

Wind Loads

**Guide to the Wind Load
Provisions of ASCE 7-10**

Kishor C. Mehta, Ph.D., P.E.
William L. Coulbourne, P.E.



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Other Titles of Interest

Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-10. (ASCE Standard, 2010). Provides requirements for general structural design and includes means for determining various load and their combinations. Includes commentary. (ISBN 9780784410851)

Significant Changes to the Wind Load Provisions of ASCE 7-10: An Illustrated Guide, by T. Eric Stafford. (ASCE Press, 2011). Summarizes changes to the wind load requirements set forth in Standard ASCE/SEI 7-10. (ISBN 9780784411162)

Significant Changes to the Seismic Load Provisions of ASCE 7-10: An Illustrated Guide, by S. K. Ghosh, Susan Dowty, and Prabuddha Dasgupta. (ASCE Press, 2011). Describes the revisions to the seismic requirements set forth in Standard ASCE/SEI 7-10. (ISBN 9780784411179)

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Urban Aerodynamics: Wind Engineering for Urban Planners and Designers, by the Task Committee on Urban Aerodynamics. (Technical Report, 2011). Introduces the basic tools and technology used by engineers to determine the effects of wind on city streets and structures. (ISBN 9780784411797)

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Preface

This guide is designed to assist professionals in the use of the wind load provisions of *Minimum Design Loads for Buildings and Other Structures*, Standard ASCE/SEI 7-10, published by the American Society of Civil Engineers (ASCE). The guide is a revision of *Wind Loads: Guide to the Wind Load Provisions of ASCE 7-05*, reflecting the significant changes made to wind load provisions from the previous version of the Standard, ASCE/SEI 7-05. The guide contains 13 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 of the American Society of Civil Engineers.

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Unit Conversions

Measurement	S.I. Units	Customary units
Abbreviations	m = meter (S.I. base unit of length) cm = centimeter km = kilometer ha = hectare L = liter (S.I. base unit of volume) mL = milliliter kg = kilogram (S.I. base unit of mass) g = gram N = Newton ($\text{m} \cdot \text{kg} \cdot \text{s}^{-2}$) Pa = Pascal (N/m^2) kPa = kilopascal J = Joule W = watt kW = kilowatt s = second (S.I. base unit of time) min = minute h = hour day °C = degrees Celsius ppm = parts per million	yd = yard ft = foot in. = inch mi = mile acre gal = gallon qt = quart lb = pound oz = ounce plf = lbs per foot lbf = pound-force (lb/ft) psi = pounds per square inch atm = atmosphere ft·lbf = feet per pound-force Btu = British thermal unit hp = horsepower s = second min = minute h = hour day °F = degrees Fahrenheit ppm = parts per million
Length	1 m = 3.2808 ft = 1.0936 yd 1 cm = 0.3937 in. 1 km = 0.6214 mile	1 ft = 3 yd = 0.3048 m 1 in. = 2.54 cm 1 mile = 0.869 nautical mile = 1.6093 km
Area	1 m ² = 10.7643 ft ² 1 km ² = 0.3861 mi ² 1 ha = 2.4710 acre	1 ft ² = 0.0929 m ² 1 mi ² = 2.59 km ² 1 acre = 43,560 ft ² = 0.4047 ha
Volume	1 L = 0.2642 gal 1 mL = 1 cm ³	1 gal = 4 qt = 3.7854 L 1 ft ³ = 7.481 gal = 28.32 L
Mass	1 g = 0.0353 oz 1 kg = 2.2046 lb	1 oz = 28.3495 g 1 lb = 0.4536 kg
Force	1 N = 0.2248 lb/ft	1 lbf = 4.4482 N
Density	1 kg/m ³ = 0.2048 lb/ft ³ 1 kg/m ³ = 6.2427 lb/ft ³	1 lb/ft ³ = 4.882 kg/m ³ 1 lb/ft ³ = 16.018 kg/m ³
Pressure	1 kPa = 0.145 psi	1 psi = 6.8948 kPa 1 atm = 14.7 psi = 101.35 kPa
Energy and Power	1 J = 1.00 W·s = 0.7376 ft·lbf 1 kJ = 0.2778 W·h = 0.948 Btu 1 W = 0.7376 ft·lbf/s = 3.4122 Btu/h 1 kW = 1.3410 hp	1 ft·lbf = 1.3558 J 1 Btu = 1.0551 kJ 1 ft·lbf/s = 1.3558 W 1 hp = 550 ft·lb/s = 0.7457 kW
Flow	1 L/s = 15.85 gal/min = 2.119 ft ³ /min	1 gal/min = 0.1337 ft ³ /min = 0.0631 L/s
Concentration	mg/L = ppm _m (in dilute solutions)	
Temperature	°C = (°F - 32) × 5/9	°F = (°C × 9/5) + 32
Fundamental	Acceleration of gravity	32.2 ft/s ² = 9.81 m/s ²
Constants and	Density of water (at 4 °C) =	1,000 kg/m ³ = 1 g/cm ³
Relationships	Specific weight of water (15 °C) =	62.4 lb/ft ³ = 9,810 N/m ³
	Weight of water	1 gal = 8.345 lb = 3.7854 kg

Chapter 1

Introduction

The American Society of Civil Engineers (ASCE) publication, ASCE/SEI Standard 7-10, *Minimum Design Loads for Buildings and Other Structures*, is a consensus standard. It originated in 1972 when the American National Standards Institute (ANSI) published a standard with the same title (ANSI A58.1-1972). That 1972 standard was revised 10 years later, containing an innovative approach to wind loads for components and cladding (C&C) of buildings (ANSI A58.1-1982). Wind load criteria were based on the understanding of aerodynamics of wind pressures in building corners, eaves, and ridge areas, as well as the effects of area averaging on pressures.

In the mid-1980s, ASCE assumed responsibility for the Minimum Design Loads for Buildings and Other Structures Standards Committee, which establishes design loads. The document published by ASCE (ASCE 7-88) contained design load criteria for live loads, snow loads, wind loads, earthquake loads, and other environmental loads, as well as load combinations. The ASCE 7 Standards Committee consists of voting membership representing all aspects of the building construction industry. The criteria for each of the environmental loads are developed by respective subcommittees.

The wind load criteria of ASCE 7-88 (ASCE, 1990) were essentially the same as ANSI A58.1-1982. In 1996, ASCE published ASCE 7-95 (ASCE, 1996). This version contained major changes in wind load criteria: the basic wind speed averaging time was changed from fastest-mile to 3-second gust. This in turn necessitated significant changes in boundary-layer profile parameters, gust effect factor, and some pressure coefficients. A *Guide to the Use of the Wind Load Provisions of ASCE 7-95* (Mehta and Marshall, 1997) was published by ASCE to assist practicing professionals in the use of wind load provisions of ASCE 7-95.

In 2001, ASCE published a revision of ASCE 7-95 with updated wind load provisions. The document is termed ASCE 7-98 and has the same title (ASCE, 2001). The International Building Code (ICC 2000) adopted the wind load criteria of ASCE 7-98 by reference. This was a major milestone 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* (Kishor and Perry, 2001) was published soon after publication of ASCE 7-98. After each revision of the ASCE/SEI standard in 2002 and 2005 *Guide to the Use of the Wind*

Load Provisions are published by ASCE (Mehta and Delahay, 2003, Mehta and Coulbourne, 2010).

A revised standard, ASCE/SEI 7-10, was published by ASCE (ASCE, 2010). This version of the standard contains significant changes in wind speed maps, load factors, and format of the wind load provisions. This document, *Wind Loads: Guide to the Wind Load Provisions of ASCE 7-10*, contains explanations and guidance to the changes in the wind load provisions. Two items in the previous guides were well received by practitioners: Examples and Frequently Asked Questions; these items are revised and retained in this updated guide.

1.1 Objective of the Guide

The objective of this guide is to provide direction in the use of wind load provisions of ASCE 7-10 (referred to as “the Standard”). The Commentary of ASCE 7-10 (chapters C26 through C31) contains a good background and discussion of the wind load criteria; that information is not repeated in this document.

Chapters 4 through 13 of this guide contain 14 worked examples. Various examples illustrate different methods of obtaining wind loads given in the Standard. Sufficient details of calculation of wind loads are provided to help the reader properly interpret the wind load provisions of the Standard. Sections of the Standard, as well as the figures and tables of the Standard, are cited liberally in the examples. The equation numbers given in the examples are from the Standard to allow users to track steps of the Standard. *It is necessary to have a copy of ASCE 7-10 to follow the examples and work with this Guide.* A copy of ASCE 7-10 can be ordered by calling 1-800-548-ASCE or ordered on-line at <http://www.asce.org/bookstore>.

1.2 Significant Changes and Additions

The wind load provisions of ASCE 7-10 appear completely different from the previous versions of the Standard because of a major change in the format. Wind load provisions contained in one chapter (chapter 6) in previous versions are expanded into chapters 26 through 31. This expansion is designed to make provisions more user-friendly. The provisions are organized by the type of building or structure under consideration, and equations and tables are repeated to provide all necessary items in one location or chapter.

In addition to format, other significant changes include wind speed maps that are related to limit state loads, an addition of a simplified procedure for enclosed buildings with roof height equal to or less than 160 ft, and clarifications/modifications of exposure categories, debris zones, and minimum loads. The basic approach to assessing wind loading has not changed. Major changes in format are listed as follows by each chapter.

Chapter 26 contains general requirements for wind load determination. General requirements for all buildings and structures include wind speed, wind directionality, exposure category, topographic effect, gust effect factor, enclosure classification, wind-borne debris regions, internal pressure coefficient, symbols, and definitions. Decisions regarding these requirements can be made prior to obtaining wind loads for surfaces of buildings and structures.

Chapter 27 contains wind load criteria for the main wind force-resisting system (MWFRS) of buildings using the directional approach. This approach is the traditional approach used since ANSI A58.1-1972. Wind loads criteria in Part 1 are applicable to enclosed, partially enclosed, or open buildings of any height. Criteria of Part 1 are necessary if wind loads are to be determined for windward, leeward, and side walls and roof including internal pressures for the MWFRS. Part 2 is a new simplified procedure for buildings with roof height equal to or less than 160 ft; the procedure is restricted to enclosed buildings with simple diaphragms. Simplification in Part 2 constitutes a tabular form of pressure values. There are other restrictions for use of the simplified procedure of Part 2; these are shown in **Chapter 2** of this guide.

Chapter 28 contains wind loads criteria for the MWFRS of low-rise buildings (envelope approach for buildings with roof height h less than or equal to 60 ft). Part 1 of the chapter gives equations for velocity pressures and design pressures for windward, leeward, and side walls and roof of the building. Part 2 is a simplified procedure in which horizontal and vertical design pressures are given in tabular form.

Chapter 29 contains wind loads criteria for the MWFRS of other structures and building appurtenances. Structures include chimneys, signs, walls, towers, and others. Building appurtenances are limited to rooftop equipment. Wind loads on parapets and overhangs are referred to in appropriate sections in other chapters. This cross referencing of sections where loading criteria can be found is designed to make the standard more user-friendly.

Chapter 30 contains wind loads criteria for components and cladding (C&C). Because wind loads on C&C of different buildings are given in various formats, the chapter is divided into six parts:

- Part 1 is applicable to low-rise buildings.
- Part 2 is a simplified approach for low-rise buildings.
- Part 3 is applicable to buildings of any height.
- Part 4 is a simplified approach and is applicable to buildings with roof height equal to or less than 160 ft.
- Part 5 is applicable to open buildings.
- Part 6 is applicable to building appurtenances, such as rooftop equipment, roof overhangs, and parapets.

Chapter 31 contains the criteria for the wind tunnel procedure. The criteria include wind tunnel test conditions and limitations on wind loads.

As noted, the basic methodology of the Standard remains the same as in ASCE 7-05. Additional significant changes in the standard are listed as follows.

- Three wind speed maps are given. Each map is related to the risk category of building specified in Table 1.5-1 of the Standard. Spec-

fication of three wind speed maps eliminates the need for Importance Factors used in previous versions of the Standard.

- Wind Speeds in the maps are related to limit state strength design. The load factors in load combinations of Sections 2.3 and 2.4 of the Standard are associated with these wind speeds.
- State of Hawaii is designated as a special wind region; basic wind speed will be in accordance with local jurisdictions.
- Exposure Category D is applicable to water surfaces including hurricane prone regions.
- New simplified procedures to obtain wind loads for MWFRS and C&C of enclosed buildings with roof height equal to or less than 160 ft are added. These procedures provide wind loads in tabular form.
- Provisions for calculations of natural frequency for building frames to determine gust effect factor are added.
- Minimum design loads for MWFRS are revised; horizontal loads on wall surfaces are increased to 16 psf to make them consistent with strength design. On vertical projection of roof, horizontal loads are specified as one-half of minimum loads on walls.
- Wind-borne debris regions are revised to relate to building risk categories.
- Lower limits of wind loads obtained from wind tunnel procedures are specified in the Standard.

The aforementioned changes are reflected in the example problems of this guide. A new chapter 3 is added to this guide to explain wind speed maps as they relate to risk categories of buildings and structures and load factors in load combinations.

1.3 Limitations of the Standard

Successful use of the Standard is dependent on 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 given as follows.

Assessment of Wind Climate

The current edition of the Standard provides a more realistic description of wind speeds than did the previous editions. 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, and river valleys. Unusual winds may be encountered in these regions because of topographic effects or because of the channeling of wind. The Standard permits climatological studies using regional climatic data and consultation with a wind engineer or a meteorologist.

Tornado winds are not included in development of the basic wind speed maps (Fig. 26.5-1 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 150–200 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.

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 can be 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 could be prone to damaging vortex excitation. If across-wind or torsional excitation appears to be a possibility, expert advice should be obtained.

Even though the standard does not specify limiting values for serviceability, a discussion of serviceability is given in the Commentary Appendix C of the Standard. The user of the Standard is encouraged to review this appendix to obtain guidelines for drift of walls and frames and vibrations.

Limitations in Shapes of Buildings and Other Structures

The pressure and force coefficients given in the Standard are limited. Many of the building shapes (e.g., “Y,” “T,” and “L” shapes) or buildings with stepped elevations are not included (except as shown in Fig. 30.4-3). Fortunately, this information may be found in other sources (see Table G1-1).

When coefficients for a specific shape are not given in the Standard, the designer is encouraged to use values through interpretation of the intent of the Standard (see Chapter 8 for L-shape, Chapter 9 for U-shape, and Chapter 12 for oddly shaped buildings). Pressure coefficients that are available in the literature also can be used. However, the use of prudent judgment is advised, and the following caveats should be addressed:

1. The coefficients should be obtained from proper turbulent boundary layer wind tunnel (BLWT) tests.
2. The averaging time used should 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 .

Table G1-1**Technical Literature**

<i>Subject</i>	<i>Selected Reference Material (see References Section of this guide)</i>
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); Dyrbye and Hansen (1997); Holmes (2007)
Codes and standards	NRCC (2010); British Standard BS 6399 (1997); Eurocode 1 (2005); ISO (1997); Australian/New Zealand Standard AS/NZS 1170.2 (2002); Stafford (2010)
Wind tunnel testing	Reinhold (1982); ASCE (2012)
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); Coulbourne, Tezak, and McAllister (2002); FEMA 320 (2008); FEMA 361 (2008); ICC 500 (2008)
Impact resistance protocol	SBCCI (1999); ASTM E1886-05 (2005), ASTM E1996-09 (2009); Miami/Dade County Building Code Compliance Office Protocol PA 201-94 and PA 203-94 (1994)

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 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 four 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 13), *Proceedings of the Americas Conferences on Wind Engineering* (11), *Proceedings of the Asia-Pacific Conferences on Wind Engineering* (6) and *Proceedings of the European-African Conferences on Wind*

Engineering (5). 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 G1-1**. The items are listed by subject matter for easy identification. Detailed references for some of these items are given in the citations in References of this guide.

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Chapter 2

Wind Load Provisions

2.1 Format

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

1. Analytical procedure,
2. Simplified procedure, and
3. Wind tunnel procedure.

The analytical procedure can be used for any and all buildings and structures, whereas the simplified procedure is restricted to certain types of buildings, as well as other restrictive requirements. The wind tunnel procedure can be used for any building or structure, though because of its cost and extra time required to perform the customized tests, it is used for buildings or structures of unusual shape or when wind loads are critical.

One of the key elements in wind loading is the method by which pressure and force coefficients, which are specified in the Standard, are obtained through wind tunnel tests. The Standard provides separate wind loading criteria for two analytical procedures; the “Directional Procedure” and “Envelope Procedure.” The directional procedure provides specification of pressure and force coefficients associated with specific directions (e.g., windward wall, leeward wall, and others). The envelope procedure provides pressure coefficients that are independent of wind direction. Additional explanations of these two procedures are given subsequently.

2.2 Velocity Pressure

Irrespective of the procedure used to determine wind loads for buildings or structures, it is necessary to determine velocity pressure, q . Velocity pressure depends on wind speed, surrounding terrain including topography, and

probabilities of directionality and occurrence of wind speed. In the Standard, the equation for velocity pressure q is given as

$$q = 0.00256 K_z K_{zt} K_d V^2 \text{ (lb/ft}^2\text{)}$$

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 26.7)
- K_{zt} = topographic factor that accounts for wind speed-up over hills and escarpments (see Section 26.8.2 and Fig. 26.8-1)
- K_d = directionality factor (see Section 26.6 and Table 26.6-1)
- 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 appropriate for the risk category related to building use (see Table 1.5-1, Section 26.5, and Figs. 26.5-1A, 26.5-1B, or 26.5-1C)

Since q is to be determined for any procedure for all buildings and structures, the equation is repeated in each chapter (Eqs. 27.3-1, 28.3-1, 29.3-1, and 30.3-1).

2.3 Analytical Procedure

The analytical procedure can be used for any building or structure. The wind loads obtained using the directional procedure or the envelope procedure will depend on the pressure and force coefficients specified in the standard. The new format of the standard is made user-friendly by specifying clearly what type of structures and what loads are obtained with the directional or envelope procedure.

Directional Procedure

As indicated previously, the wind loads obtained with the directional procedure are related to a specific direction of wind. Pressure coefficients specified in the Standard are associated with specific directions since they are measured in the wind tunnel for wind flow in that direction.

The design wind pressure for the MWFRS of enclosed and partially enclosed buildings of any height is determined by the following equation:

$$p = qGC_p - q_i (GC_{pi}) \text{ (lb/ft}^2\text{)} \quad (\text{Eq. 27.4-1})$$

where

- $q_i = q_z$ evaluated at height z or q_h evaluated at roof height h . For appropriate use of q_z or q_h , see Section 27.4.1

G = gust effect factor; it can be 0.85 for rigid buildings and G_f for flexible buildings (Section 26.9)

C_p = external pressure coefficient from Figs. 27.4-1, 27.4-2, and 27.4-3

GC_{pi} = internal pressure coefficient from Table 26.11-1

The terms in Eq. 2-2 are defined as aforementioned. The effective velocity pressure related to internal pressure, q_i , is generally used as q_h (see Section 27.4.1 of the Standard). Only for high-rise buildings may it be advantageous to use q_i as defined in Section 27.4.1 related to positive internal pressure. Use of this term is illustrated in **Section 4.1** of this guide.

Similar equations are given for determining wind loads on MWFRS of open buildings with monoslope and other shape roofs, parapets of buildings, as well as for other structures. In these cases the second term involving internal pressure is not given because internal pressure occurs only in enclosed and partially enclosed buildings.

Key elements for application of loads to keep in mind are as follows:

- Wind load sign is positive (+) when acting toward a surface and negative (–) when acting away from the surface.
- On building surfaces, pressures act normal to the surface unless otherwise noted differently on a sketch.
- On structures, forces act in the direction of the wind unless shown differently.
- Internal pressures act inward or outward on all surfaces; inward and outward pressures are not to be combined.

Envelope Procedure for MWFRS

There is a special provision for MWFRS of low-rise buildings in which the wind loads are obtained through the envelope procedure. A low-rise building is defined as a building with mean roof height h less than or equal to 60 ft, and the height h does not exceed the least horizontal dimension. The design wind pressures for MWFRS of low-rise buildings are determined using the equation

$$P = q_h [(GC_{pf}) - (GC_{pi})] \text{ (lb/ft}^2\text{)} \quad (\text{Eq. 28.4-1})$$

where

GC_{pf} = product of the equivalent external pressure coefficient and gust effect factor from Fig. 2-3

Other terms in the equation are defined previously. The term GC_{pf} represents “pseudo” pressure coefficients, not pressure coefficients measured in the wind tunnel. The wind tunnel tests determined internal reactions, such as total uplift, total horizontal shear, bending moments, among others, for frames of the shape shown in Fig. 28.4-1 for wind blowing from all directions. Enveloping pressure coefficients were determined for maximum values of internal reactions. This procedure provided simple coefficients that yield design loads in a convenient way. However, it should be recognized that pressure coefficients are valid when pressures are applied to all surfaces at the

same time. In addition, this procedure is difficult to interpret when the shape of the building differs significantly from the rectangular shape.

Since coefficients used in the directional approach and the envelope approach were obtained through two totally different ways, the final loads obtained from the two approaches do not match. It is not possible to reconcile the differences between the final loads obtained by the two procedures. The Standard permits use of either procedure for buildings with height h less than 60 ft.

Procedure for Components and Cladding

Design pressures for components and cladding (C&C) are obtained using equations similar to the equations used for MWFRS. Effective velocity pressure is obtained using the equation shown in **Section 2.2** of this guide. Effective velocity pressure is multiplied with the combination of the appropriate gust effect factor and external pressure coefficient and the appropriate internal pressure coefficient. When combined values of GC_p are given, they should not be separated.

Pressure coefficients for C&C are obtained as in the envelope procedure. The building or structure is rotated in the wind tunnel, and the maximum value of the pressure is converted to a pressure coefficient. This method of obtaining pressure coefficients eliminates consideration of direction of approach of wind. Terrain surrounding a building or structure that gives the highest value of load governs the wind load for C&C.

Because wind loads fluctuate over space at any given time, pressure coefficients for C&C are dependent on effective area of a component. The larger the effective area of a component, the smaller the average wind load. This dependence on effective area is reflected in the values of pressure coefficients given in chapter 30 of the Standard. Examples given in **chapters 4 through 13** of this guide illustrate appropriate use of effective areas of C&C.

2.4 Simplified Procedure

To help practicing professionals obtain wind loads quickly, simplified procedures are given in the Standard for two types of buildings:

1. Buildings with roof height h equal to or less than 160 ft and
2. Low-rise buildings. Simplification is achieved by providing wind loads in a tabular form for MWFRS and for C & C. There are limitations for which precalculated loads in tables can be used; these limitations are provided in the Standard.

Buildings with Roof Height Equal to or Less Than 160 ft

Main Wind Force-Resisting System (MWFRS)

MWFRS wind loads are given in chapter 27, part 2. This procedure is restricted to enclosed buildings with simple diaphragms. The definition of

simple diaphragm is given in Section 26.2. The concept of a simple diaphragm is that wind loads on windward and leeward walls are transmitted through floor and roof diaphragms. Some of the other conditions required for use of this simplified procedure are given in Section 27.5.2. Horizontal net wind loads at the top and bottom of the buildings are given in Table 27.6-1 as a function of wind speed, height of the building, and aspect ratio of the building. The pressures given in the table are combined pressures of windward and leeward walls. It is possible to divide pressures into windward and leeward wall pressures using Note 4 in Table 27.6-1. Tables of wind pressures are given for Exposure B, C, and D. Internal pressures are not included in tabulated wall pressure values because they cancel out. It is assumed that wind pressures vary linearly along the height of the building. This assumption of linear variation along with other approximations yields pressures by this simplified procedure that do not match the pressures obtained with the analytical procedure given in chapter 27, part 1. However, the total shear at the base is close, though overturning moment at the base does not match; see Sections 4.1 and 4.3 in this guide. The designer needs to make a judgment on the importance of these differences.

Roof pressures for MWFRS using this simplified procedure are given in Table 27.6-2. The values given in the table are for Exposure C; they should be multiplied by factors in Table 27.6-2 for Exposures B and D. Tabulated values in Table 27.6-2 are listed as a function of wind speed, zone of the roof, and roof height and slope. Zones on various shapes of roofs are shown in the sketches in Table 27.6-2.

Components and Cladding (C&C)

Wind pressures for C&C of walls and roofs are determined using tabulated values in Table 30.7-2. The simplified procedure is restricted to an enclosed building. Pressures in Table 30.7-2 are a function of wind speed, building height, roof shape, and zone of wall and roof. The tabulated values are for Exposure C and for an effective wind area of 10 ft². The equation for determining design wind pressures is

$$p = p_{\text{table}} (\text{EAF}) (\text{RF}) K_{zt} \text{ (psf)} \quad (\text{Eq. 30.7-1})$$

where

EAF = exposure adjustment factor from Table 30.7-2

RF = effective area reduction factor from Table 30.7-2

K_{zt} = topographic factor given in Section 26.8 (1.0 when not needed)

Zone designations on wall and roof surfaces are given in sketches in Table 30.7-2. The effective area reduction factor, RF, can be determined from a graph and table of Table 30.7-2. The tabulated values of pressures include internal pressure for an enclosed building with the appropriate sign that generates maximum pressure values. (Note: internal pressures are plus and minus; both of them are individually applicable.)

Pressures obtained for C&C using this simplified procedure match well with the pressures obtained using the analytical procedure except for round-

off errors. Concepts and the values used in simplified and analytical procedures are the same, so they should provide similar answers.

Low-Rise Buildings with Height h Equal to or Less Than 60 ft

Main Wind Force-Resisting System

This simplified procedure for MWFRS is applicable to a special class of low-rise buildings with simple diaphragms. The procedure to determine wind loads is given in chapter 28, part 2. The building has to be classified as an enclosed building. It is required to be a regular shape and has approximately a symmetrical cross section. Tabulated pressures given in Fig. 28.6-1 are horizontal and vertical pressures; a sketch showing direction of pressures as well as different surface zones is given in Fig. 28.6-1. The tabulated pressures, p_{s30} , are at 30 ft above ground for Exposure B. Pressures in the table of Fig. 28.6-1 are related to wind speed, slope, and zone of roof, as well as zone of wall. Net design pressures p_s are determined by the following equation:

$$p_s = (\lambda) K_{zt} p_{s30} \text{ (psf)} \quad (\text{Eq. 28.6-1})$$

where

p_{s30} = tabulated pressures in Fig. 28.6-1

λ = adjustment factor for building height and exposure, Fig. 28.6-1

K_{zt} = topographic factor given in Section 26.8 (1.0 when not needed)

Horizontal net pressures are combined windward and leeward wall pressures, as well as horizontal components of pressures acting on the roof. Vertical pressures on projected areas of the roof are the vertical component of roof pressures. Internal pressures acting on the walls cancel out. However, internal pressures acting on roof surfaces are added to the horizontal and vertical pressures on projected areas of the roof. The tabulated pressures are derived from the analytical procedure for low-rise buildings, but the final results of simplified and analytical procedures show some differences because of the use of horizontal and vertical components.

Components and Cladding

This simplified procedure is applicable to C&C of low-rise enclosed buildings. The procedure is given in chapter 30, part 1. The tabulated values of pressures given in Fig. 30.5-1 are related to the wind speed, roof slope, zones on roof and wall surfaces, and the effective wind area. The values are for height above ground of 30 ft and Exposure B. The equation to determine net pressures is

$$p_{\text{net}} = (\lambda) K_{zt} p_{\text{net}30} \text{ (psf)} \quad (\text{Eq. 30.5-1})$$

where

$p_{\text{net}30}$ = net pressure from table of Fig. 30.5-1

λ = adjustment factor for building height and exposure, Fig. 30.5-1

K_{zt} = topographic factor given in Section 26.8 (1.0 when not needed)

Pressures act normal to the surface of the building. Net pressure includes internal pressure associated with an enclosed building; sign of internal pressure used gives maximum pressure for design. Tabulated pressures are derived from the analytical procedure for C&C of low-rise buildings given in chapter 30, part 1. The design pressures from the simplified procedure match well with the pressures obtained from the analytical procedure.

2.5 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 31.2 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 Cermak (1977), Reinhold (1982), and ASCE (2012).

2.6 Equations for Graphs

Figures 30.4-1 through 30.4-6 of the Standard give external pressure coefficient, GC_p , 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 G2-1 through G2-10. The equations can be used to determine pressure coefficient values related to effective wind area.

Table G2-1Walls for Buildings with $h \leq 60$ ft (Figure 30.4-1)

Positive: Zones 4 and 5	$(GC_p) = 1.0$	for $A = 10 \text{ ft}^2$
	$(GC_p) = 1.1766 - 0.1766 \log A$	for $10 < A \leq 500 \text{ ft}^2$
	$(GC_p) = 0.7$	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 $10 < A \leq 500 \text{ ft}^2$
	$(GC_p) = -0.8$	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 $10 < A \leq 500 \text{ ft}^2$
	$(GC_p) = -0.8$	for $A > 500 \text{ ft}^2$

Note: Zones are shown in the figures referenced in ASCE 7-10.

Table G2-2Gable Roofs with $h \leq 60$ ft, $\theta \leq 7^\circ$ (Figure 30.4-2A)

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

Note: Zones are shown in the figures referenced in ASCE 7-10.

Table G2-3Gable and Hip Roofs with $h \leq 60$ ft, $7^\circ < \theta \leq 27^\circ$ (Figure 30.4-2B)

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

Note: Zones are shown in the figures referenced in ASCE 7-10.

Table G2-4Gable Roofs with $h \leq 60$ ft, $27^\circ < \theta \leq 45^\circ$ (Figure 30.4-2C)

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

Note: Zones are shown in the figures referenced in ASCE 7-10.

Table G2-5Multispan Gabled Roofs with $h \leq 60$ ft, $10^\circ < \theta \leq 30^\circ$ (Figure 30.4-4)

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

Note: Zones are shown in the figures referenced in ASCE 7-10.

Table G2-6Multispan Gable Roofs with $h \leq 60$ ft, $30^\circ < \theta \leq 45^\circ$ (Figure 30.4-4)

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

Note: Zones are shown in the figures referenced in ASCE 7-10.

Table G2-7Monoslope Roofs with $h \leq 60$ ft, $3^\circ < \theta \leq 10^\circ$ (Figure 30.4-5A)

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

Note: Zones are shown in the figures referenced in ASCE 7-10.

Table G2-8Monoslope Roofs with $h \leq 60$ ft, $10^\circ < \theta \leq 30^\circ$ (Figure 30.4-5B)

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

Note: Zones are shown in the figures referenced in ASCE 7-10.

Table G2-9

Sawtooth Roofs with $h \leq 60$ ft (Figure 30.4-6)

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

Note: Zones are shown in the figures referenced in ASCE 7-10.

Table G2-10Roof and Walls for Buildings with $h > 60$ ft (Figure 30.6-1)

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

Note: Zones are shown in the figures referenced in ASCE 7-10.

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Chapter 3

Wind Speeds and Related Risks

ASCE 7-10 contains three wind speed maps in chapter 26 that look significantly different from the one map that was in ASCE 7-95 through 7-05. The new wind speed maps relate to the risk categories for buildings and structures and load combinations and load factors. These items—wind speed values in Fig. 26.5-1A, B, and C; risk categories in Table 1.5-1; and load combinations in Sections 2.3 and 2.4—are interrelated. The most important aspect for the user of Standard ASCE 7-10 is that the wind loads determined from chapters 26 through 30 are for strength design and are not associated with allowable stress design as in previous editions of the Standard. Load factors in Sections 2.3 and 2.4 of the Standard are adjusted to reflect this aspect. These and other items related to the wind speed maps and associated risks for buildings and structures are discussed in this chapter.

3.1 Wind Speed Maps

Three maps in ASCE 7-10, shown in Fig. 26.5-1A, B, and C, provide wind speed values and contours similar to wind speed maps of the previous versions. The three maps are associated with mean recurrence intervals of 300, 700, and 1,700 years for Risk Category I, II, and III/IV structures, respectively. These mean recurrence intervals are related to limit state loads and strength design. The primary reasons for going to the limit state loads are to provide consistent probability of occurrence of loads in hurricane prone regions and in nonhurricane regions (interior of the country), and to make loads consistent with material standards for steel, concrete, wood, and others.

Occurrence of extreme winds in the hurricane prone regions and interior of the country are because of two unrelated atmospheric phenomena; hur-



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ricanes affect the Gulf Coast, Atlantic Coast, and island areas while thunderstorms affect the rest of the country. Because of these unrelated atmospheric phenomena affecting design wind speeds, it is very difficult to obtain wind speeds that have consistent probabilities related to allowable stress design. In ASCE 7-05 and earlier versions one wind speed map with an importance factor was used. However, it was recognized that it was not appropriate to use one value for the importance factor for hurricane prone areas all along the coastline; it varied up and down the coastal area. Use of three maps reflecting different probabilities of occurrence of wind speed eliminates the use of an importance factor and provides consistent probabilities of wind speeds along the coast.

Determination of annual probability of occurrence (mean recurrence interval, MRI) for the wind speed is based on the provisions of ASCE 7-05. In ASCE 7-05, the wind speed map was associated with a 50-year mean recurrence interval (MRI), and the load factor for wind for strength design was 1.6. To obtain a wind speed V_T that is related to the new MRI associated with limit state design the wind load ratio between ASCE 7-10 and ASCE 7-05 needs to be 1.6. Since load is a function of square of the wind speed, the wind speed ratio needs to be the square root of 1.6. Peterka and Shahid (1998) developed the following equation for the relationship between MRI values and wind speed:

$$V_T/V_{50} = [0.36 + 0.1 \ln(12T)] \quad (\text{Eq. C26.5-2})$$

where

V_T = wind speed associated with MRI of T in years

V_{50} = wind speed associated with MRI of 50 years

T = MRI in years

By substituting the square root of 1.6 for V_T/V_{50} , the value of T is 700 years. The wind speed map in Fig. 26.5-1A for Risk Category II structures is for an MRI of 700 years.

This derivation makes it clear that the wind design using Fig. 26.5-1A of ASCE 7-10 will be the same as the one with the 50-year MRI wind speed map (Fig. 6.1) of ASCE 7-05. Wind speeds in map of Fig. 26.5-1B (1,700 MRI) are equivalent to 100-year MRI wind speeds of ASCE 7-05, and the ones in Fig. 26.5-1C (300 MRI) are equivalent to 25-year MRI of ASCE 7-05.

Hurricane simulation technology continues to evolve. The numerical model is tweaked to match new actual measured wind speed data in hurricanes of recent years. Wind speed contours for hurricane prone areas are updated with improved numerical simulation models. In some coastal areas wind speeds in hurricane areas are lower in ASCE 7-10 than the ones in ASCE 7-05. To compare wind speed values in two maps, it is necessary to divide wind speeds of ASCE 7-10 by the square root of 1.6. As an example, the new wind speed contour in the Keys of Florida in Fig. 26.5-1A of ASCE 7-10 is 180 mph. This value is reduced to 140 mph when divided by the square root of 1.6. In Fig. 6.1 of ASCE 7-05 the wind speed contour for the

Keys of Florida is 150 mph. It is observed that the adjusted hurricane model renders a lower wind speed than the previous one for this location.

3.2 Load Factors

Load factors in Sections 2.3 and 2.4 of ASCE 7-10 are adjusted to reflect increased wind speeds in Figs. 26.5-1A, B, and C. Since the wind speeds are related to limit state loads and strength design, the load factor for wind load W in Section 2.3 for strength design is 1.0. In case of allowable stress design in Section 2.4 the load factor for wind load W is 0.6. With these changes in load factors, parity of design is achieved whether wind speeds of ASCE 7-05 or the ones provided in ASCE 7-10 are used.

3.3 Wind Risks

Buildings and other structures are categorized for risk in ASCE 7-10 and are organized similar to the categories in ASCE 7-05. Risk categories for different occupancies and uses are given in Table 1.5-1. Some revisions are made in identifying buildings and other structures for each category, though the concept is the same as in ASCE 7-05. Risk Category II is the default category. Instead of using an Importance Factor, the appropriate wind speed map is used to obtain wind speed values. For Category II, wind speeds are obtained from Fig. 26.5-1A; Category I wind speeds are in Fig. 26.5-1B, while wind speeds from Fig. 26.5-1C are used for Categories III and IV.

There is always a chance, however small, that wind speed will be exceeded during the life of a building or other structure. The equation for the probability of exceeding any specific wind speed (wind loads) is

$$P_n = 1 - (1 - P_a)^n \quad (\text{Eq. C26.5-7})$$

where

P_n = probability that wind speed will be exceeded during the life of building

P_a = annual probability (inverse of MRI)

n = life of the building or structure

Probabilities of wind speed exceeding at least once during the life of a building or structure are given in **Table G3-1**. For example, for a building in Risk Category II there is a 6.9 percent probability that during the building life of 50 years the wind speed will exceed the basic mapped wind speed. Even though the load is limit state associated with strength design, the building should not collapse because of strength reduction in material standards if structural integrity is maintained and progressive collapse is prevented.

Table G3-1**Probabilities of Exceeding Wind Loads**

<i>Annual probability</i>	<i>MRI in years</i>	<i>Life of a building in years</i>		
		<i>25</i>	<i>50</i>	<i>100</i>
0.00333	300	8.0%	15.4%	28.4%
0.00143	700	3.5%	6.9%	13.3%
0.000588	1,700	1.5%	2.9%	5.7%
0.00010	10,000	0.3%	0.5%	1.0%

Chapter 4

160-ft-Tall Office Building

This building is illustrated in **Fig. G4-1**. Data for the building are listed in **Table G4-1**. Glazing panels are 5 ft wide $\times$ 5 ft 6 in. high (typical); they are wind-borne debris impact resistant in the bottom 60 ft, as required by Section 26.10.3 of the Standard.

The analytical procedure of ASCE 7-10 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 Risk Category II is appropriate (see Table 1.5-1 of the Standard). The wind speed map for this Risk Category is Fig. 26.5-1A.

Basic Wind Speed

Selection of the basic wind speed is addressed in Section 26.5 of the Standard. The vicinity of Houston, Texas, is located on the 140-mph contour. The basic wind speed $V = 140$ mph (see Fig. 26.5-1A of the Standard).

Exposure

The building is located in a suburban area. If suburban terrain prevails for at least $20 \times h = 3,140$ ft in all directions, it is appropriate to use Exposure B (Section 26.7.3).

Velocity Pressures

The velocity pressures are computed using the following equation:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 \text{ psf} \quad (\text{Eq. 27.3-1})$$

where

K_z = value obtained from Table 27.3-1 for MWFRS and from Table 30.3-1 for C&C (see **Table G4-2**)

$K_{zt} = 1.0$ for homogeneous topography



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Fig. G4-1
 Building characteristics
 for 160-ft-tall office
 building

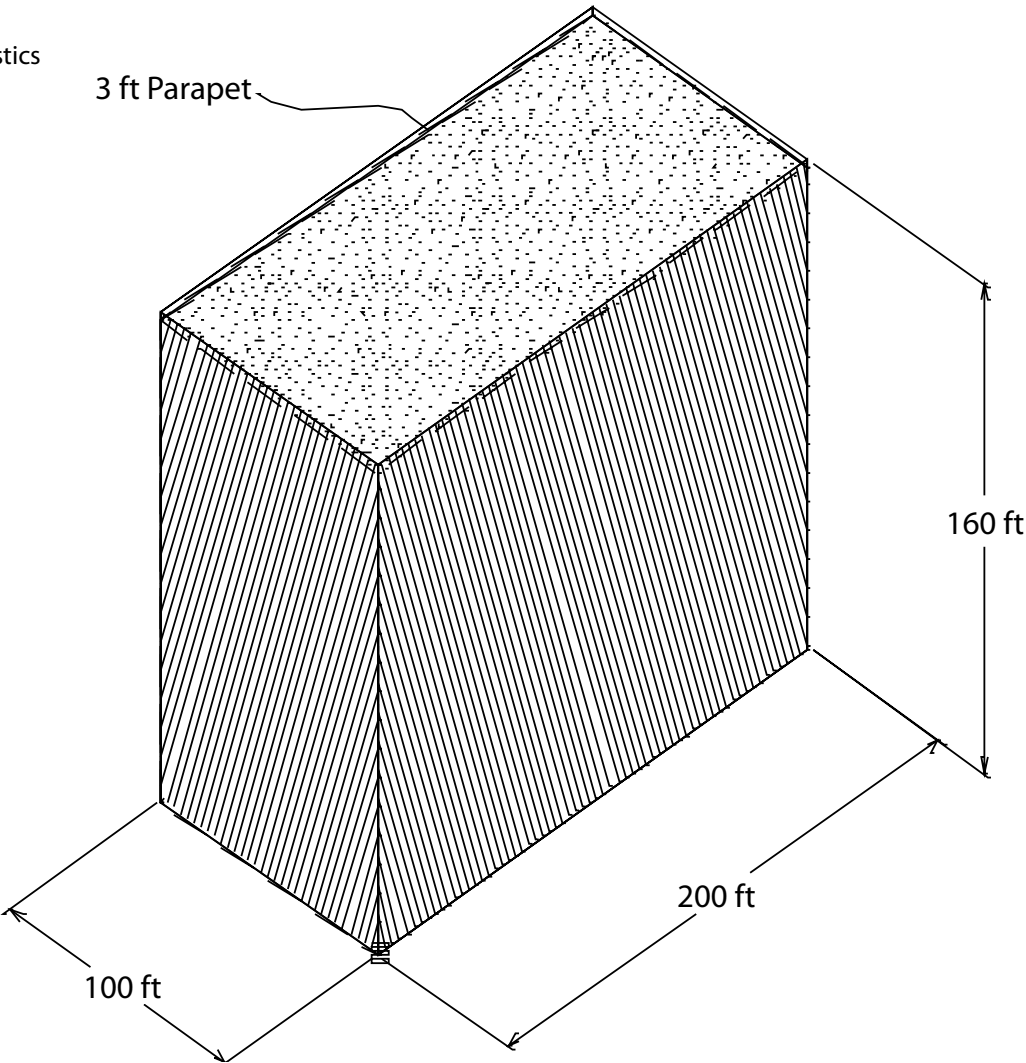


Table G4-1 Data for 160-ft-Tall Office Building

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 assumed to be greater than 1 Hz
Cladding	Mullions for glazing panels span 11 ft between floor slabs Mullion spacing is 5 ft

Table G4-2**Velocity Pressures for 160-ft-Tall Office Building**

Height, ft	MWFRS		C&C	
	K_z	q_z , psf	K_z	q_z , psf
0–15	0.57	24.3	0.70	29.8
30	0.70	29.8	0.70	29.8
50	0.81	34.5	0.81	34.5
80	0.93	39.6	0.93	39.6
120	1.04	44.3	1.04	44.3
Roof = 157	1.12	47.7	1.12	47.7
Parapet = 160	1.13	48.1	1.13	48.1

Note: $q_h = 47.7$ psf.

$K_d = 0.85$ for buildings (see Table 26.6-1 of the Standard)

$V = 140$ mph

Therefore

$$q_z = 0.00256 K_z(1.0)(0.85)(140)^2 = 42.6 K_z \text{ psf}$$

Values for K_z and the resulting velocity pressures are given in **Table G4-2**. The velocity pressure at mean roof height, q_h , is 47.7 psf.

4.1 Analytical Procedure

The design wind pressures for the MWFRS for this building are obtained by equation:

$$p = qGC_p - q_i(GC_{pi}) \quad (\text{Eq. 27.4-1})$$

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. For buildings sited in wind-borne debris regions, glazing that is not impact resistant or protected shall be treated as an opening in accordance with Section 26.10.3. Conservatively q_h can be used.

G = gust effect factor as specified in Section 26.9

C_p = external pressure coefficient

(GC_{pi}) = internal pressure coefficient

Gust Effect Factor

Gust effect factor, G , for buildings and other structures depends on whether a building is flexible or not. In Section 26.2 a flexible building is defined as one with fundamental natural frequency less than 1 Hz (period greater than 1 s). A building can be considered rigid if the fundamental natural frequency is greater than 1 Hz (period less than 1 s).

Fundamental natural frequency depends on the structural system of the building as well as on construction materials. Approximate determination of frequency is given in Section 26.9.3. More discussion for approximate natural frequency is given in commentary Section C26.9 (Eqs. C26.9-6 through C26.9-15). These approximations vary by a factor of 2 or more. Design practices have software that can calculate fundamental frequency more accurately.

For illustration purposes, this example assumes that the natural fundamental frequency is greater than 1 Hz; hence the building is considered a rigid building.

Section 26.9.1 of the Standard permits use of 0.85 for gust effect factor, G , or calculate in accordance with equations in Section 26.9.4. In this example, G is calculated using equations.

$$G = 0.925 \left(\frac{1 + 1.7 g_Q I_{\bar{z}} Q}{1 + 1.7 g_v I_{\bar{z}}} \right) \quad (\text{Eq. 26.9-6})$$

where

$$\begin{aligned} g_Q &= g_v = 3 \quad (\text{given in Section 26.9.4}) \\ \bar{z} &= 0.6(157) = 94.2 \text{ ft (controls)} \quad (\text{Section 26.9.3}) \\ \bar{z} &= z_{\min} = 30 \text{ ft} \quad (\text{Table 26.9-1}) \\ c &= 0.30 \quad (\text{Table 26.9-1}) \end{aligned}$$

Therefore

$$I_{\bar{z}} = \left(\frac{33}{\bar{z}} \right)^{1/6} = 0.30(33/94.2)^{1/6} = 0.25$$

$$L_{\bar{z}} = \ell \left(\frac{\bar{z}}{33} \right)^{1/3} = 320(94.2/33)^{1/3} = 454 \text{ ft}$$

$$Q = \sqrt{\frac{1}{1 + 0.63 \left(\frac{B + b}{L_{\bar{z}}} \right)^{0.63}}}$$

where

$$\begin{aligned} B &= 100 \text{ ft (smaller value gives larger } G) \\ Q &= \sqrt{1/1 + 0.63((100 + 157)/454)^{0.63}} = 0.83 \\ G &= 0.925\{(1 + 1.7 \times 3.4 \times 0.25 \times 0.83)/(1 + 1.7 \times 3.4 \times 0.25)\} = 0.83 \end{aligned}$$

Table G4-3**Wall Pressure Coefficient**

<i>Surface</i>	<i>Wind direction</i>	<i>L/B</i>	<i>C_p</i>
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

Wall External Pressure Coefficients

The values for the external pressure coefficients, C_p , for the various wall surfaces are obtained from Fig. 27.4-1 of the Standard and shown in **Table G4-3**. 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 the 100-ft face, $L/B = 200/100 = 2.0$; therefore, the leeward wall pressure coefficient is −0.3.

Roof Pressure Coefficient with the Wind Normal to the 200-ft face

Roof pressure coefficients, C_p , with the wind normal to the 200-ft face are shown in **Table G4-4**. For $h/L = 157/100 \approx 1.6 > 1.0$, and $\theta < 10^\circ$, two zones are specified in Fig. 27.4-1 of the Standard.

First value

$$\begin{aligned} 0 \text{ to } h/2, C_p &= -1.3 \\ > h/2, C_p &= -0.7 \end{aligned}$$

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.

$$\begin{aligned} \text{Area} &= 200 \times (157/2) = 15,800 \text{ ft}^2 \\ \text{Reduction factor} &= 0.8 \\ \text{Reduced } C_p &= 0.8 \times (-1.3) = -1.04 \end{aligned}$$

Table G4-4**Roof Pressure Coefficient for Wind Normal to 200-ft Face**

<i>Distance from leading edge</i>	<i>C_p</i>
0 to $h/2$	−1.04
$> h/2$	−0.70

Note: $h = 157$ ft.

Table G4-5Roof Pressure Coefficient, 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

Roof Pressure Coefficient with the Wind Normal to the 100-ft face

Roof pressure coefficients, C_p , with the wind normal to the 100-ft face are shown in Table G4-5. For $h/L = 157/200 \times 0.8$, interpolation in Fig. 27.4-1 of the Standard is required.

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

$$\text{External pressure} = 47.7(0.83)(-1.04) = -41.2$$

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

$$\text{External pressure} = 47.7(0.83)(-0.70) = -27.7$$

External pressures are summarized in Tables G4-6 and G4-7.

Internal Pressure Coefficients

The building is in a hurricane prone and wind-borne debris region. The glazing is required to be debris resistant up to 60 ft above ground (Section 26.10.3 of the Standard). In addition, if there is a debris source, such as an aggregate surfaced roof within 1,500 ft of the subject building, glazing that is

Table G4-6

External Pressures for MWFRS for Wind Normal to 200-ft Face

Surface	z (ft)	q (psf)	C_p	External pressure (psf)
Windward wall	0 to 15	24.3	0.80	16.1
	30	29.8	0.80	19.8
	50	34.5	0.80	22.9
	80	39.6	0.80	26.3
	120	44.3	0.80	29.4
	157	47.7	0.80	31.7
Leeward wall	All	47.7	-0.50	-19.8
Side walls	All	47.7	-0.70	-27.7
Roof	0 to 79	47.7	-1.04	-41.2
	79 to 100	47.7	-0.70	-27.7

Note: $q_h = 47.7$ psf; $G = 0.83$.

Table G4-7**External Pressures for MWFRS for Wind Normal to 100-ft Face**

Surface	z (ft)	q (psf)	C_p	External pressure (pdf)
Windward wall	0 to 15	24.3	0.80	16.1
	30	29.8	0.80	19.8
	50	34.5	0.80	22.9
	80	39.6	0.80	26.3
	120	44.3	0.80	29.4
	157	47.7	0.80	31.7
Leeward wall	All	47.7	-0.30	-11.9
Side walls	All	47.7	-0.70	-27.7
Roof	0 to 79	47.7	-0.98	-38.8
	79 to 157	47.7	-0.78	-30.9
	157 to 200	47.7	-0.62	-24.5

Note: $q_h = 47.7$ psf; $G = 0.83$.

up to 30 ft above the aggregate surfaced roof must be debris impact resistant from the ground to 30 ft above the adjacent building roof.

Since there is no information in this example about an adjacent aggregate surfaced roof, the lower 60 ft of the building is considered as an enclosed building using $GC_{pi} = \pm 0.18$ because of impact-resistant glazing. Above 60 ft, glazing is considered as an opening; the building is classified as a partially enclosed building.

Design Pressures for the MWFRS

$$p = qGC_p - q_i(GC_{pi}) \quad (\text{Eq. 27.4-1})$$

For enclosed buildings

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

For partially enclosed buildings

$$GC_{pi} = \pm 0.55 \quad (\text{Table 26.11-1})$$

For q_i , $q_h = 47.7$ psf for negative internal pressure, and q_z will be evaluated at 60 ft for positive internal pressure (the point at which the enclosed building classification changes to partially enclosed).

Internal Pressure Calculation

$$\text{Negative internal pressure} = 47.7 \times (-0.55) = -26.2 \text{ psf}$$

$$\text{Positive internal pressure} = 36.2 \times 0.55 = 19.9 \text{ psf } (q_z \text{ is obtained by interpolation at height } z = 60 \text{ ft from Table G4-2 of this guide})$$

Parapet Load on MWFRS

According to Section 27.4.5 of the Standard,

$$P_p = q_p G C_{pn} \quad (\text{Eq. 27.4-4})$$

$$q_p = 48.1 \text{ psf}$$

$$G C_{pn} = 1.5 \text{ for windward parapet}$$

$$= -1.0 \text{ for leeward parapet}$$

Pressure on parapets of MWFRS can be determined as follows:

$$P_p = 48.1 \times 1.5 = 72.2 \text{ psf for windward parapet}$$

$$= 48.1 \times -1.0 = -48.1 \text{ psf for leeward parapet}$$

Design wind pressures for MWFRS are shown in Fig. G4-2 for wind normal to 200-ft face and in Fig. G4-3 for wind normal to 100-ft face.

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

Design Wind Load Cases

Section 27.4.6 of the Standard requires that any building whose wind loads have been determined under the provisions of Sections 27.4.1 and 27.4.2 shall be designed for wind load cases as defined in Fig. 27.4-8. Case 1 includes the loadings determined in this example and shown in Figs. G4-2 and G4-3. A combination of windward (P_W) and leeward (P_L) loads are applied for Load Cases 2, 3, and 4 as shown in Fig. G4-4.

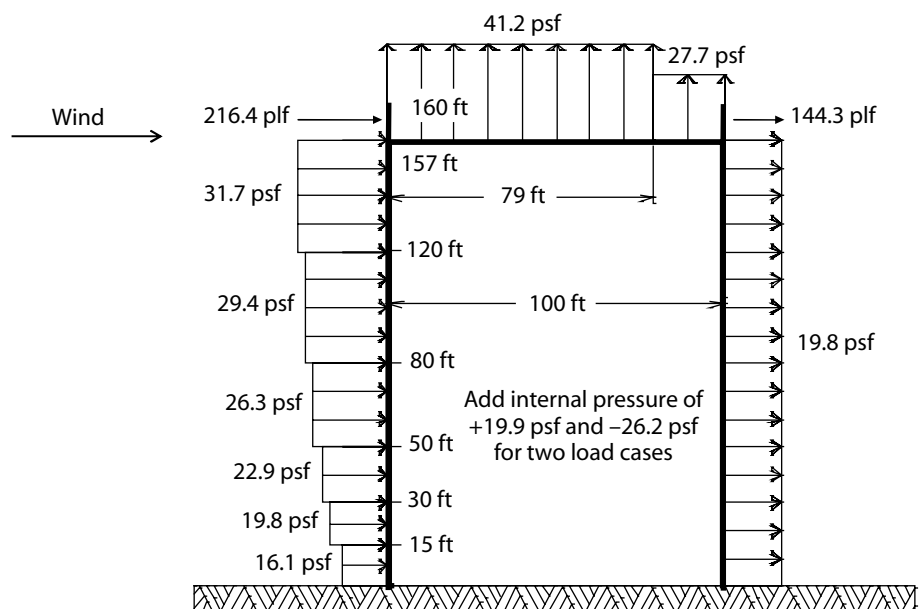
Design Pressures for Components and Cladding

Design pressure for C&C is obtained according to chapter 30, part 3. The equation is

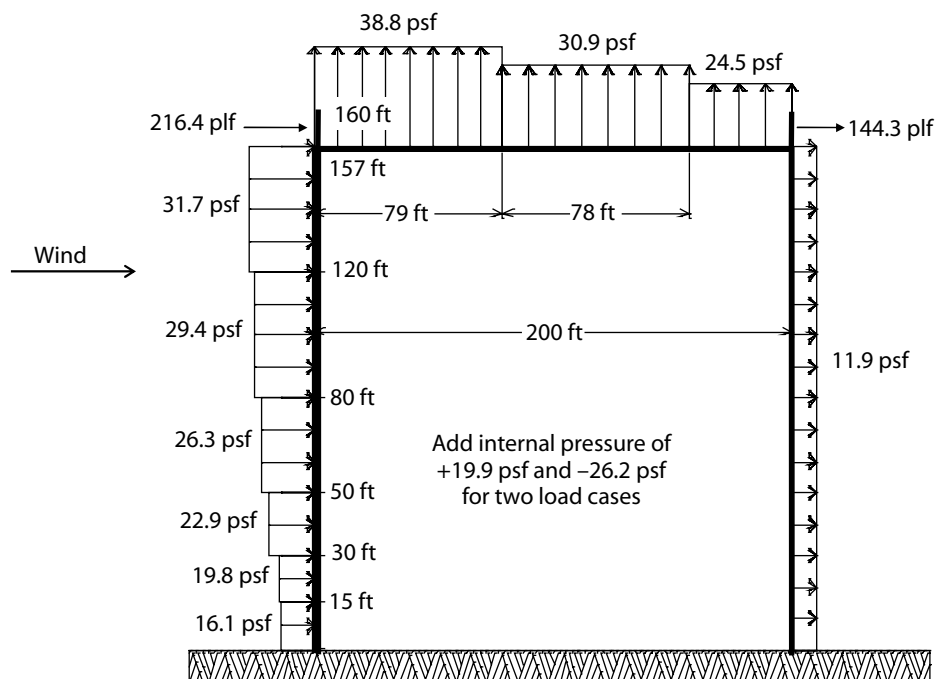
$$p = q(GC_p) - q_i(GC_{pi}) \quad (\text{Eq. 30.6-1})$$

Fig. G4-2

Design pressures for MWFRS for wind normal to the 200-ft face



Design pressures for
MWFRS for wind normal
to the 100-ft face



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

$$= q_z \text{ evaluated at } 60 \text{ ft} = 36.2 \text{ psf for positive internal pressure}$$

(GC_p) = External pressure coefficient (see Fig. 30.6-1 of the Standard for flat roofs)

(GC_{pi}) = Internal pressure coefficient (see Table 26.11-1 of the Standard)

Design pressures in Case B for MWFRS for wind normal to 100-ft face

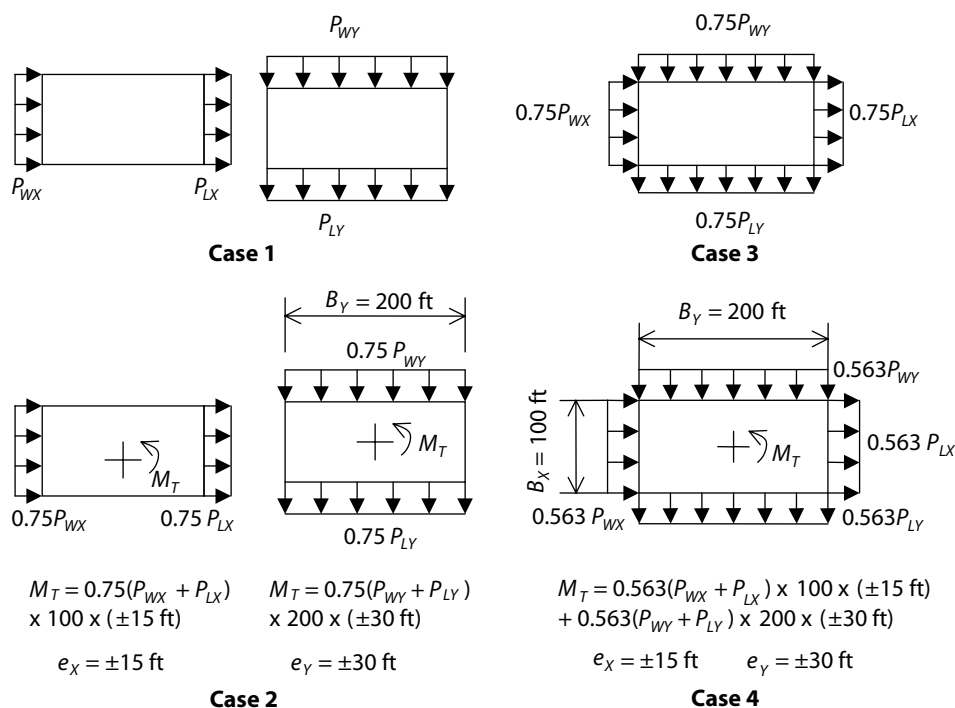


Table G4-8**Wall Pressure Coefficient**

<i>Component</i>	<i>A</i> (ft ²)	<i>Zones 4 and 5</i> (+GC _p)	<i>Zone 4</i> (-GC _p)	<i>Zone 5</i> (-GC _p)
Mullion	55	0.81	-0.84	-1.55
Panel	27.5	0.87	-0.88	-1.72

Wall Design Pressures

The pressure coefficients (GC_p) are a function of effective wind area (see Table G4-8). 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 26.2 of the Standard). The effective wind areas, A , for wall components are

Mullion

$$\begin{array}{ll} \text{larger of} & A = 11(5) = 55 \text{ ft}^2 \text{ (controls)} \\ \text{or} & A = 11(11/3) = 40.3 \text{ ft}^2 \end{array}$$

Glazing panel

$$\begin{array}{ll} \text{larger of} & A = 5(5.5) = 27.5 \text{ ft}^2 \text{ (controls)} \\ \text{or} & A = 5(5/3) = 8.3 \text{ ft}^2 \end{array}$$

Width of Corner Zone 5

$$\begin{array}{ll} \text{larger of} & a = 0.1(100) = 10 \text{ ft (controls)} \\ \text{or} & a = 3 \text{ ft} \end{array}$$

The internal pressure coefficient (GC_{pi}) = ± 0.55 (Table 26.11-1). See the aforementioned notes about the location of enclosed building area and internal pressure coefficient of $GC_{pi} = \pm 0.18$ can be used in the bottom 60 ft.

Typical design pressure calculations

Design pressures for mullions are shown in Table G4-9 and for panels in Table G4-10.

Table G4-9**Controlling Design Pressures for Mullions (psf)**

<i>z (ft)</i>	<i>Zone 4</i>		<i>Zone 5</i>	
	<i>Positive</i>	<i>Negative</i>	<i>Positive</i>	<i>Negative</i>
0 to 15	28.3	-46.6	28.3	-80.4
15 to 30	32.7	-46.6	32.7	-80.4
30 to 50	36.5	-46.6	36.5	-80.4
50 to 80	58.3	-60.0	58.3	-93.8
80 to 120	62.1	-60.0	62.1	-93.8
120 to 157	64.8	-60.0	64.8	-93.8

Table G4-10**Design Pressures for Panels (psf)**

<i>z</i> (ft)	<i>Zone 4</i>		<i>Zone 5</i>	
	<i>Positive</i>	<i>Negative</i>	<i>Positive</i>	<i>Negative</i>
0 to 15	29.7	−48.5	29.7	−88.5
15 to 30	34.5	−48.5	34.5	−88.5
30 to 50	38.6	−48.5	38.6	−88.5
50 to 80	60.6	−61.9	60.6	−101.9
80 to 120	64.7	−61.9	64.7	−101.9
120 to 157	67.7	−61.9	67.7	−101.9

Controlling negative design pressure for mullion in Zone 4 of walls for $h = 60$ ft and above:

$$= 47.7(-0.84) - 36.2 \times 0.55$$

$$= -60.0 \text{ psf (positive internal pressure controls)}$$

Controlling negative design pressure for mullion in Zone 4 for wall below 60 ft:

$$= 47.7(-0.84) - 36.2 \times 0.18$$

$$= -46.6 \text{ psf (positive internal pressure controls)}$$

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

$$= 47.7 \times 0.81 - 47.7 \times (-0.55)$$

$$= 64.8 \text{ psf (negative internal pressure controls)}$$

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 chapter 30, part 6. In this example, the effective wind area is assumed to be $3 \text{ ft} \times 3 \text{ ft} = 9 \text{ ft}^2$. Equation for design pressure is as follows:

$$p = q_p ((GC_p) - (GC_{pi})) \quad (\text{Eq. 30.9-1})$$

where

q_p = Velocity pressure evaluated at the top of parapet

GC_p = External pressure coefficient from Figs. 30.4-1 through 30.6-1 of the Standard

GC_{pi} = Internal pressure coefficient from Table 26.11-1 of the Standard, based on the porosity of the parapet envelope. In this example, internal pressure is not included since the parapet is assumed to be nonporous.

Note that, according to Note 7 of Fig. 30.6-1, Zone 3 is treated as Zone 2.

Table G4-11**Roof External Pressure Coefficient**

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 Fig. 30.6-1 of the Standard permits treatment of Zone 3 as Zone 2 if parapet of 3 ft or higher is provided.

Load Case A

$$48.1 \times [(0.9) - (-2.3)] = 153.9 \text{ psf (directed inward)}$$

Load Case B

$$48.1 \times [(0.9) - (-1.8)] = 129.9 \text{ psf (directed outward)}$$

Roof Design Pressures

The C&C roof pressure coefficients are given in 30.6-1 of the Standard. The pressure coefficients (Table G4-11) 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):

$$\text{Design internal pressures} = 47.7 \times 0.55 = 26.2 \text{ psf}$$

$$\text{Design pressures} = q_h (GC_p) - 26.2 = 47.7(GC_p) - 26.2$$

Design pressures are summarized in Table G4-12.

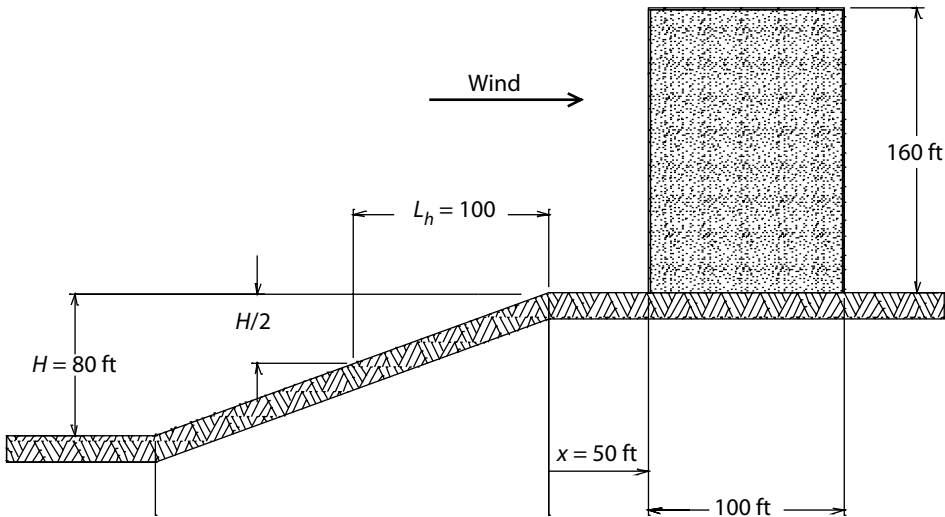
Table G4-12**Roof Design Pressures**

A (ft ²)	Design pressures negative (psf)	
	Zone 1	Zones 2 and 3
≤10	-93.0	-135.9
20	-88.7	-130.2
100	-79.1	-116.4
250	-73.4	-108.2
400	-70.6	-104.4
500	-69.1	-102.5

Fig. G4-5

Building characteristics of an office building on an escarpment

Note: L_h is measured from midheight to top of the slope. Distance x is taken to the front of the building as a conservative value.



4.2 Building Located on an Escarpment

In this example, velocity pressures for the office building in Section 4.1 of this guide, when it is located on an escarpment in a city in Alaska, are determined. This example illustrates use of topography factor K_{zt} given in Section 26.8 and Fig. 26.8-1 of the Standard. Design pressures for MWFRS and C&C can be determined in the same manner as stated once velocity pressures q_z and q_h are determined. The building and topographic feature are illustrated in Fig. G4-5; data are provided in Table G4-13.

Glazing panels are 5 ft wide $\times$ 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. This information is not critical for the example.

Exposure, Building Classification, and Basic Wind Speed

Exposure B, same as Example 3:

Category II

$V = 140$ mph, same as Example 3

Table G4-13

Data for a Building on an Escarpment

Location	City in Alaska
Topography	Escarpment as shown in Fig. G4-5
Terrain	Suburban
Dimensions	100 ft $\times$ 200 ft in plan Roof height of 157 ft with 3-ft parapet Flat roof
Framing	Reinforced concrete frame in both directions Floor and roof slabs provide diaphragm action
Cladding	Mullions for glazing panels span 11 ft between floor slabs Mullion spacing is 5 ft

Velocity Pressures

The velocity pressure equation is

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

where

K_z = value for MWFRS obtained from Table 27.3-1 of the Standard

K_{zt} = value determined using Section 26.8 and Fig. 26.8-1 of the Standard

K_d = 0.85 from Table 26.6-1 of the Standard

V = 140 mph.

Determination of K_{zt}

The topographic effect of escarpment applies only when all conditions of Section 26.8.1 are met.

1. 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.
2. The escarpment is assumed to protrude above upwind terrain features by a factor of more than 2 within a 2-mi distance.
3. The building is on the crest of the escarpment.
4. $H/L_b = 80/100 = 0.8 > 0.2$ (Note: L_b is measured to half the height of the escarpment)
5. H of 80 ft is greater than 60 ft

For use in Fig. 26.8-1 of the Standard:

$$H = 80 \text{ ft}$$

$$L_b = 100 \text{ ft}$$

$$x = 50 \text{ ft (distance to the front face of the building)}$$

Since $H/L_b = 0.8 > 0.5$, according to Note 2 in Fig. 26.8-1 of the Standard, use $H/L_b = 0.5$ and $L_b = 2H = 160 \text{ ft}$.

The building is on a 2-dimensional escarpment.

Exposure B

$$K_1/(H/L_b) = 0.75, \text{ therefore } K_1 = (0.75)(0.5) = 0.38 \quad (\text{Fig. 26.8-1})$$

$$x/L_b = 50 \text{ ft}/160 \text{ ft} = 0.31; K_2 = [1 - (0.31/4)] = 0.92 \quad (\text{Fig. 26.8-1})$$

$$K_3 = e^{-2.5z/L_b} \text{ (values in table for } z\text{)}$$

$$K_{zt} = (1 + K_1 K_2 K_3)^2 \quad (\text{Eq. 26.8-1})$$

$$q_z = 0.00256 K_z K_{zt} (0.85)(140)^2$$

Values for q_z are shown in Table G4-14.

Effect of Escarpment

Velocity pressures q_z are compared with the values of Section 4.1 of this guide in Table G4-15 to assess the effect of the escarpment. The increase in

Table G4-14**Speed-up Velocity Pressures**

Height (ft)	K_z	z/L_h^*	K_3	K_{zt}	q_z (psf)
0 to 15	0.57	0.05	0.88	1.71	41.6
30	0.70	0.14	0.71	1.56	46.5
50	0.81	0.25	0.54	1.41	48.6
80	0.93	0.41	0.36	1.27	50.3
120	1.04	0.63	0.21	1.15	50.9
$h = 157$	$K_h = 1.12$	0.87	0.11	1.08	51.5

Notes: 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. K_z values are obtained from Table 27.3-1 of the Standard.

velocity pressures does not directly translate into an increase in design pressures as discussed following.

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

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

4.3 Simplified Method for Buildings Less Than 160 ft Tall

In this example, design pressures for the office building of Section 4.1 of this guide are determined using the simplified procedures for buildings less than 160 ft tall that are in chapter 27, part 2. The building is illustrated in Fig. G4-1; data are provided in Table G4-1.

Glazing panels are 5 ft wide $\times$ 5 ft 6 in. high (typical). In order to use the simplified procedure in chapter 27, the building must be classified as an

Table G4-15**Comparison of Velocity Pressures, q_z (psf)**

Height (ft)	Homogeneous terrain (Section 4.1)	Escarpment (Section 4.2)	% Increase
0 to 15	24.3	41.6	71
30	29.8	46.5	56
50	34.5	48.6	41
80	39.6	50.3	27
120	44.3	50.9	15
157 (roof)	47.7	51.5	8