and assess the effects of features such as hills, mountains, or river gorges on the near-surface winds. Details on wind tunnel modeling for structural or civil engineering applications may be found in ASCE (1999), Cermak (1977), and Reinhold (1982).

2.4 Equations for Graphs

Figures 6-11A through 6-17 of the Standard give external pressure coefficient (GCp) values for C&C for buildings as a function of effective area of component and cladding. Wind tunnel results found this relationship between pressure coefficients and effective area to be a logarithmic function. The scale of effective area in the figures is a log scale, which makes it very difficult to interpolate. Equations for each of the lines in these figures are given in Tables 2-1 through 2-10, below. The equations can be used to determine wind loads.

Table 2-1 Walls for Buildings with $h \leq 60$ ft (Figure 6-11A)

Positive: Zones 4 and 5	
$(GC_p) = 1.0$	for $A = 10\ \text{ft}^2$
$(GC_p) = 1.1766 - 0.1766 \log A$	for $10 < A = 500\ \text{ft}^2$
$(GC_p) = 0.7$	for $A > 500\ \text{ft}^2$
Negative: Zone 4	
$(GC_p) = -1.1$	for $A = 10\ \text{ft}^2$
$(GC_p) = -1.2766 + 0.1766 \log A$	for $10 < A = 500\ \text{ft}^2$
$(GC_p) = -0.8$	for $A > 500\ \text{ft}^2$
Negative: Zone 5	
$(GC_p) = -1.4$	for $A = 10\ \text{ft}^2$
$(GC_p) = -1.7532 + 0.3532 \log A$	for $10 < A = 500\ \text{ft}^2$
$(GC_p) = -0.8$	for $A > 500\ \text{ft}^2$

Note: Zones are shown in the figures referenced in ASCE 7-02.

Table 2-2 Gable Roofs with $h \leq 60$ ft, $\theta \leq 7°$ (Figure 6-11B)

Positive with and without overhang: Zones 1, 2, and 3	
$(GC_p) = 0.3$	for $A = 10\ \text{ft}^2$
$(GC_p) = 0.4000 - 0.1000 \log A$	for $10 < A = 100\ \text{ft}^2$
$(GC_p) = 0.2$	for $A > 100\ \text{ft}^2$
Negative without overhang: Zone 1	
$(GC_p) = -1.0$	for $A = 10\ \text{ft}^2$
$(GC_p) = -1.1000 + 0.1000 \log A$	for $10 < A = 100\ \text{ft}^2$
$(GC_p) = -0.9$	for $A > 100\ \text{ft}^2$
Negative without overhang: Zone 2	
$(GC_p) = -1.8$	for $A = 10\ \text{ft}^2$
$(GC_p) = -2.5000 + 0.7000 \log A$	for $10 < A = 100\ \text{ft}^2$
$(GC_p) = -1.1$	for $A > 100\ \text{ft}^2$
Negative without overhang: Zone 3	
$(GC_p) = -2.8$	for $A = 10\ \text{ft}^2$
$(GC_p) = -4.5000 + 1.7000 \log A$	for $10 < A = 100\ \text{ft}^2$
$(GC_p) = -1.1$	for $A > 100\ \text{ft}^2$
Negative with overhang: Zones 1 and 2	
$(GC_p) = -1.7$	for $A = 10\ \text{ft}^2$
$(GC_p) = -1.8000 + 0.1000 \log A$	for $10 < A = 100\ \text{ft}^2$
$(GC_p) = -3.0307 + 0.7153 \log A$	for $100 < A = 500\ \text{ft}^2$
$(GC_p) = -1.1$	for $A > 500\ \text{ft}^2$
Negative with overhang: Zone 3	
$(GC_p) = -2.8$	for $A = 10\ \text{ft}^2$
$(GC_p) = -4.8000 + 2.0000 \log A$	for $10 < A = 100\ \text{ft}^2$
$(GC_p) = -0.8$	for $A > 100\ \text{ft}^2$

Note: Zones are shown in the figures referenced in ASCE 7-02.

Table 2-3 Gable and Hip Roofs with $h \leq 60$ ft, $7° < \theta \leq 27°$ (Figure 6-11C)

Positive with and without overhang: Zones 1, 2, and 3	
$(GC_p) = 0.5$	for $A = 10$ ft^2
$(GC_p) = 0.7000 - 0.2000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = 0.3$	for $A > 100$ ft^2
Negative with and without overhang: Zone 1	
$(GC_p) = -0.9$	for $A = 10$ ft^2
$(GC_p) = -1.0000 + 0.1000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -0.8$	for $A > 100$ ft^2
Negative without overhang: Zone 2	
$(GC_p) = -1.7$	for $A = 10$ ft^2
$(GC_p) = -2.2000 + 0.5000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -1.2$	for $A > 100$ ft^2
Negative without overhang: Zone 3	
$(GC_p) = -2.6$	for $A = 10$ ft^2
$(GC_p) = -3.2000 + 0.6000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -2.0$	for $A > 100$ ft^2
Negative with overhang: Zone 2	
$(GC_p) = -2.2$	for all A ft^2
Negative with overhang: Zone 3	
$(GC_p) = -3.7$	for $A = 10$ ft^2
$(GC_p) = -4.9000 + 1.2000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -2.5$	for $A > 100$ ft^2

Note: Zones are shown in the figures referenced in ASCE 7-02.

Table 2-4 Gable Roofs with $h \leq 60$ ft, $27° < \theta \leq 45°$ (Figure 6-11D)

Positive with and without overhang: Zones 1, 2, and 3	
$(GC_p) = 0.9$	for $A = 10$ ft^2
$(GC_p) = 1.0000 - 0.1000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = 0.8$	for $A > 100$ ft^2
Negative with and without overhang: Zone 1	
$(GC_p) = -1.0$	for $A = 10$ ft^2
$(GC_p) = -1.2000 + 0.2000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -0.8$	for $A > 100$ ft^2
Negative without overhang: Zones 2 and 3	
$(GC_p) = -1.2$	for $A = 10$ ft^2
$(GC_p) = -1.4000 + 0.2000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -1.0$	for $A > 100$ ft^2
Negative with overhang: Zones 2 and 3	
$(GC_p) = -2.0$	for $A = 10$ ft^2
$(GC_p) = -2.2000 + 0.2000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -1.8$	for $A > 100$ ft^2

Note: Zones are shown in the figures referenced in ASCE 7-02.

Table 2-5 Multispan Gabled Roofs with $h \leq 60$ ft, $10° < \theta \leq 30°$ (Figure 6-13)

Positive: Zones 1, 2, and 3	
$(GC_p) = 0.6$	for $A = 10$ ft^2
$(GC_p) = 0.8000 - 0.2000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = 0.4$	for $A > 100$ ft^2
Negative: Zone 1	
$(GC_p) = -1.6$	for $A = 10$ ft^2
$(GC_p) = -1.8000 + 0.2000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -1.4$	for $A > 100$ ft^2
Negative: Zone 2	
$(GC_p) = -2.2$	for $A = 10$ ft^2
$(GC_p) = -2.7000 + 0.5000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -1.7$	for $A > 100$ ft^2
Negative: Zone 3	
$(GC_p) = -2.7$	for $A = 10$ ft^2
$(GC_p) = -3.7000 + 1.0000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -1.7$	for $A > 100$ ft^2

Note: Zones are shown in the figures referenced in ASCE 7-02.

Table 2-6 Multispan Gable Roofs with $h \leq 60$ ft, $30° < \theta \leq 45°$ (Figure 6-13)

Positive: Zones 1, 2, and 3	
$(GC_p) = 1.0$	for $A = 10$ ft^2
$(GC_p) = 1.2000 - 0.2000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = 0.8$	for $A > 100$ ft^2
Negative: Zone 1	
$(GC_p) = -2.0$	for $A = 10$ ft^2
$(GC_p) = -2.9000 + 0.9000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -1.1$	for $A > 100$ ft^2
Negative: Zone 2	
$(GC_p) = -2.5$	for $A = 10$ ft^2
$(GC_p) = -3.3000 + 0.8000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -1.7$	for $A > 100$ ft^2
Negative: Zone 3	
$(GC_p) = -2.6$	for $A = 10$ ft^2
$(GC_p) = -3.5000 + 0.9000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -1.7$	for $A > 100$ ft^2

Note: Zones are shown in the figures referenced in ASCE 7-02.

Table 2-7 Monoslope Roofs with $h \leq 60$ ft, $3° < \theta \leq 10°$ (Figure 6-14A)

Positive: All Zones	
$(GC_p) = 0.3$	for $A = 10$ ft^2
$(GC_p) = 0.4000 - 0.1000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = 0.2$	for $A > 100$ ft^2
Negative: Zone 1	
$(GC_p) = -1.1$	for all A ft^2
Negative: Zone 2	
$(GC_p) = -1.3$	for $A = 10$ ft^2
$(GC_p) = -1.4000 + 0.1000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -1.2$	for $A > 100$ ft^2
Negative: Zone 2′	
$(GC_p) = -1.6$	for $A = 10$ ft^2
$(GC_p) = -1.7000 + 0.1000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -1.5$	for $A > 100$ ft^2
Negative: Zone 3	
$(GC_p) = -1.8$	for $A = 10$ ft^2
$(GC_p) = -2.4000 + 0.6000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -1.2$	for $A > 100$ ft^2
Negative: Zone 3′	
$(GC_p) = -2.6$	for $A = 10$ ft^2
$(GC_p) = -3.6000 + 1.0000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -1.6$	for $A > 100$ ft^2

Note: Zones are shown in the figures referenced in ASCE 7-02.

Table 2-8 Monoslope Roofs with $h \leq 60$ ft, $10° < \theta \leq 30°$ (Figure 6-14B)

Positive: All Zones	
$(GC_p) = 0.4$	for $A = 10$ ft^2
$(GC_p) = 0.5000 - 0.1000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = 0.3$	for $A > 100$ ft^2
Negative: Zone 1	
$(GC_p) = -1.3$	for $A = 10$ ft^2
$(GC_p) = -1.5000 + 0.2000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -1.1$	for $A > 100$ ft^2
Negative: Zone 2	
$(GC_p) = -1.6$	for $A = 10$ ft^2
$(GC_p) = -2.0000 + 0.4000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -1.2$	for $A > 100$ ft^2
Negative: Zone 3	
$(GC_p) = -2.9$	for $A = 10$ ft^2
$(GC_p) = -3.8000 + 0.9000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -2.0$	for $A > 100$ ft^2

Note: Zones are shown in the figures referenced in ASCE 7-02.

Table 2-9 Sawtooth Roofs with $h \leq 60$ ft (Figure 6-15)

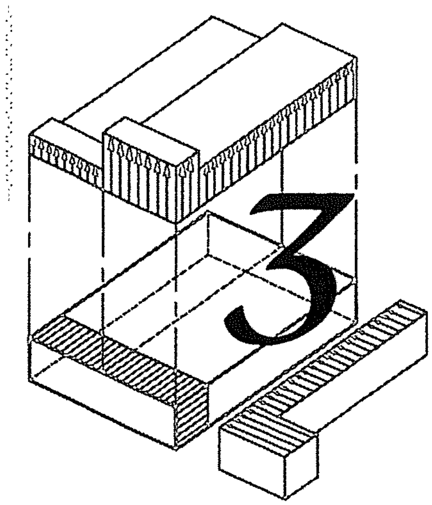
Positive: Zone 1	
$(GC_p) = 0.7$	for $A = 10$ ft^2
$(GC_p) = 0.8766 - 0.1766 \log A$	for $10 < A = 500$ ft^2
$(GC_p) = 0.4$	for $A > 500$ ft^2
Positive: Zone 2	
$(GC_p) = 1.1$	for $A = 10$ ft^2
$(GC_p) = 1.4000 - 0.3000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = 0.8$	for $A > 100$ ft^2
Positive: Zone 3	
$(GC_p) = 0.8$	for $A = 10$ ft^2
$(GC_p) = 0.9000 - 0.1000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = 0.7$	for $A > 100$ ft^2
Negative: Zone 1	
$(GC_p) = -2.2$	for $A = 10$ ft^2
$(GC_p) = -2.8474 + 0.6474 \log A$	for $10 < A = 500$ ft^2
$(GC_p) = -1.1$	for $A > 500$ ft^2
Negative: Zone 2	
$(GC_p) = -3.2$	for $A = 10$ ft^2
$(GC_p) = -4.1418 + 0.9418 \log A$	for $10 < A = 500$ ft^2
$(GC_p) = -1.6$	for $A > 500$ ft^2
Negative: Zone 3 (span A)	
$(GC_p) = -4.1$	for $A = 10$ ft^2
$(GC_p) = -4.5000 + 0.4000 \log A$	for $10 < A = 100$ ft^2
$(GC_p) = -8.2782 + 2.2891 \log A$	for $100 < A = 500$ ft^2
$(GC_p) = -2.1$	for $A > 500$ ft^2
Negative: Zone 3 (spans B, C, D)	
$(GC_p) = -2.6$	for $A = 100$ ft^2
$(GC_p) = -4.6030 + 1.0015 \log A$	for $100 < A = 500$ ft^2
$(GC_p) = -1.9$	for $A > 500$ ft^2

Note: Zones are shown in the figures referenced in ASCE 7-02.

Table 2-10 Roofs and Walls for Buildings with $h > 60$ ft (Figure 6-17)

Roofs $\theta \leq 10°$	
Negative: Zone 1	
$(GC_p) = -1.4$	for $A = 10$ ft^2
$(GC_p) = -1.6943 + 0.2943 \log A$	for $10 < A = 500$ ft^2
$(GC_p) = -0.9$	for $A > 500$ ft^2
Negative: Zone 2	
$(GC_p) = -2.3$	for $A = 10$ ft^2
$(GC_p) = -2.7120 + 0.4120 \log A$	for $10 < A = 500$ ft^2
$(GC_p) = -1.6$	for $A > 500$ ft^2
Negative: Zone 3	
$(GC_p) = -3.2$	for $A = 10$ ft^2
$(GC_p) = -3.7297 + 0.5297 \log A$	for $10 < A = 500$ ft^2
$(GC_p) = -2.3$	for $A > 500$ ft^2
Walls all θ	
Positive: Zones 4 and 5	
$(GC_p) = 0.9$	for $A = 20$ ft^2
$(GC_p) = 1.1792 - 0.2146 \log A$	for $20 < A = 500$ ft^2
$(GC_p) = 0.6$	for $A > 500$ ft^2
Negative: Zone 4	
$(GC_p) = -0.9$	for $A = 20$ ft^2
$(GC_p) = -1.0861 + 0.1431 \log A$	for $20 < A = 500$ ft^2
$(GC_p) = -0.7$	for $A > 500$ ft^2
Negative: Zone 5	
$(GC_p) = -1.8$	for $A = 20$ ft^2
$(GC_p) = -2.5445 + 0.5723 \log A$	for $20 < A = 500$ ft^2
$(GC_p) = -1.0$	for $A > 500$ ft^2

Note: Zones are shown in the figures referenced in ASCE 7-02.



3 Examples

In this chapter, twelve examples illustrate how wind loads are determined using the simplified and analytical procedures described in ASCE 7-02. These examples provide guidance to the user of the Standard in determining wind loads for several types of buildings:

Example no.	*Building/methodology*	*Section no.*	*Figure no. (this Guide)*
1	30-ft × 60-ft × 15-ft commercial building with concrete masonry unit (CMU) walls	3.1	3-1
2	Commercial building from Ex. 1 using simplified procedure	3.2	3-1
3	100-ft × 200-ft × 160-ft-high office building located in hurricane zone	3.3	3-6
4	Office building from Ex. 3 located on an escarpment	3.4	3-10
5	A typical 2,500-ft^2 house with gable/hip roof	3.5	3-11(a)–(d)
6	200-ft × 250-ft gable roof commercial/warehouse building using all height provisions	3.6	3-13
7	Commercial/warehouse building from Ex. 6 using low-rise building provisions	3.7	3-13
8	40-ft × 80-ft commercial building with monoslope roof with overhang	3.8	3-27
9	U-shaped apartment building	3.9	3-33
10	50-ft × 20-ft billboard sign on poles (flexible) 60 ft above ground	3.10	3-38
11	Domed roof building	3.11	3-40
12	Unusually shaped building	3.12	3-43

These examples represent a variety of situations in determination of wind loads. The equation, table, figure, and section numbers of ASCE 7-02 are cited where appropriate. Every effort has been made to check the accuracy of the numbers in calculations, although no absolute assurance is given.

3.1 Example 1 30-ft × 60-ft × 15-ft Commercial Building with Concrete Masonry Unit Walls

In this example, design wind pressures for a typical load-bearing one-story masonry building are determined. The building is shown in Figure 3-1, and data is as follows:

Location:	Corpus Christi, Texas
Topography:	Homogeneous
Terrain:	Flat, open terrain
Dimensions:	30 ft × 60 ft × 15 ft, flat roof
Framing:	CMU walls on three sides
	Steel framing in front with glass
	Open web joists, 30-ft span spaced at 5 ft on center, covered with metal panel to provide roof diaphragm action
Cladding:	Roof metal panels are 2-ft wide, 20-ft long
	Doors and glass size vary; glass is debris resistant

This example uses Method 2, Analytical Procedure, of Section 6.3 of ASCE 7-02 for rigid buildings of all heights. The same building is illustrated in Ex. 2 (Section 3.2) using Method 1, Simplified Procedure, of Section 6.4 of ASCE 7-02.

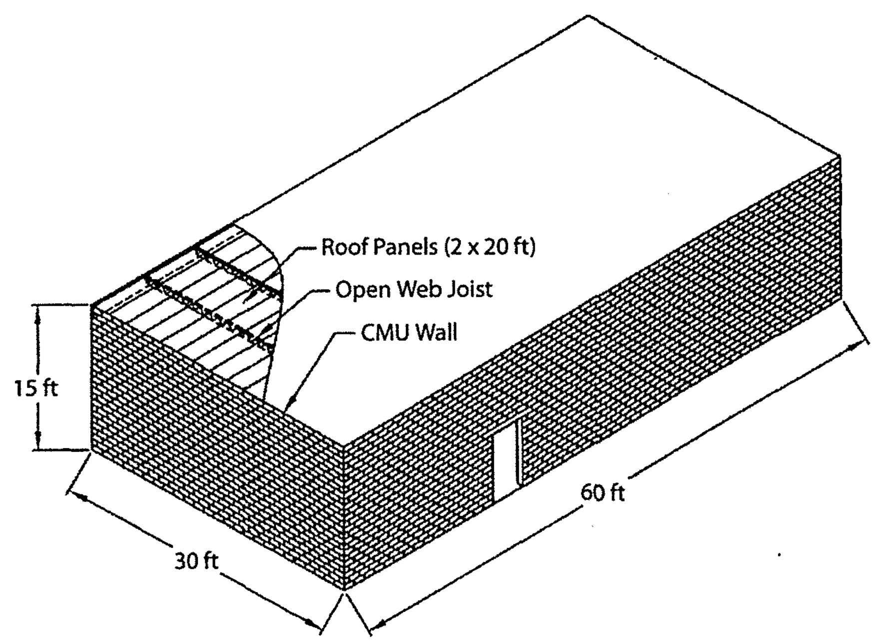


Figure 3-1 Building Dimensions for Examples 1 and 2

Basic Wind Speed

Selection of the basic wind speed is addressed in Section 6.5.4 of the Standard. Basic wind speed for Corpus Christi, Texas, is 130 mph (Figure 6-1a of the Standard).

Exposure

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

Building Classification

The building function is shops. It is not considered an essential facility. Building Category II is appropriate; see Table 1-1 of the Standard.

Velocity Pressure

The velocity pressures are computed using

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

where

K_z = 0.85 from Table 6-3 of the Standard for Case 1 (C&C) and Case 2 (MWFRS); for 0 to 15 ft, there is only one value: $K_z = k_h$

K_{zt} = 1.0 for homogeneous topography (see Section 6.5.7 of the Standard)

K_d = 0.85 for buildings (see Table 6-4 of the Standard)

V = 130 mph (see Figure 6-1a of the Standard)

I = 1.0 for Category II building (see Table 6-1 of the Standard)

$$q_z = 0.00256\ (0.85)\ (1.0)\ (0.85)\ (130)^2\ (1.0)$$
$$= 31.3 \text{ psf}$$

$q_h = 31.3$ psf for $h = 15$ ft

Gust Effect Factor

The building is considered a rigid structure. Section 6.5.8.1 of the Standard permits use of $G = 0.85$.

If the detailed procedure for a rigid structure is used (Section 6.5.8.1 of the Standard), the calculated value of $G = 0.89$; however, the Standard permits the use of the value of $G = 0.85$. Detailed calculations for G value are illustrated in Ex. 3 (Section 3.3 of this guide).

Use $G = 0.85$ for this example.

Internal Pressure Coefficient

The building is located in a hurricane-prone area (see definition of wind-borne debris region in Section 6.2 of the Standard). Section 6.5.9.3 requires that glazing be considered openings unless it is protected or debris resistant.

The example building has debris-resistant glazing, and other openings are such that it does not qualify for partially enclosed or open buildings.

Use (GC_{pi}) = +0.18 and −0.18 for enclosed buildings (see Figure 6-5 of the Standard).

Design Wind Pressures for MWFRS

Design wind pressures are determined using the equation

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

where

q = q_z for windward wall (31.3 psf for this example)
q = q_h for leeward wall, side walls, and roof (31.3 psf for this example)
G = 0.85
C_p = Values of external pressure coefficients
q_i = q_h for enclosed building (31.3 psf)
(GC_{pi}) = +0.18 and –0.18

The values of external pressure coefficients are obtained from Figure 6-6 of the Standard.

Wall C_p

The windward wall pressure coefficient is 0.8.

The side wall pressure coefficient is –0.7.

The leeward wall pressure coefficients are a function of L/B ratio:

For $L/B = 0.5$, $C_p = -0.5$ for wind normal to 60 ft

For $L/B = 2.0$, $C_p = -0.3$ for wind normal to 30 ft

Roof C_p

The roof pressure coefficients are a function of roof slope and h/L. For $\theta < 10°$ and $h/L = 0.25$ and 0.5:

First value:

$C_p = -0.9$ for distance 0 to h
$C_p = -0.5$ for distance h to $2h$
$C_p = -0.3$ for distance $>2h$

Second value:

$C_p = -0.18$ for distance 0 to end. This value 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 Section 2.3 and 2.4 of the Standard. For brevity, loading for this value is not shown in this example.

MWFRS Pressures

Windward wall:

$$p = 31.3\ (0.85)\ (0.8) - 31.3\ (\pm 0.18)$$
$$= 21.3 \pm 5.6 \text{ psf}$$

Leeward wall:

$$p = 31.3\ (0.85)\ (-0.5) - 31.3\ (\pm 0.18)$$
$$= -13.3 \pm 5.6 \text{ psf for wind normal to 60 ft}$$

Leeward wall:

$$p = 31.3\ (0.85)\ (-0.3) - 31.3\ (\pm 0.18)$$
$$= -8.0 \pm 5.6 \text{ psf for wind normal to 30 ft}$$

Roof – First value:

$$p = 31.3\ (0.85)\ (-0.9) - 31.3\ (\pm 0.18)$$
$$= -23.9 \pm 5.6 \text{ psf for 0 to 15 ft}$$
$$= -13.3 \pm 5.6 \text{ psf for 15 to 30 ft}$$
$$= -8.0 \pm 5.6 \text{ psf for} > 30 \text{ ft}$$

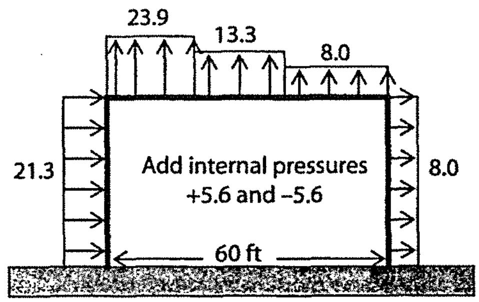


Figure 3-2 Design Pressures for MWFRS when Wind is Normal to 30-ft Wall

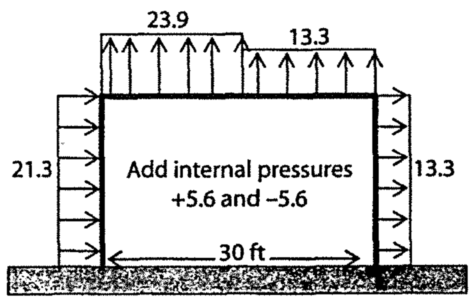


Figure 3-3 Design Pressures for MWFRS when Wind is Normal to 60-ft Wall

The MWFRS design pressures for two directions are shown in Figures 3-2 and 3-3. The internal pressures shown are to be added to the external pressures as appropriate. The internal pressures of the same sign act on all surfaces; thus, they cancel out for total horizontal shear.

Design Wind Load Cases

According to Section 6.5.12.3 of the Standard, this building shall be designed for the wind Load Cases 1 and 3 as defined in Figure 6-9.

Load Case 1 has been considered above. Figure 3-4 is for Load Case 3.

Design Pressures for C&C

Design wind pressures are determined using the equation

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

where

q_h = 31.3 psf

(GC_p) = Values obtained from Figure 6-11 of the Standard; they are a function of effective area and zone

$(GCpi)$ = +0.18 and −0.18

Wall Pressures

CMU walls are supported at the roof diaphragm and at ground, span = 15 ft.

CMU wall effective wind area is determined using the definition from Section 6.2 of the Standard: "the width of effective area need not be less than one-third of the span."

CMU wall effective wind area, $A = 15\ (15/3) = 75\ \text{ft}^2$

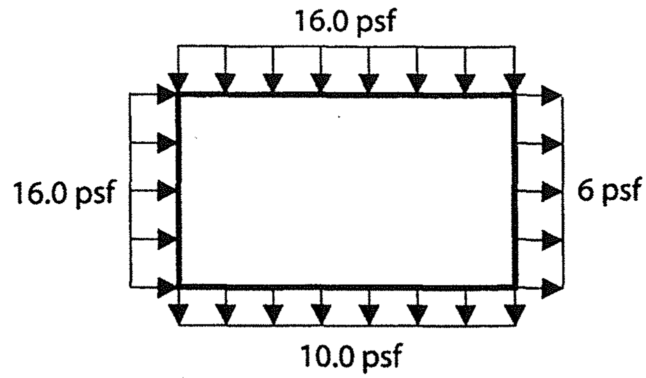


Figure 3-4 Load Case 3

In Figure 6-11A of the Standard, Note 5 suggests that the pressure coefficient values for walls can be reduced by 10% for roof slope of 10° or less. The values of (GC_p) are obtained from the figure or from equations of the graphs (see Section 2.4 of this guide).

Corner Zone 5 distance:

smaller of

$a = 0.1\ (30) = 3$ ft (controls)

or

$a = 0.4\ (15) = 6$ ft

Corner Zone 5:

$p = 31.3\ [(-1.09)\ (0.9) - (\pm 0.18)] = -36.3$ psf

$p = 31.3\ [(0.85)\ (0.9) - (\pm 0.18)] = +29.6$ psf

Interior Zone 4:

$p = 31.3\ [(-0.95)\ (0.9) - (\pm 0.18)] = -32.4$ psf

$p = 31.3\ [(0.85)\ (0.9) - (\pm 0.18)] = +29.6$ psf

Note: The CMU walls have uplift pressure from the roof, which is determined on the basis of MWFRS.

Pressure for glazing and mullions can be determined similarly with the known effective wind area.

Roof Joist Pressures

Roof joists span 30 ft and are spaced 5 ft apart. The joist can be in Zone 1 (interior of roof) or Zone 2 (eave area). Zone 3 (roof corner area) acts only on a part of the joist.

Width of Zones 2 and 3: (Figure 6–11B)

smaller of

$a = 0.1\ (30) = 3$ ft (controls)

or

$a = 0.4\ (15) = 6$ ft

Joist Effective Wind Area:

larger of

$A = 30 \times 5 = 150$ ft

or

$A = 30 \times (30/3) = 300$ ft (controls)

The values of (GC_p) are obtained from Figure 6-11B of the Standard or from equations of the graphs using effective area $A = 300\ \text{ft}^2$.

Interior Zone 1:

$$p = 31.3\ [-0.9 \pm 0.18] = -33.8\ \text{psf}$$

$$p = 31.3\ [+0.2 \pm 0.18] = +11.9\ \text{psf}$$

Eave Zone 2 and Corner Zone 3:

$$p = 31.3\ [-1.1 \pm 0.18] = -40.1\ \text{psf}$$

$$p = 31.3\ [+0.2 \pm 0.18] = +11.9\ \text{psf}$$

Roof Panel Pressures

Even though roof panel length is 20 ft, each panel spans 5 ft between joists.

Roof Panel Effective Area:

larger of

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

or

$A = 5 \times (5/3) = 8\ \text{ft}^2$

(width of Zones 2 and 3, $a = 3$ ft)

Interior Zone 1:

$$p = 31.3\ [-1.0 \pm 0.18] = -36.9\ \text{psf}$$

$$p = 31.3\ [+0.3 \pm 0.18] = +15.0\ \text{psf}$$

Eave Zone 2:

$$p = 31.3\ [-1.8 \pm 0.18] = -62.0\ \text{psf}$$

$$p = 31.3\ [+0.3 \pm 0.18] = +15.0\ \text{psf}$$

Corner Zone 3:

$$p = 31.3\ [-2.8 \pm 0.18] = -93.3\ \text{psf}$$

$$p = 31.3\ [+0.3 \pm 0.18] = +15.0\ \text{psf}$$

Notes:

- Internal pressure coefficient of +0.18 or −0.18 is used to give critical pressures.
- The roof panel fasteners design pressures will be the same as metal panel since values of (GC_p) are the same for wind effective areas less than $10\ \text{ft}^2$.

3.2 Example 2 Ex. 1 Using Simplified Procedure

In this example, design wind pressures for the building of Ex. 1 are determined using the simplified procedure of Section 6.4 of the Standard. Data for the building are the same as Ex. 1 (see Section 3.1 and Figure 3-1).

In order to use the simplified procedure, all conditions of Section 6.4.1 of the Standard must be satisfied:

1. It is a simple diaphragm building.
2. Its mean roof height h is less than 60 ft and does not exceed the least horizontal dimension.
3. Since the building has debris-resistant glazing and no dominant opening in any one wall, it can be classified as an enclosed building. It also conforms to the wind-borne debris provisions of Section 6.5.9.3 of the Standard.
4. It has a regular shape.
5. It is a rigid building (h/width << 4) (see Commentary in the Standard).
6. There is no expansion joint.
7. There is no abrupt change in topography (see Section 6.5.7.1 of the Standard for requirements of topographic effects).
8. It has an approximately symmetrical cross section in each direction with a flat roof.

Wind pressures for both the MWFRS and C&C can be obtained using the simplified procedure.

Basic Wind Speed

Basic wind speed for Corpus Christi, Texas, is 130 mph (see Figure 6-1a of the Standard).

Building Classification

Building Category II is appropriate. Importance Factor $I = 1.00$ (see Table 6-1 of the Standard).

Exposure

The building is located on flat and open terrain. It does not fit Exposures B or D; therefore, use Exposure C (Sections 6.5.6.2 and 6.5.6.3 of the Standard). Note that wind pressure values given in Figures 6-2 and 6-3 of the Standard are for Exposure B.

Height and Exposure Adjustment Coefficient λ

From Figure 6-2 of the Standard, $\lambda = 1.21$.

Design Wind Pressures for MWFRS (See Table 3-1)

This building has a flat roof, so only Load Case 1 is checked.

$$p_s = \lambda I p_{s30} \quad \text{(Eq. 6-1)}$$
$$= 1.21 \times 1.0 \times p_{s30}$$

In the simplified procedure, design roof pressure includes internal pressure. The wall pressure is the combined windward and leeward wall pressures (internal pressure cancels).

Table 3-1 Design Wind Pressures

Zones	*A*	*C*	*E*	*F*	*G*	*H*
p_s (psf)	32.4	21.5	–39.0	–22.1	–27.1	–17.2

Notes:

(1) Zones are defined in Figure 6-2 of the Standard: a = smaller of 0.1 × 30 = 3 ft (control) or 0.4 × 15 = 6 ft.

(2) The load patterns shown in Figure 3-5 shall be applied to each corner of the building in turn as the reference corner.

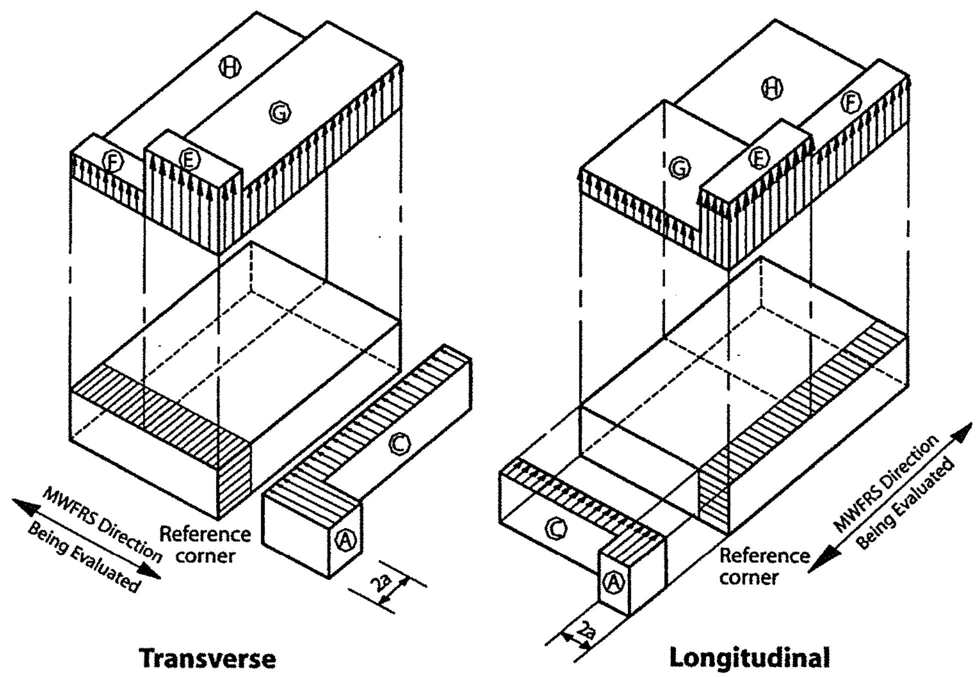


Figure 3-5 Design Wind Pressure

Design Pressures for C&C

According to Section 6.4.2.2 of the Standard:

$$p_{net} = \lambda I p_{net30} \qquad \text{(Eq. 6-2)}$$

$$= 1.21 \times 1.0 \times p_{net30}$$

Wall Pressures

The effective wind area for a CMU wall is 75 ft^2 (see Ex. 1). Linear interpolation is permitted in Figure 6-3 of the Standard.

Zone 4:

$$p_{net} = 1.21 \times 1.0 \times 26.6 = 32.2 \text{ psf}$$

$$p_{net} = 1.21 \times 1.0 \times (-29.1) = -35.2 \text{ psf}$$

Zone 5:

$$p_{net} = 1.21 \times 1.0 \times 26.6 = 32.2 \text{ psf}$$

$$p_{net} = 1.21 \times 1.0 \times (-33.0) = -39.9 \text{ psf}$$

Roof Joist Pressures

From Figure 6-3 of the Standard, for $V = 130$ mph, for effective wind area of 300 ft^2, the design pressures are:

Zone 1:

$$p_{net} = 1.21 \times 1.0 \times 9.8 = 11.9 \text{ psf}$$

$$p_{net} = 1.21 \times 1.0 \times (-27.8) = -33.6 \text{ psf}$$

Zones 2 and 3:

$$p_{net} = 1.21 \times 1.0 \times 9.8 = 11.9 \text{ psf}$$

$$p_{net} = 1.21 \times 1.0 \times (-33.0) = -39.9 \text{ psf}$$

Roof Panel Pressures

Effective wind area for roof panel is 10 ft^2 (see Ex. 1).

From Figure 6-3 of the Standard, for $V = 130$ mph, for effective wind area of 10 ft^2, the design pressures are:

Zone 1:

$$p_{net} = 1.21 \times 1.0 \times 12.4 = 15.0 \text{ psf}$$

$$p_{net} = 1.21 \times 1.0 \times (-30.4) = -36.8 \text{ psf}$$

Zone 2:

$$p_{net} = 1.21 \times 1.0 \times 12.4 = 15.0 \text{ psf}$$

$$p_{net} = 1.21 \times 1.0 \times (-51.0) = -61.7 \text{ psf}$$

Zone 3:

$$p_{net} = 1.21 \times 1.0 \times 12.4 = 15.0 \text{ psf}$$

$$p_{net} = 1.21 \times 1.0 \times (-76.8) = -92.9 \text{ psf}$$

The analytical procedure in Ex. 1 yields C&C design pressures close to the results of the simplified procedure.

3.3 Example 3 100-ft × 200-ft × 160-ft-High Office Building

This building is illustrated in Figure 3-6; data for the building is as follows:

Location:	Near Houston, Texas
Topography:	Homogeneous
Terrain:	Suburban
Dimensions:	100 ft × 200 ft in plan Roof height of 157 ft with 3-ft parapet Flat roof
Framing:	Reinforced concrete rigid frame in both directions Floor and roof slabs provide diaphragm action Fundamental natural frequency is greater than 1 Hz (Since the height to least horizontal dimension is less than 4, the fundamental frequency is judged to be greater than 1 Hz.)
Cladding:	Mullions for glazing panels span 11 ft between floor slabs Mullion spacing is 5 ft

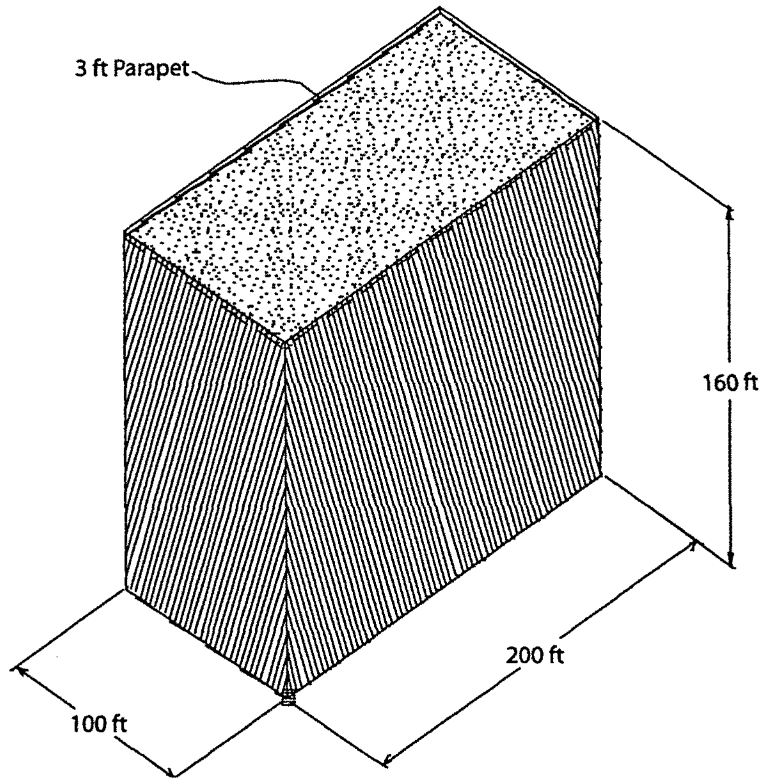


Figure 3-6 100-ft × 200-ft × 160-ft Building

Glazing panels are 5-ft wide × 5-ft 6 in. high (typical); they are not resistant to wind-borne debris impact. Also, there are buildings up to 60 ft in height located within 1,500-ft radius.

The analytical procedure of ASCE 7-02 is to be used.

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 office space. It is not considered an essential facility or 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).

Basic Wind Speed

Selection of the basic wind speed is addressed in Section 6.5.4 of the Standard. Vicinity of Houston, Texas, is located on the 120-mph contour. The basic wind speed V = 120 mph (see Figure 6-1a 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

Table 3-2 q_z Velocity Pressures

	MWFRS		*C&C*	
Height (ft)	K_z	q_z *(psf)*	K_z	q_z *(psf)*
0–15	0.57	17.9	0.70	21.9
30	0.70	21.9	0.70	21.9
50	0.81	25.4	0.81	25.4
80	0.93	29.1	0.93	29.1
120	1.04	32.6	1.04	32.6
Roof = 157	1.12	35.1	1.12	35.1
Parapet = 160	1.13	35.4	1.13	35.4

Note: q_h = 35.1 psf.

K_d = 0.85 for buildings (see Table 6-4 of the Standard)
V = 120 mph
I = 1.0 for Category II classification (see Table 6-1 of the Standard)

$$q_z = 0.00256K_z\,(1.0)\;(0.85)\;(120)^2\;(1.0)$$
$$= 31.3\;K_z\text{ psf}$$

Values for K_z and the resulting velocity pressures are given in Table 3-2. The velocity pressure at mean roof height, q_h, is 35.1 psf.

Design Wind Pressures for the MWFRS

The design pressures for this building are obtained by 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 for leeward wall, side walls, and roof
q_i = q_h for windward walls, side walls, leeward walls, and roofs for negative internal pressure evaluation in partially enclosed building
q_i = q_z for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure
G = Gust effect factor for rigid building and structure
C_p = External pressure coefficient
(GC_{pi}) = Internal pressure coefficient

Gust Effect Factor, G

Dimensions of this building where h/least width = 1.6 < 4.0 indicates that it is a rigid structure:

$$G = 0.925\left\{\frac{(1+1.7\,g_Q I_{\bar{z}} Q)}{(1+1.7\,g_v I_{\bar{z}})}\right\} \qquad \text{(Eq. 6-4)}$$

$$g_Q = g_v = 3.4 \qquad \text{(Sec. 6.5.8.1)}$$

$$\bar{z} = 0.6(157) = 94.2 \text{ ft (controls)} \qquad \text{(Sec. 6.5.8.1)}$$

$$\bar{z} = z_{min} = 30 \text{ ft} \qquad \text{(Table 6-2)}$$

$$c = 0.30 \qquad \text{(Table 6-2)}$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{94.2}\right)^{1/6} = 0.25 \qquad \text{(Eq. 6-5)}$$

$$L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{94.2}{33}\right)^{1/3} = 454 \text{ ft} \qquad \text{(Eq. 6-7)}$$

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

$B = 100$ ft (smaller value gives larger G)

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{100+157}{454}\right)^{0.63}}} = 0.83$$

$$G = 0.925\left\{\frac{(1+1.7\times 3.4\times 0.25\times 0.83)}{(1+1.7\times 3.4\times 0.25)}\right\} = 0.83$$

Wall External Pressure Coefficients, C_p (see Table 3-3)

The values for the external pressure coefficients for the various wall surfaces are obtained from Figure 6-6 of the Standard. The windward wall pressure coefficient is 0.8. The side wall pressure coefficient is –0.7.

The leeward wall pressure coefficient is a function of the L/B ratio. For wind normal to the 200-ft face, $L/B = 100/200 = 0.5$; therefore, the leeward wall pressure coefficient is –0.5. For wind normal to 100-ft face, $L/B = 200/100 = 2.0$; therefore, the leeward wall pressure coefficient is –0.3.

Roof C_p (with the Wind Normal to the 200-ft Face) (Table 3-4)

For $h/L = 157/100 \approx 1.6 > 1.0$, and $\theta < 10°$, two zones are specified in Figure 6-6 of the Standard:

First value:

0 to $h/2$, $C_p = -1.3$

$> h/2$, $C_p = -0.7$

Table 3-3 Wall C_p for Ex. 3

Surface	*Wind direction*	*L/B*	C_p
Windward wall	All	All	0.80
Leeward wall	⊥ to 200-ft face	0.5	–0.50
	‖ to 200-ft face	2.0	–0.30
Side wall	All	All	–0.70

Table 3-4 Roof C_p for Wind Normal to 200-ft Face

Distance from leading edge	C_p
0 to $h/2$	–1.04
> $h/2$	–0.70
Note: h = 157 ft.	

Table 3-5 Roof C_p for Wind Normal to 100-ft Face

Distance from windward edge	$h/L \leq 0.5$	$h/L = 0.8$	$h/L \geq 1.0$
0 to $h/2$	–0.9	–0.98	–1.04
$h/2$ to h	–0.9	–0.78	–0.7
h to $2h$	–0.5	–0.62	–0.7

Second value:

C_p = –0.18. This value 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 Section 2.3 and 2.4 of the Standard. For brevity, loading for this value is not shown in this example.

The $C_p = -1.3$ may be reduced with the area over which it is applicable.

Area = 200 × 79 = 15,800 ft^2

Reduction factor = 0.8

Reduced $C_p = 0.8 \times (-1.3) = -1.04$

Roof C_p (with the Wind Normal to 100-ft Face) (Table 3-5)

For h/L = 157/200 ≈ 0.8, interpolation in Figure 6-6 of the Standard is required.

Roof Calculation for 0 to 79 ft (h/2) from Edge (Wind Normal to 200-ft Face)

External pressure = 35.1(0.83)(–1.04) = –30.3

Roof Calculation for 79 (h/2) to 100-ft from Edge (Wind Normal to 200-ft Face)

External pressure = 35.1(0.83)(–0.70) = –20.4

External pressures are summarized in Tables 3-6 and 3-7.

Internal Pressure Coefficients, (GC_{pi})

The building is in a hurricane-prone region. Since the glazing is assumed not to be debris resistant, glazing is considered as opening. The building is classified as partially enclosed building.

MWFRS Pressures

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

For partially enclosed buildings:

Table 3-6 External Pressures for MWFRS: Wind Normal to 200-ft Face

Surface	*z* *(ft)*	*q* *(psf)*	C_p	*External pressures* *(psf)*
Windward wall	0–15	17.9	0.80	11.9
	30	21.9	0.80	14.5
	50	25.4	0.80	16.9
	80	29.1	0.80	19.3
	120	32.6	0.80	21.7
	157	35.1	0.80	23.3
Leeward wall	All	35.1	–0.50	–14.6
Side walls	All	35.1	–0.70	–20.4
Roof	0–79	35.1	–1.04	–30.3
	79–100	35.1	–0.70	–20.4

Note: q_h = 35.1 psf; G = 0.83.

Table 3-7 External Pressures for MWFRS: Wind Normal to 100-ft Face

Surface	*z* *(ft)*	*q* *(psf)*	C_p	*External pressure* *(psf)*
Windward wall	0–15	17.9	0.80	11.9
	30	21.9	0.80	14.5
	50	25.4	0.80	16.9
	80	29.1	0.80	19.3
	120	32.6	0.80	21.7
	157	35.1	0.80	23.3
Leeward wall	All	35.1	–0.30	–8.7
Side walls	All	35.1	–0.70	–20.4
Roof	0–79	35.1	–0.98	–28.6
	79–157	35.1	–0.78	–22.7
	157–200	35.1	–0.62	–18.1

Note: q_h = 35.1 psf; G = 0.83.

$$GC_{pi} = \pm\, 0.55 \qquad \text{(Table 6-7)}$$

For q_i, q_h = 35.1 psf for negative internal pressure, and q_z will be evaluated at 90 ft for positive internal pressure (30 ft above height of surrounding buildings in 1,500-ft radius; see Section 6.5.9.3 of the Standard).

Internal Pressure Calculation

Negative internal pressure = 35.1 × (–0.55) = –19.3 psf

Positive internal pressure = 30.0 × 0.55 = 16.5 psf (q_z is gained by interpolation at height z = 90 ft)

Parapet Load on MWFRS

According to Section 6.5.12.2.4 of the Standard:

$$P_p = q_p GC_{pn} \quad \text{(Eq. 6-20)}$$

$q_p = 35.4$ psf

GC_{pn} = 1.8 for windward parapet
= −1.1 for leeward parapet

Parapet is 3 ft high; the force on parapets of MWFRS can be determined as follows:

F = 35.4 × 1.8 × 3 = 191.2 plf for windward parapet
= 35.4 × (−1.1) × 3 = −116.8 plf for leeward parapet

This force is to be applied on the windward parapet and to the leeward parapet.

Design wind pressures for MWFRS are shown in Figure 3-7 for wind normal to 200-ft face and in Figure 3-8 for wind normal to 100-ft face.

For design of parapet, see the loads on components and cladding.

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 Figures 3-7 and 3-8. A combination of windward (P_W) and leeward (P_L) loads are applied for Load Cases 2, 3, and 4 as shown in Figure 3-9.

Design Pressures for C&C

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

$$p = q(GC_p) - q_i(GC_{pi}) \quad \text{(Eq. 6-23)}$$

where

q = q_z for windward wall calculated at height z and q_h for leeward wall, side walls, and roof calculated at height h

q_i = q_h = 35.1 psf for negative internal pressure
= q_z evaluated at 90 ft = 30.0 psf for positive internal pressure

(GC_p) = External pressure coefficient (see Figure 6-17 of the Standard)

(GC_{pi}) = Internal pressure coefficient (see Figure 6-5 of the Standard)

Wall Design Pressures (Table 3-8)

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:

Mullion:

larger of

$A = 11(5) = 55\ \text{ft}^2$ (controls)

or

$A = 11(11/3) = 40.3\ \text{ft}^2$

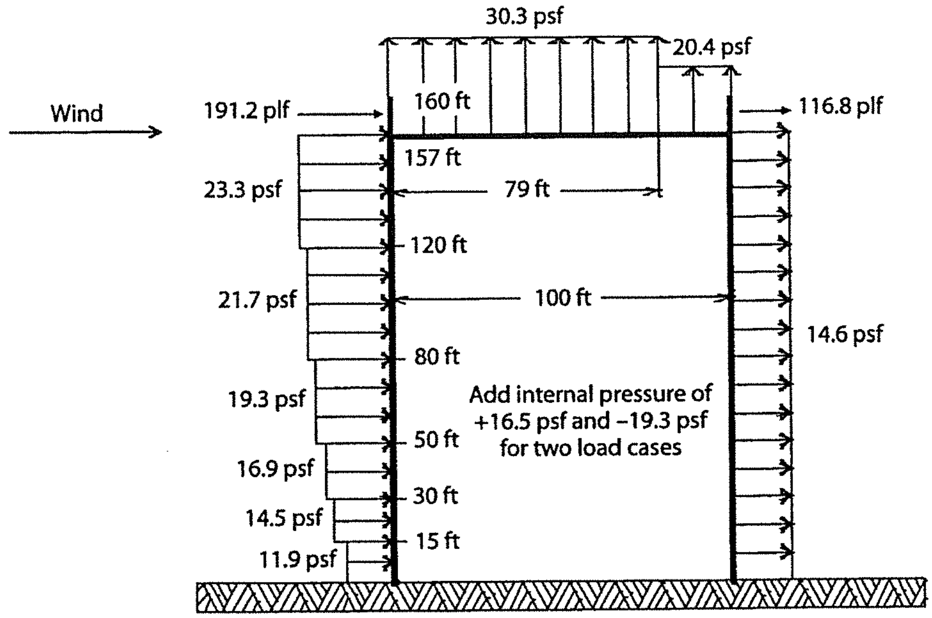


Figure 3-7 Pressures for MWFRS for Wind Normal to the 200-ft Face

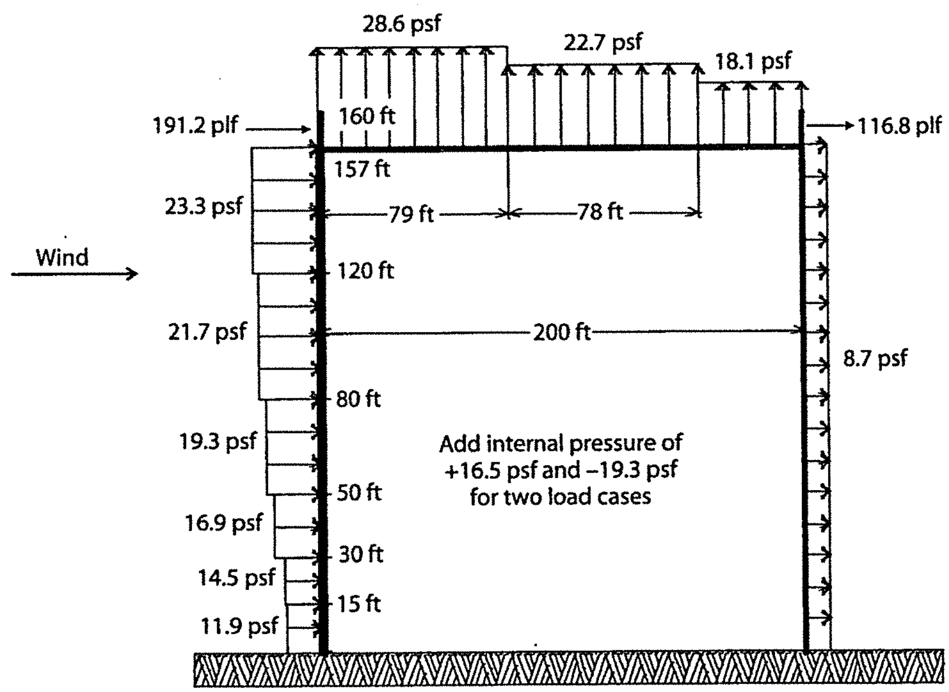


Figure 3-8 Pressures for MWFRS for Wind Normal to 100-ft Face

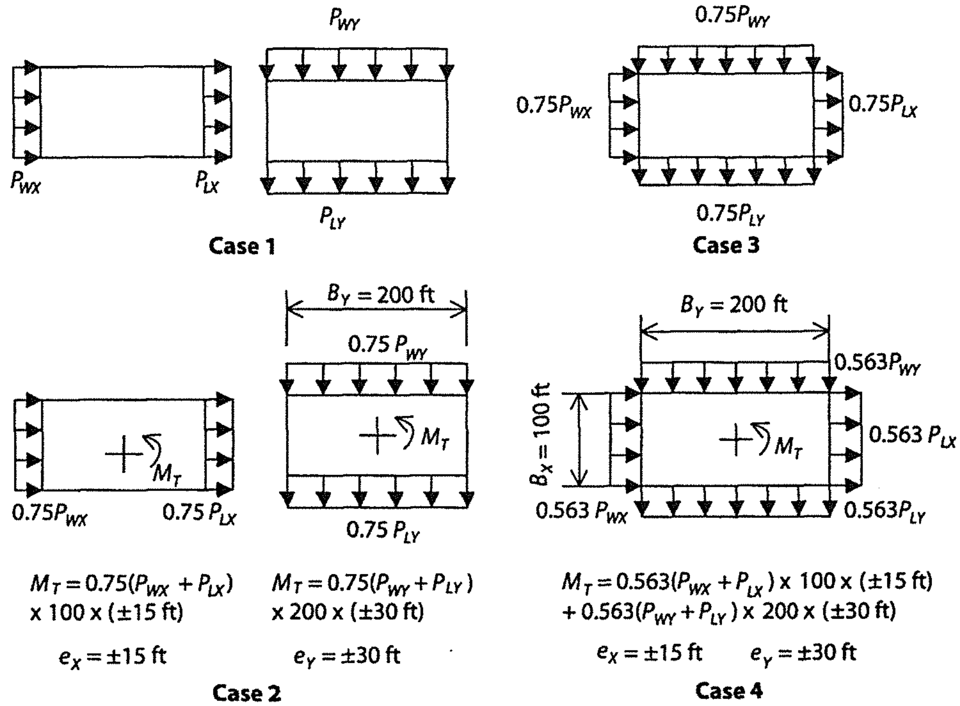


Figure 3-9 Pressures in Case B for MWFRS for Wind Normal to 100-ft Face

Table 3-8 Wall (GC_p) for Ex. 3

Component	*A* (ft^2)	*Zones 4 and 5* ($+GC_p$)	*Zone 4* ($-GC_p$)	*Zone 5* ($-GC_p$)
Mullion	55	0.81	–0.84	–1.55
Panel	27.5	0.87	–0.88	–1.72

Glazing panel:

larger of

$A = 5(5.5) = 27.5\ ft^2$ (controls)

or

$A = 5(5/3) = 8.3\ ft^2$

Width of corner Zone 5:

larger of

$a = 0.1(100) = 10$ ft (controls)

or

$a = 3$ ft

The internal pressure coefficient $(GC_{pi}) = \pm 0.55$ (Figure 6-5)

Typical Design Pressure Calculations (Tables 3-9 and 3-10)

Controlling negative design pressure for mullion in Zone 4 of walls:

$= 35.1(-0.84) - 30 \times 0.55$

$= -46.0$ psf (positive internal pressure controls)

Table 3-9 Controlling Design Pressures for Mullions (psf)

	Design pressure			
	Zone 4		*Zone 5*	
z (ft)	*Positive*	*Negative*	*Positive*	*Negative*
0–15	37.0	–46.0	37.0	–70.9
15–30	37.0	–46.0	37.0	–70.9
30–50	39.9	–46.0	39.9	–70.9
50–80	42.9	–46.0	42.9	–70.9
80–120	45.7	–46.0	45.7	–70.9
120–157	47.7	–46.0	47.7	–70.9

Table 3-10 Design Pressures of Panels (psf)

	Design pressure			
	Zone 4		*Zone 5*	
z (ft)	*Positive*	*Negative*	*Positive*	*Negative*
0–15	38.4	–47.4	38.4	–76.9
15–30	38.4	–47.4	38.4	–76.9
30–50	41.4	–47.4	41.4	–76.9
50–80	44.6	–47.4	44.6	–76.9
80–120	47.7	–47.4	47.7	–76.9
120–157	49.8	–47.4	49.8	–76.9

Controlling positive design pressure for mullion in Zone 4 of walls at roof height:

$= 35.1 \times 0.81 - 35.1 \times (-0.55)$
$= 47.7$ psf (negative internal pressure controls)

Controlling negative pressure is obtained with positive internal pressure, and controlling positive pressure is obtained with negative internal pressure.

Parapet Design Pressures

The design wind pressure on the C&C elements of parapets shall be determined according to the following equation (Section 6.5.12.4.4 of the Standard). In this example, the effective wind area is assumed to be 3 ft × 3 ft = 9 ft^2.

$$p = q_p\,(GC_p - GC_{pi}) \qquad \text{(Eq. 6-24)}$$

where

q_p = Velocity pressure evaluated at the top of parapet

GC_p = External pressure coefficient from Figures 6-11 through 6-17 of the Standard

GC_{pi} = Internal pressure coefficient from Figure 6-5 of the Standard, based on the porosity of the parapet envelope. In this example, internal pressure is not included since parapet is assumed to be nonporous.

Note that, according to Note 7 of Figure 6-17, Zone 3 is treated as Zone 2.

Load Case A:

$$35.4 \times [(0.9) - (-2.3)] = 113.3 \text{ psf (directed inward)}$$

Load Case B:

$$35.4 \times [(0.9) - (-1.8)] = 95.6 \text{ psf (directed outward)}$$

Roof Design Pressures

The C&C roof pressure coefficients are given in Figure 6-17 of the Standard. The pressure coefficients (Table 3-11) are a function of the effective wind area. Since specific components of roofs are not identified, design pressures are given for various effective wind areas, *A*.

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

$$\text{Design internal pressures} = 30 \times 0.55 = 16.5 \text{ psf}$$

$$\text{Design pressures} = q_h\ (GC_p) - 16.5 = 35.1\ (GC_p) - 16.5$$

Design pressures are summarized in Table 3-12.

Table 3-11 Roof External Pressure Coefficient (GC_p)

A (ft^2)	Zone 1 GC_p	Zones 2 and 3 $-GC_p$*
≤ 10	–1.40	–2.30
20	–1.31	–2.18
100	–1.11	–1.89
250	–0.99	–1.72
400	–0.93	–1.64
≥ 500	–0.90	–1.60

*Note 7 in Figure 6-17 of the Standard permits treatment of Zone 3 as Zone 2 if parapet of 3 ft or higher is provided.

Table 3-12 Roof Design Pressures (psf)

	Design pressures negative	
A(ft^2)	Zone 1	Zones 2 and 3
≤ 10	–65.6	–97.2
20	–62.5	–93.0
100	–55.5	–82.8
250	–51.2	–76.9
400	–49.1	–74.1
500	–48.1	–72.7

3.4 Example 4 Office Building of Ex. 3 Located on an Escarpment

In this example, velocity pressures for the office building of Ex. 3, when it is located on an escarpment, are determined. Design pressures for MWFRS and C&C can be determined in the same manner as Ex. 3 once velocity pressures q_z and q_h are determined. The building is illustrated in Figure 3-10; data are provided below.

Location:	City in Alaska
Topography:	Escarpment as shown
Terrain:	Suburban
Dimensions:	100 ft × 200 ft in plan Roof height of 157 ft with 3-ft parapet Flat roof
Framing:	Reinforced concrete rigid frame in both directions Floor and roof slabs provide diaphragm action Fundamental natural frequency is greater than 1 Hz
Cladding:	Mullions for glazing panels span 11 ft between floor slabs Mullion spacing is 5 ft

Glazing panels are 5-ft wide × 5-ft 6 in. high (typical). Glazing does not have to be wind-borne debris impact resistant because Alaska is not in a hurricane-prone region (see Section 6.5.9.3 of the Standard).

Exposure, Building Classification, and Basic Wind Speed

Same as Ex. 3: Exposure B

Category II

V = 120 mph, same as Ex. 3

Velocity Pressures

The velocity pressure equation is:

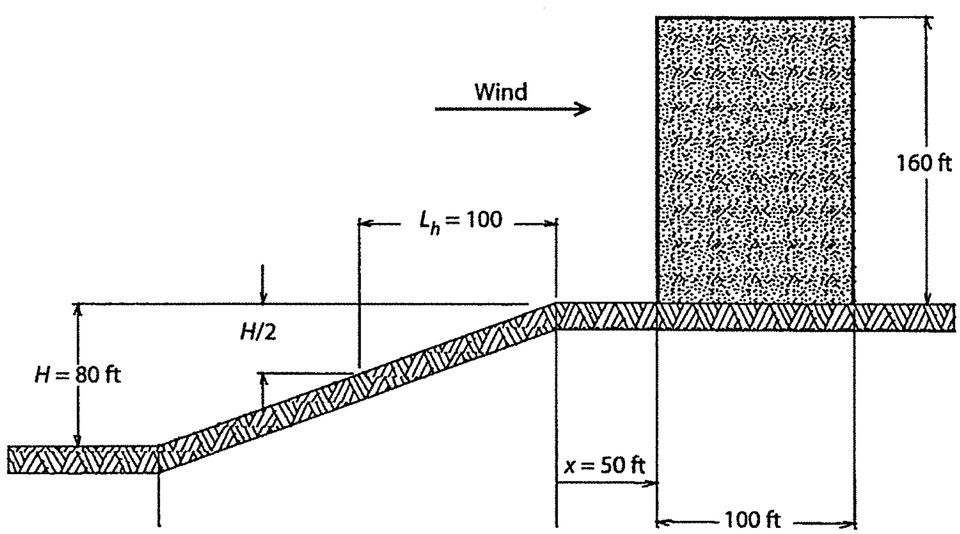


Figure 3-10 Office Building on Escarpment

Notes: 1) L_h is measured from mid-height to top of the slope. 2) x distance is taken to the front of the building as a conservative value.

Table 3-13 Speed-up Velocity Pressures (psf)

Height (ft)	K_z	z/L_h*	K_3	K_{zt}	q_z *(psf)*
0–15	0.57	0.05	0.88	1.71	30.5
30	0.70	0.14	0.71	1.56	34.2
50	0.81	0.25	0.54	1.41	35.8
80	0.93	0.41	0.36	1.27	37.0
120	1.04	0.63	0.21	1.15	37.5
h = 157	K_h = 1.12	0.87	0.11	1.08	37.9

*z is taken midway between the height range because it is unconservative for K_{zt} to take top height of the range. L_h = 160 ft.

$$q_z = 0.00256\ K_z K_{zt} K_d V^2 I \text{ psf}$$

For this example, K_z is obtained from Table 6-3 of the Standard, and K_{zt} is determined using Figure 6-4. K_d = 0.85 from Table 6-4 of the Standard; I = 1.0 for Category II from Table 6-1 and V = 120 mph.

Determination of K_{zt}

The topographic effect of escarpment applies only when the upwind terrain is free of topographic features for a distance equal to 100 H or 2 mi, whichever is smaller. For this example, it is assumed that there are no topographic features upwind for a distance of 8,000 ft.

For use in Figure 6-4 of the Standard:

H = 80 ft

L_h = 100 ft

x = 50 ft (distance to the front face of the building)

Since $H/L_h = 0.8 > 0.5$, according to Note 2 in Figure 6-4 of the Standard, use $H/L_h = 0.5$ and $L_h = 2H = 160$ ft.

The building is on a 2-D escarpment.

For Exposure B,

$K_1/(H/L_h) = 0.75$, therefore $K_1 = (0.75)(0.5) = 0.38$ (Figure 6-4)

For x/L_h = 50 ft/160 ft = 0.31;

$K_2 = [1 - (0.31/4)] = 0.92$ (Figure 6-4)

$$K_3 = e^{-2.5z/L_h} \text{ (values in table for } z)$$
$$K_{zt} = (1 + K_1 K_2 K_3)^2 \quad \text{(Eq. 6-3)}$$
$$q_z = 0.00256 K_z K_{zt} (0.85)(120)^2 (1.0)$$

Values for q_z are shown in Table 3-13.

Effect of Escarpment

Velocity pressures q_z are compared with the values of Ex. 3 in Table 3-14 to assess the effect of the escarpment. The increase in velocity pressures does not directly translate into an increase in design pressures as discussed below.

Table 3-14 Velocity Pressure q_z (psf)

Height (ft)	*Homogeneous terrain (Ex. 3)*	*Escarpment (Ex. 4)*	*% Increase*
0–15	17.9	30.5	71
30	21.9	34.2	56
50	25.4	35.8	41
80	29.1	37.0	27
120	32.6	37.5	15
157 (roof)	35.1	37.9	8

For MWFRS, the external windward wall pressures will increase by the percentages shown at various heights; however, the external leeward wall, side wall, and roof pressures will increase by 8% since these pressures are controlled by velocity pressure at roof height, q_h. Internal pressures will depend on assessment of openings.

For C&C, the negative (outward acting) external pressures will also increase by only 8%.

3.5 Example 5 2,500-ft^2 House with Gable/Hip Roof

Design wind pressures for a typical one-story house are to be determined. Various views of the house are provided in Figure 3-11(a) through 3-11(d). The physical data are as follows:

Location:	Dallas–Fort Worth, Texas
Topography:	Homogeneous
Terrain:	Suburban
Dimensions:	80 ft × 40 ft (including porch) footprint Porch is 8 ft × 48 ft Wall eave height is 10 ft Roof gable $\theta = 15°$; roof overhang is 2 ft all around
Framing:	Typical timber construction Wall studs are spaced 16 in. on center Roof trusses spanning 32 ft are spaced 4 ft on center Roof panels are 4 ft × 8 ft

Glazing is uniformly distributed (pressures on C&C will depend on effective area and location; for brevity, all items are not included).

Wind speed $V = 90$ mph

Importance factor $I = 1.0$

Topography factor $K_{zt} = 1.0$

Directionality factor $K_d = 0.85$ (for buildings)

The building is located in a suburban area; according to Section 6.5.6.2 and 6.5.6.3 of the Standard, Exposure B is used.

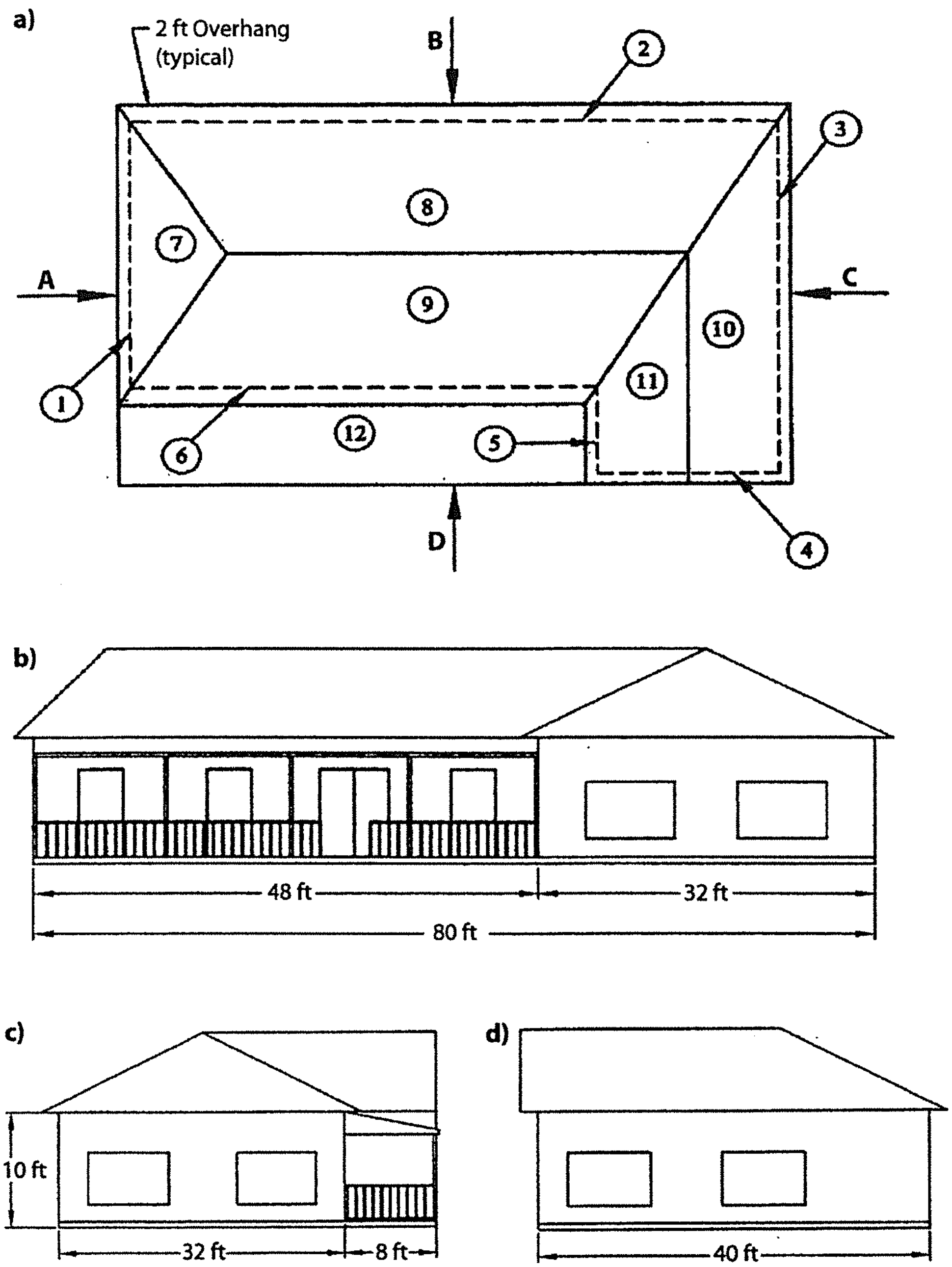


Figure 3-11 a, View of Roof of 2,500-ft^2 House; b, Front View D; c, Side View A; d, Side View C

$$\text{mean roof height} = 10 + \frac{(16)(\tan 15°)}{2} = 12.1 \text{ ft}$$

Since K_z is constant in the 0 to 15 ft region, from Table 6-3 of the Standard:

$K_z = K_h = 0.70$ for Case 1 (C&C)

$K_z = K_h = 0.57$ for Case 2 (MWFRS)

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

For MWFRS,

$$q_z = q_h = 0.00256\ (0.57)\ (1.0)\ (0.85)\ (90)^2\ (1.0)$$
$$= 10.1 \text{ psf}$$

For C&C,

$$q_z = q_h = 0.00256\ (0.7)\ (1.0)\ (0.85)\ (90)^2\ (1.0)$$
$$= 12.3 \text{ psf}$$

Gust Effect Factor

$G = 0.85$ (Section 6.5.8.1)

$(GC_{pi}) = +0.18$ and -0.18 (Figure 6-5)

Wind Pressure for MWFRS

Because of asymmetry, all four wind directions are considered (normal to walls).

The wall surfaces are numbered 1 through 6; roof surfaces are 7 through 11; porch roof surface is 12.

Wind Direction A

Wall pressures:

Surface 1: $p = 10.1(0.85)(0.8) - 10.1(\pm 0.18) = +6.9 \pm 1.8$ psf (windward)

Surface 2: $p = 10.1(0.85)(-0.7) - 10.1(\pm 0.18) = -6.0 \pm 1.8$ psf (side)

Surface 3: $p = 10.1(0.85)(-0.3) - 10.1(\pm 0.18) = -2.6 \pm 1.8$ psf (leeward) (for $L/B = 80/40 = 2$; $C_p = -0.3$)

Surface 4: $p = -6.0 \pm 1.8$ psf (side)

Surface 5: $p = +6.9 \pm 1.8$ psf (windward)

Surface 6: $p = -6.0 \pm 1.8$ psf (side)

Roof pressures:

Roof C_p from Figure 6-3 of the Standard are shown in Table 3-15:

Table 3-15 Roof C_p* for Wind Direction A

Surface	*7*	*8*			*9*	*10*	*11*	*12*
C_p	−0.5 0.0*	Horiz. distance from windward edge			Same as surface 8	−0.5 −0.5*	−0.5 0.0*	Same as surface 8
		1 to 12.1 ft	12.1 to 24.2 ft	24.2 ft to end				
		−0.9 −0.18*	−0.5 −0.18*	−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 here.

Roof pressures calculation:

Surface 7: $p = 10.1(0.85)(-0.5) - 10.1(\pm 0.18) = -4.3 \pm 1.8$ psf (windward)

Surface 8: for $\theta = 0°$; pressure varies along the roof

$p = 10.1(0.85)(-0.9) - 10.1(\pm 0.18) = -7.7 \pm 1.8$ psf; 1 to 12.1 ft

$p = 10.1(0.85)(-0.5) - 10.1(\pm 0.18) = -4.3 \pm 1.8$ psf; 12.1 to 24.2 ft

$p = 10.1(0.85)(-0.3) - 10.1(\pm 0.18) = -2.6 \pm 1.8$ psf; 24.2 ft to end

Surface 9: Same pressures as surface 8

Surface 10: $p = 10.1(0.85)(-0.5) - 10.1(\pm 0.18) = -4.3 \pm 1.8$ psf (leeward)

Surface 11: $p = 10.1(0.85)(-0.5) - 10.1(\pm 0.18) = -4.3 \pm 1.8$ (windward)

Surface 12: Same as surface 8 without internal pressure

Overhang pressures:
At wall surfaces 1 and 5:

$p = 10.1\ (0.85)\ (0.8) = +6.9$ psf

Internal pressure is of the same sign on all applicable surfaces.

Wind Direction B

Wall pressures:

Surface 1: $p = -6.0 \pm 1.8$ psf (side)

Surface 2: $p = +6.9 \pm 1.8$ psf (windward)

Surface 3: $p = -6.0 \pm 1.8$ psf (side)

Surface 4: $p = 10.1(0.85)(-0.5) - 10.1(\pm 0.18) = -4.3 \pm 1.8$ psf (leeward) (for $L/B = 40/80 = 0.5$; $C_p = -0.5$)

Surface 5: Even though technically this surface is side wall, it is likely to see the same pressure as surface 6

Surface 6: Same pressure as surface 4

Roof pressures:

$h/L = 12.1/40 = 0.3$; $\theta = 15°$

Roof C_p from Figure 6-6 of the Standard are tabulated in Table 3-16:

For windward, $C_p = -0.54$ (interpolated between –0.5 and –0.7)

For leeward, $C_p = -0.5$

For parallel to ridge:

$C_p = -0.9$; 1 to 12.1 ft

$C_p = -0.5$; 12.1 to 24.2 ft

$C_p = -0.3$; 24.2 ft to end

Table 3-16 Roof C_p for Wind Direction B

Surface	*7*	*8*	*9*	*10*	*11*	*12*
C_p	Same as surface 8 for Wind Direction A	−0.54*	−0.5	Same as surface 8 for Wind Direction A	Same as surface 9	−0.3

* Value is gained by interpolation.

Roof pressures calculation:

Surface 7: Same pressures as surface 8 for Wind Direction A

Surface 8: $p = 10.1(0.85)(-0.54) - 10.1(\pm 0.18) = -4.6 \pm 1.8$ psf (windward)

Surface 9: $p = 10.1(0.85)(-0.5) - 10.1(\pm 0.18) = -4.3 \pm 1.8$ psf (leeward)

Surface 10: Same pressures as surface 8 for Wind Direction A

Surface 11: Same as surface 9 because it is sloping with respect to ridge

Surface 12: This surface is at a distance greater than $2h$

$p = 10.1(0.85)(-0.3) = -2.6$ psf; no internal pressure

Overhang pressures:

At wall surface 2:

$p = 10.1\ (0.85)\ (0.8) = +6.9$ psf

Internal pressure is of the same sign on all applicable surfaces.

Wind Direction C

Wall pressures:

Surfaces 1 and 5: $p = -2.6 \pm 1.8$ psf (leeward)

Surfaces 2, 4, and 6: $p = -6.0 \pm 1.8$ psf (side)

Surface 3: $p = +6.9 \pm 1.8$ psf (windward)

Roof pressures:

Surfaces 7 and 11: $p = -4.3 \pm 1.8$ psf (leeward)

Surfaces 8 and 9: Pressures vary along the roof; same pressures as surface 8 for Wind Direction A

Surface 10: $p = -4.3 \pm 1.8$ psf (windward)

Surface 12: Same pressures as surface 9 without internal pressures

Overhang pressures:
At wall surface 3

$p = 10.1\ (0.85)\ (0.8) = +6.9$ psf

Internal pressure is of the same sign on all applicable surfaces.

Wind Direction D

Wall pressures:

Surfaces 1 and 3: $p = -6.0 \pm 1.8$ psf (side)

Surface 2: $p = -4.3 \pm 1.8$ psf (leeward)

Surfaces 4, 5, and 6: $p = +6.9 \pm 1.8$ psf (windward)

Roof pressures:

Surfaces 7, 10, and 11: Pressures vary along the roof; same pressures as surface 8 for Wind Direction A

Surface 8: $p = -4.3 \pm 1.8$ psf (leeward)

Surface 9: $p = -4.3 \pm 1.8$ psf (windward)

Surface 12: This surface will see pressures on top and bottom surfaces; they will add algebraically.

For $\theta = 0°$, $h/L < 0.5$, $C_p = -0.9$ (Load Case 1)

$p = 10.1(0.85)(-0.9) - 10.1(0.85)\ (+0.8) = -14.6$ psf uplift

Overhang pressures:

At wall surfaces 4, 5, and 6:

$p = 10.1\ (0.85)\ (0.8) = +6.9$ psf

Internal pressure is of the same sign on all applicable surfaces.

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 Fig. 6-9. Case 1 includes the loadings analyzed above. A combination of windward (P_W) and leeward (P_L) loads is applied for other Load Cases. This building has mean roof height h of less than 30 ft; hence it comes under exception specified in Section 6.5.12.3 of the standard. Only Load Cases 1 and 3 shown in Figure 6-9 of the Standard have to be considered.

The following two points need to be highlighted:

1. Because of asymmetry, all four wind directions are considered when combining wind loads according to Figure 6-9 of the Standard. For example, when combining wind loads in Case 3, there are four kinds of combinations of wind loads that need to be considered, which are shown as in Figure 3-12.
2. Due to the slightness of the roof slope, the wind load acted on the roof is negligible here.

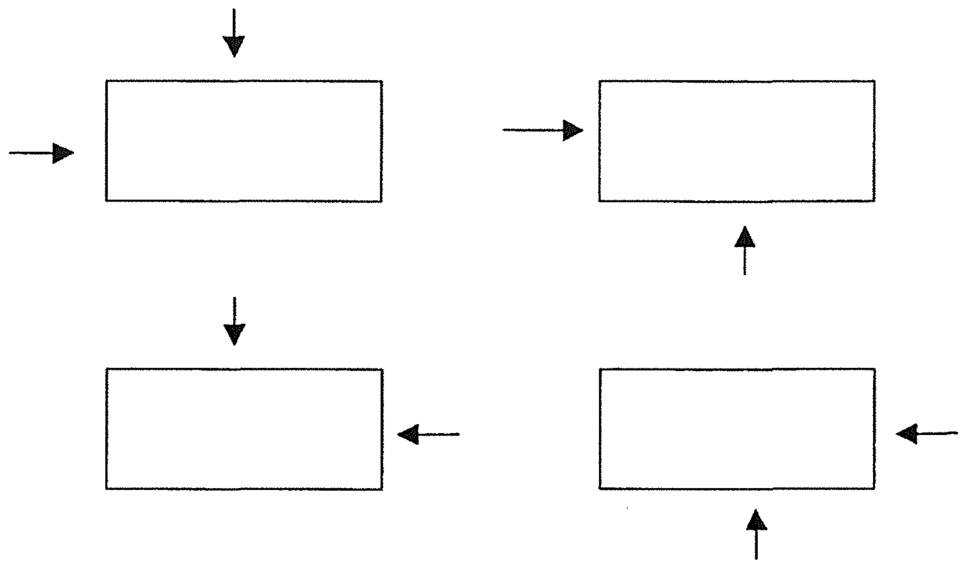


Figure 3-12 Combinations of Wind Loads

Note: Arrows show the wind directions.

Design Pressures for C&C

Wall Component

Wall studs are 10-ft long and spaced 16 in. apart.

Effective area:

larger of

$10 \times 1.33 = 13.3 \text{ ft}^2$

or

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

From Figure 6-11A of the Standard, equations in Chapter 2 of this guide are used:

$(GC_p) = +0.91$ for Zones 4 and 5

$(GC_p) = -1.01$ for Zone 4

$(GC_p) = -1.22$ for Zone 5

Distance "a":

smaller of

0.1 (40) = 4 ft (controls)

or

0.4 (12.1) = 4.8 ft

Design pressure:

$p = 12.3\ (0.91 + 0.18) = +13.4$ psf (all walls)

$p = 12.3\ (-1.01 - 0.18) = -14.6$ psf (middle)

$p = 12.3\ (-1.22 - 0.18) = -17.2$ psf (corner)