Wind Loads

Guide to the Wind Load Provisions of ASCE 7-10

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

ASCE PRESS

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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)

Snow Loads: Guide to the Snow Load Provisions of ASCE 7-10, **by Michael O'Rourke**. (ASCE Press, 2010). Presents a detailed, authoritative interpretation of the snow load provisions of ASCE/SEI 7-10, including worked examples and FAQs. (ISBN 9780784411117)

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)

Wind Issues in the Design of Buildings, **by Leighton Cochram**. (Technical Report, 2012). Explains the ways that structural designers accommodate the effect of extreme wind events on the built environment. (ISBN 9780784412251)

Wind Loads for Petrochemical and Other Industrial Facilities, **by Task Committee on Wind-Induced Forces of the Petrochemical Committee of the Energy Division**. (Technical Report, 2011). Provides state-of-the-practice guidelines for the computation of wind-induced forces on industrial facilities with structural features outside the scope of current codes and standards. (ISBN 9780784411803)

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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.

Unit Conversions

Measurement	*S.I. Units*	*Customary units*
Abbreviations	m = meter (S.I. base unit of length)	yd = yard
	cm = centimeter	ft = foot
	km = kilometer	in. = inch
	ha = hectare	mi = mile
	L = liter (S.I. base unit of volume)	acre
	mL = milliliter	gal = gallon
	kg = kilogram (S.I. base unit of mass)	qt = quart
	g = gram	lb = pound
	N = Newton ($m \cdot kg \cdot s^2$)	oz = ounce
	Pa = Pascal (N/m^2)	plf = lbs per foot
	kPa = kilopascal	lbf = pound-force (lb/ft)
	J = Joule	psi = pounds per square inch
	W = watt	atm = atmosphere
	kW = kilowatt	ft·lbf = feet per pound-force
	s = second (S.I. base unit of time)	Btu = British thermal unit
	min = minute	hp = horsepower
	h = hour	s = second
	day	min = minute
	°C = degrees Celsius	h = hour
	ppm = parts per million	day
		°F = degrees Fahrenheit
		ppm = parts per million
Length	1 m = 3.2808 ft = 1.0936 yd	1 ft = 3 yd = 0.3048 m
	1 cm = 0.3937 in.	1 in. = 2.54 cm
	1 km = 0.6214 mile	1 mile = 0.869 nautical mile = 1.6093 km
Area	1 m^2 = 10.7643 ft^2	1 ft^2 = 0.0929 m^2
	1 km^2 = 0.3861 mi^2	1 mi^2 = 2.59 km^2
	1 ha = 2.4710 acre	1 acre = 43,560 ft^2 = 0.4047 ha
Volume	1 L = 0.2642 gal	1 gal = 4 qt = 3.7854 L
	1 mL = 1 cm^3	1 ft^3 = 7.481 gal = 28.32 L
Mass	1 g = 0.0353 oz	1 oz = 28.3495 g
	1 kg = 2.2046 lb	1 lb = 0.4536 kg
Force	1 N = 0.2248 lb/ft	1 lbf = 4.4482 N
Density	1 kg/m^2 = 0.2048 lb/ft^2	1 lb/ft^2 = 4.882 kg/m^2
	1 kg/m^3 = 6.2427 lb/ft^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 ft·lbf = 1.3558 J
	1 kJ = 0.2778 W·h = 0.948 Btu	1 Btu = 1.0551 kJ
	1 W = 0.7376 ft·lbf/s = 3.4122 Btu/h	1 ft·lbf/s = 1.3558 W
	1 kW = 1.3410 hp	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. Specification 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

Subject	*Selected Reference Material (see References Section 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); 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:

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

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

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

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 ft^2. The equation for determining design wind pressures is

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

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:

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

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

$$p_{\text{net}} = (\lambda)\, K_{zt}\, p_{\text{net30}} \text{ (psf)} \qquad \text{(Eq. 30.5-1)}$$

where

p_{net30} = 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 G2-1 Walls for Buildings with $h \leq 60$ ft (Figure 30.4-1)

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

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

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

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

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

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

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 $10 < A \leq 100$ 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 $10 < A \leq 100$ 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 $10 < A \leq 100$ 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
Negative with overhang		
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 G2-4 Gable Roofs with $h \leq 60$ ft, $27° < \theta \leq 45°$ (Figure 30.4-2C)

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 $10 < A \leq 100$ 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 $10 < A \leq 100$ 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 $10 < A \leq 100$ 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 $10 < A \leq 100$ ft^2
	$(GC_p) = -1.8$	for $A > 100$ ft^2

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

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

Positive		
Zones 1, 2, and 3	$(GC_p) = 0.6$	for $A = 10$ ft^2
	$(GC_p) = 0.8000 - 0.2000 \log A$	for $10 < A \leq 100$ 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 $10 < A \leq 100$ 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 G2-6 Multispan Gable Roofs with $h \leq 60$ ft, $30° < \theta \leq 45°$ (Figure 30.4-4)

Positive		
Zones 1, 2, and 3	$(GC_p) = 1.0$	for $A = 10$ ft^2
	$(GC_p) = 1.2000 - 0.2000 \log A$	for $10 < A \leq 100$ 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 $10 < A \leq 100$ 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 G2-7 Monoslope Roofs with $h \leq 60$ ft, $3° < \theta \leq 10°$ (Figure 30.4-5A)

Positive		
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
Negative		
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 G2-8 Monoslope Roofs with $h \leq 60$ ft, $10° < \theta \leq 30°$ (Figure 30.4-5B)

Positive		
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
Negative		
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)

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

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

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

	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 $10 < A \leq 500$ ft^2
	$(GC_p) = -0.9$	for $A > 500$ ft^2
Zone 2	$(GC_p) = -2.3$	for $A = 10$ ft^2
	$(GC_p) = -2.7120 + 0.4120 \log A$	for $10 < A \leq 500$ ft^2
	$(GC_p) = -1.6$	for $A > 500$ ft^2
Zone 3	$(GC_p) = -3.2$	for $A = 10$ ft^2
	$(GC_p) = -3.7297 + 0.5297 \log A$	for $10 < A \leq 500$ 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 $20 < A \leq 500$ 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 $20 < A \leq 500$ ft^2
	$(GC_p) = -0.7$	for $A > 500$ ft^2
Zone 5	$(GC_p) = -1.8$	for $A = 20$ ft^2
	$(GC_p) = -2.5445 + 0.5723 \log A$	for $20 < A \leq 500$ ft^2
	$(GC_p) = -1.0$	for $A > 500$ 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; hurricanes 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:

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

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

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

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

		Life of a building in years		
Annual probability	*MRI in years*	*25*	*50*	*100*
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 × 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:

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

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

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

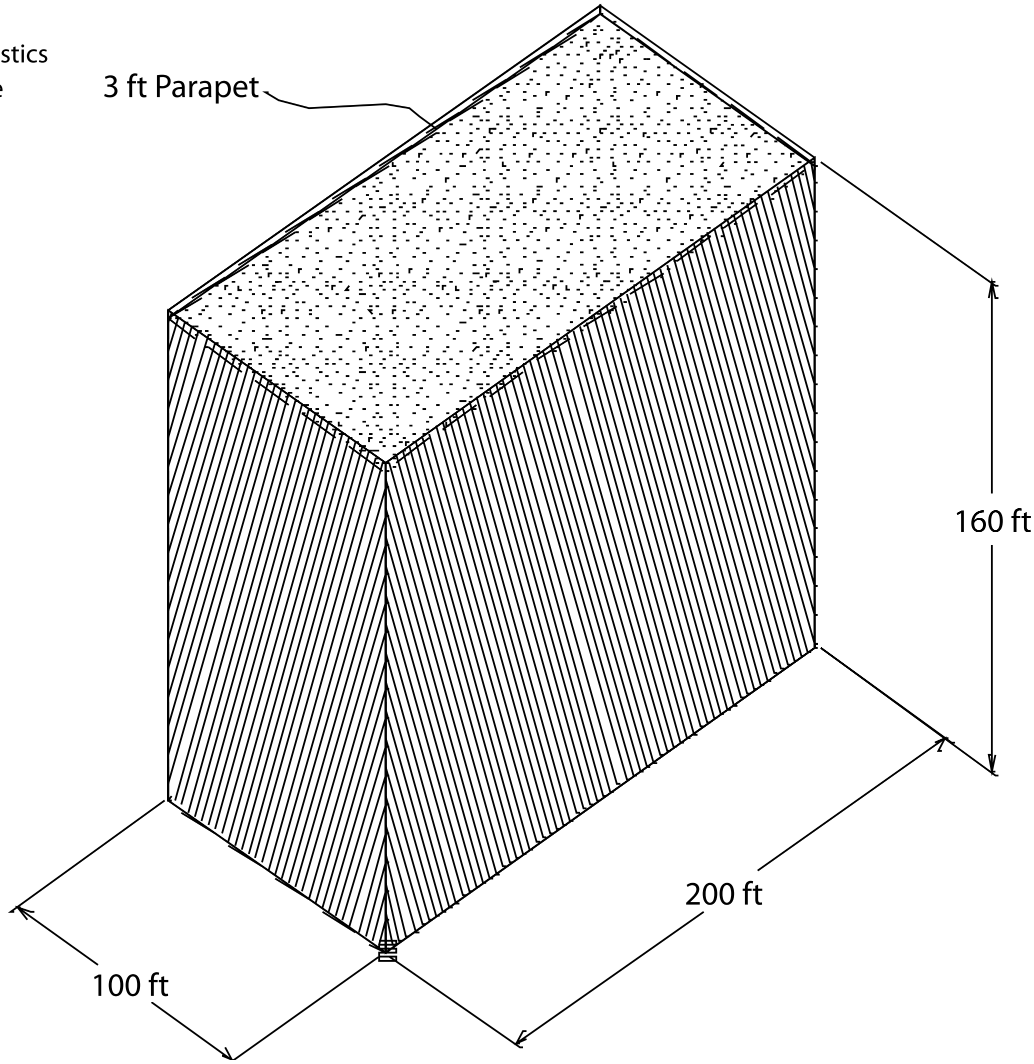


Table G4-1 Data for 160-ft-Tall Office Building

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

	MWFRS		*C&C*	
Height, ft	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.

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

where

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

Therefore

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

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

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

where

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

Table G4-3 Wall Pressure Coefficient

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

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 $\theta < 10°$, two zones are specified in Fig. 27.4-1 of the Standard.

First value

0 to $h/2$, $C_p = -1.3$
$>h/2$, $C_p = -0.7$

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

Table G4-4 Roof Pressure Coefficient 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 G4-5 Roof Pressure Coefficient, C_p, for Wind Normal to 100-ft Face

Distance from windward edge	h/L ≤ *0.5*	h/L = *0.8*	h/L ≥ *1.0*
0 to *h*/2	–0.9	–0.98	–1.04
h/2 to *h*	–0.9	–0.78	–0.7
h to 2*h*	–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 = 47.7psf; 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 = 47.7psf; 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

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

For enclosed buildings

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

For partially enclosed buildings

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

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

Negative internal pressure = 47.7 × (–0.55) = –26.2 psf
Positive internal pressure = 36.2 × 0.55 = 19.9 psf (q_z is obtained by interpolation at height z = 60 ft from **Table G4-2** of this guide)

Parapet Load on MWFRS

According to Section 27.4.5 of the Standard,

$$
\begin{aligned}
P_p &= q_p GC_{pn} \quad \text{(Eq. 27.4-4)} \\
q_p &= 48.1 \text{ psf} \\
GC_{pn} &= 1.5 \text{ for windward parapet} \\
&= -1.0 \text{ for leeward parapet}
\end{aligned}
$$

Pressure on parapets of MWFRS can be determined as follows:

$$
\begin{aligned}
P_p &= 48.1 \times 1.5 = 72.2 \text{ psf for windward parapet} \\
&= 48.1 \times -1.0 = -48.1 \text{ psf for leeward parapet}
\end{aligned}
$$

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

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

Fig. G4-2
Design pressures for MWFRS for wind normal to the 200-ft face

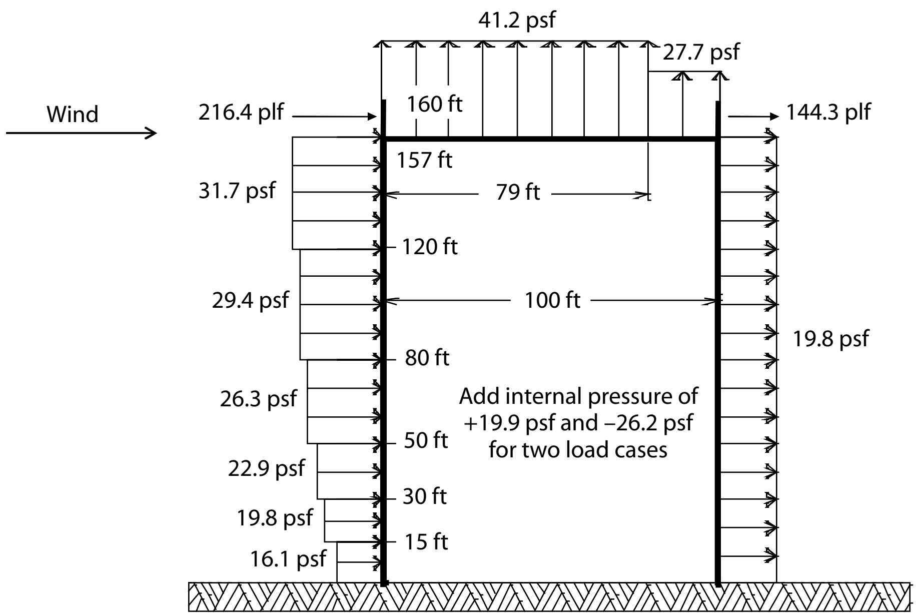
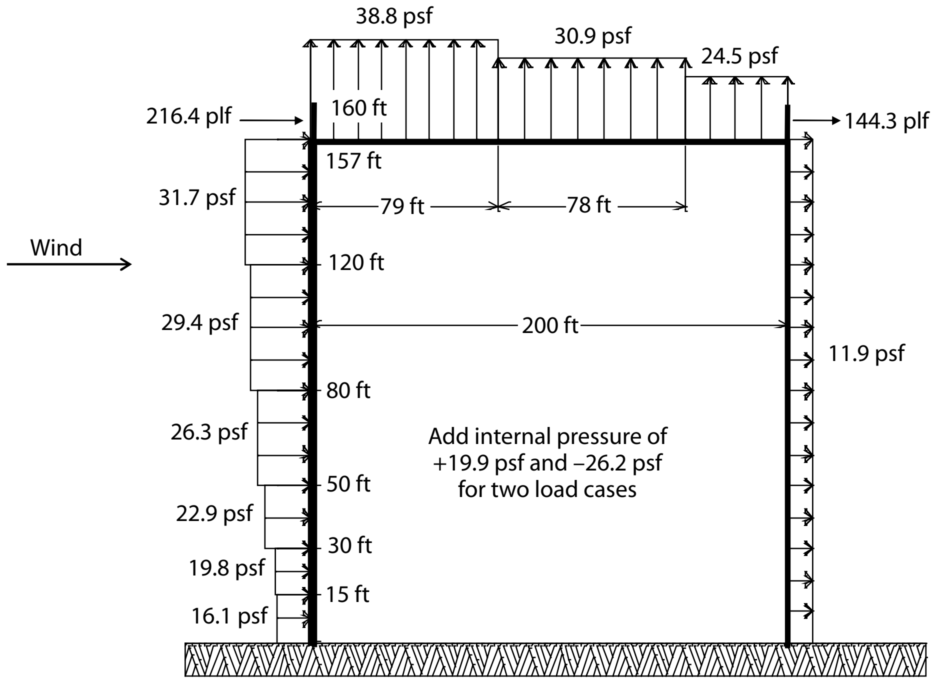


Fig. G4-3
Design pressures for MWFRS for wind normal to the 100-ft face



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 = 47.7 psf for negative internal pressure

= q_z evaluated at 60 ft = 36.2 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)

Fig. G4-4
Design pressures in Case B for MWFRS for wind normal to 100-ft face

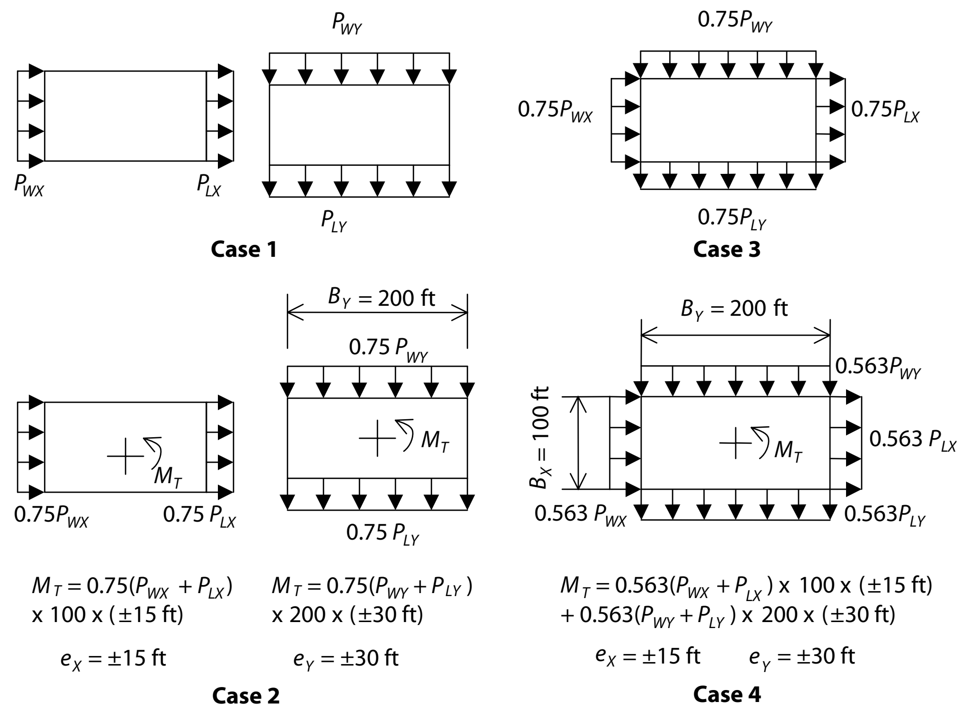


Table G4-8 Wall Pressure Coefficient

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

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

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

Glazing panel

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

Width of Corner Zone 5

larger of $a = 0.1(100) = 10$ ft (controls)
or $a = 3$ ft

The internal pressure coefficient (GC_{pi}) = ±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)

	Zone 4		Zone 5	
z (ft)	*Positive*	*Negative*	*Positive*	*Negative*
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)

	Zone 4		*Zone 5*	
z *(ft)*	*Positive*	*Negative*	*Positive*	*Negative*
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:

$= 47.7(-0.84) - 36.2 \times 0.55$
$= -60.0$ psf (positive internal pressure controls)

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

$= 47.7(-0.84) - 36.2 \times 0.18$
$= -46.6$ psf (positive internal pressure controls)

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

$= 47.7 \times 0.81 - 47.7 \times (-0.55)$
$= 64.8$ 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 ft × 3 ft = 9 ft^2. Equation for design pressure is as follows:

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

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 *(ft²)*	*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):

Design internal pressures = $47.7 \times 0.55 = 26.2$ psf
Design pressures = $q_h\,(GC_p) - 26.2 = 47.7(GC_p) - 26.2$

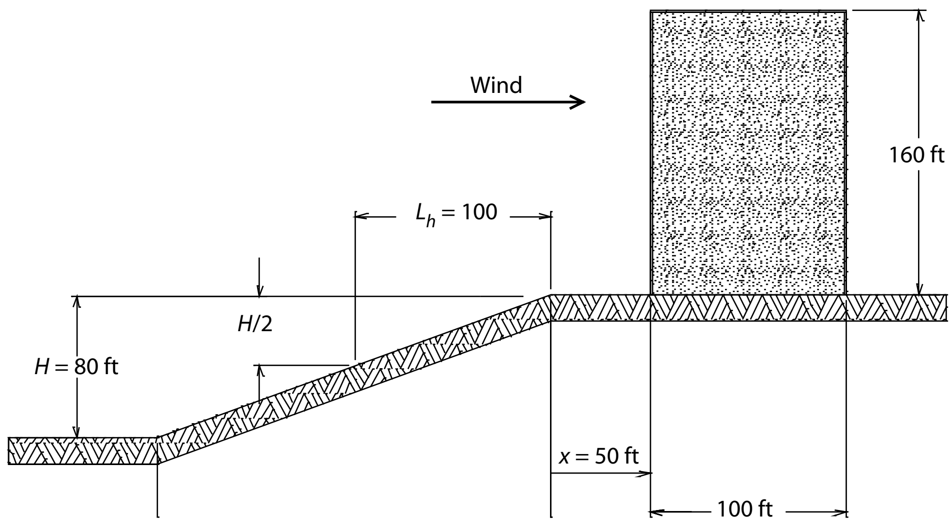
Design pressures are summarized in **Table G4-12**.

Table G4-12 Roof Design Pressures

	Design pressures negative (psf)	
A *(ft²)*	*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 × 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 × 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. $H/L_h = 80/100 = 0.8 > 0.2$ (Note: L_h 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 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 Fig. 26.8-1 of the Standard, use $H/L_h = 0.5$ and $L_h = 2H = 160$ ft.

The building is on a 2-dimensional escarpment.

Exposure B

$K_1/(H/L_h)$ = 0.75, therefore $K_1 = (0.75)\ (0.5) = 0.38$ (Fig. 26.8-1)
x/L_h = 50 ft/160 ft = 0.31; $K_2 = [1 - (0.31/4)] = 0.92$ (Fig. 26.8-1)
$K_3 = e^{-2.5z/L_h}$ (values in table for z)
$K_{zt} = (1 + K_1\ K_2\ K_3)^2$ (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 × 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

enclosed building. For this example, we will assume that the glazing is debris resistant or protected in a way to satisfy that requirement.

Exposure, Building Classification, and Basic Wind Speed

Exposure B, same as Example in **Sec. 4.1**
Category II
$V = 140$ mph, same as Example in **Sec. 4.1**

Determination of Chapter 27: Part 2 Provisions

Chapter 27, part 2, of the Standard lists a number of conditions that the building must meet in order for this simplified procedure to be used. The building in this example is a Class 2 building given the height of 160 ft. The conditions are as follows:

1. The building shall be an enclosed simple diaphragm building as defined in Section 26.2. (This building meets this condition as defined.)
2. The building shall have a mean roof height 60 ft $< h \leq 160$ ft. (This building meets this condition as defined with $h = 157$ ft.)
3. The ratio of L/B shall not be less than 0.5 nor more than 2.0. (This building has L/B values of 0.5 and 2.0; it meets the conditions.)
4. The natural frequency (hertz) of the building used to determine the gust-effect factor G_f defined in Section 26.9.2 shall not be less than $75/h$ where h is in feet. (This building has the ratio of $75/h = 75/157 = 0.48$. This requirement is liberal since a flexible building in Section 26.2 is defined as one with a fundamental natural frequency less than 1.0. It is assumed that this building meets this requirement.)
5. The topographic effect factor $K_{zt} = 1.0$ or the wind pressures determined from this section shall be multiplied by K_{zt} as determined from Section 26.8. (This building meets this condition as defined.)

Design Pressures for the MWFRS

Wall Pressures

Wall pressures are determined using Table 27.6-1. There are pressures in the table for each of the exposure categories, B, C, and D. The pressures in Table 27.6-1 are given for bottom (ground level) of building and at the roof height of the building. It is assumed that pressures vary linearly along the height of the building. The pressure at any height z is found as follows (this equation is not in the Standard):

$$p_z = p_0[1 - (z/h)] + p_h(z/h)$$

where

p_z = wall pressure at height z
p_0 = wall pressure in the table at the bottom of the building for a particular wind speed, Exposure category, L/B ratio, and roof height h

Table G4-16 Interpolation of Values of p_0 and p_h

V *(mph)* h *(ft)*, L/B	*140 mph*	
	L/B = *0.5, psf*	L/B = *2.0, psf*
160	66.3	59.7
	44.6	36.8
157	**65.6**	**59.1**
	44.3	**36.6**
150	63.9	57.6
	43.5	36.0

Note: The wall pressures at 157 ft are interpolated.

p_h = wall pressure in the table at the top of the building for a particular wind speed, Exposure category, *L/B* ratio, and roof height *h*
z = height of particular interest
h = mean roof height

For example, the wall pressure at height h = 157 ft must be interpolated between the heights provided in Table 27.6-1 of 150 ft and 160 ft. Both pressures p_0 and p_h must be found for both *L/B* ratios of 100/200 = 0.5 and 200/100 = 2.0 for two directions. The linear interpolation of these pressures is shown in **Table G4-16**.

The wall pressures applicable to this example are determined from Eq. 4-10 for p_z and shown in **Table G4-17**. However, these pressures include both windward and leeward pressures.

Side Wall and Leeward Wall Pressures

Side wall pressures are determined in accordance with Note 2 of Table 27.6-1. The side wall pressure is determined by taking 54 percent of p_h for $0.2 \le L/B \le 1.0$ and 64 percent of p_h for $2.0 \le L/B \le 5.0$. These side wall external pressures do not include the effect of internal pressure. For this example, these side wall pressures are

For $L/B = 0.5$: side wall pressure = 0.54×65.6 psf = 35.4 psf
For $L/B = 2.0$: side wall pressure = 0.64×59.1 psf = 37.8 psf

Table G4-17 Wall Pressure, p_z (psf)

z *(ft)*	L/B = *0.5 pressure*	L/B = *2.0 pressure*
0	44.3	36.6
30	48.4	40.9
50	51.1	43.8
80	55.2	48.1
120	60.5	53.8
157	65.6	59.1
160	66.3	59.7

Table G4-18 Windward and Leeward Wall Pressures (psf)

	L/B = 0.5 pressure			*L/B = 2.0 pressure*		
z (ft)	*Pressures from Table G4-17*	*Leeward pressure*	*Windward pressure*	*Pressures from Table G4-17*	*Leeward pressure*	*Windward pressure*
0	44.3	24.9	19.4	36.6	16.0	20.6
30	48.4	24.9	23.5	40.9	16.0	24.9
50	51.1	24.9	26.2	43.8	16.0	27.8
80	55.2	24.9	30.3	48.1	16.0	32.1
120	60.5	24.9	35.6	53.8	16.0	37.8
157	65.6	24.9	40.7	59.1	16.0	43.1
160	66.3	24.9	41.4	59.7	16.0	43.7

Leeward wall pressures are determined in accordance with Note 4 of Table 27.6-1. The leeward wall pressure is determined by taking 38 percent of p_h for $0.2 \le L/B \le 1.0$ and 27 percent of p_h for $2.0 \le L/B \le 5.0$. These leeward wall external pressures do not include the effects of internal pressure. For this example, these leeward wall pressures are

$$\text{For } L/B = 0.5: \text{leeward wall pressure} = 0.38 \times 65.6 \text{ psf} = 24.9 \text{ psf}$$
$$\text{For } L/B = 2.0: \text{leeward wall pressure} = 0.27 \times 59.1 \text{ psf} = 16.0 \text{ psf}$$

The windward pressures only are determined by subtracting the leeward pressures from the pressures shown in **Table G4-17**. These windward pressures are shown in **Table G4-18**.

Roof Pressures

The roof area is divided into zones in accordance with the roof sketches shown in Table 27.6-2. The roof for this example is a flat roof and thus has three zones: 3, 4, and 5. There is only one load case for this roof shape, as indicated by only one set of pressures in Table 27.6-2. The tabulated values are for Exposure C and relate to wind speed, roof height h, and roof slope. The wind pressure at h = 157 ft must be interpolated. The linear interpolation of these pressures is shown in **Table G4-19**.

The roof pressures in Table 27.6-2 are for Exposure C. The Exposure for this example is Exposure B; therefore, the pressures must be modified

Table G4-19 Interpolation of Roof Pressure, by Zone

	Pressures at 140 mph, psf		
z (ft)	*Zone 3*	*Zone 4*	*Zone 5*
160	–63.2	–56.3	–46.2
157	–62.9	–56.1	–46.0
150	–62.3	–55.6	–45.6

using the exposure adjustment factor (EAF) shown in the graph in Table 27.6-2. This EAF must also be interpolated for the height h = 157 ft. The interpolated value for h = 157 ft is 0.808. The roof pressure must also be adjusted for K_{zt} for topographic influences if required. The design roof pressures in the three zones are:

Zone 3 pressure at h = 157 ft = –62.9 psf × 0.808 (EAF) = –50.8 psf
Zone 4 pressure at h = 157 ft = –56.1 psf × 0.808 = –45.3 psf
Zone 5 pressure at h = 157 ft = –46.0 psf × 0.808 = –37.2 psf

Wind Pressure at the Parapet

Section 27.6.2 indicates that the effect of horizontal wind loads applied to the vertical surfaces of the parapets is equal to the wall pressure from Table 27.6-1 for L/B = 1.0 where h = height of the parapet or 160 ft in this example. The net pressure on the parapets is equal to 2.25 times the pressure determined from Table 27.6-1, as noted. For this example, the wall pressure at 160 ft with L/B = 1.0 = 65.4 psf. The parapet net pressure = 65.4 psf × 2.25 = 147.2 psf.

The comparative wall pressure results using the Simplified Procedure in this section and the Analytical (Directional) Procedures of chapter 27 in **Section 4-1** of this guide are shown in **Tables G4-20 and G4-21**.

It is noted that the pressures obtained from the simplified procedure are 10–20 percent higher than the ones obtained from analytical procedure. A major reason for this difference is that the Simplified Procedure used a calculated gust effect factor G_f. The Simplified Procedure is assuming a more flexible structure to be more conservative for all types of buildings found in practice. The gust effect factor used in this example is 1.02 for L/B = 0.5 and 1.08 for L/B = 2.0 with approximate fundamental frequency of 0.46 Hz (using the gust effect equations shown in Section 26.9). The Analytical (Directional) Procedure in **Section 4.1** of this guide assumed a rigid structure (fundamental frequency >1.0 Hz), which gave a value of G_f of 0.83.

Table G4-20 Comparison by Procedure of Wall Pressures at the Parapet for *L/B* of 0.5 (psf)

	Simplified procedure			*Analytical (directional) procedure*		
z (ft)	*Pressures from Table G4-17*	*Leeward pressure*	*Windward pressure*	*Total wall pressure*	*Leeward pressure*	*Pressures from Table G4-6*
0	44.3	24.9	19.4			
0 to 15				35.9	19.8	16.1
30	48.4	24.9	23.5	39.6	19.8	19.8
50	51.1	24.9	26.2	42.7	19.8	22.9
80	55.2	24.9	30.3	46.1	19.8	26.3
120	60.5	24.9	35.6	49.2	19.8	29.4
157	65.6	24.9	40.7	51.5	19.8	31.7

Table G4-21 Comparison by Procedure of Wall Pressures at the Parapet for *L/B* of 2.0 (psf)

	Simplified procedure			*Analytical (directional) procedure*		
z (ft)	*Pressures from Table G4-17*	*Leeward pressure*	*Windward pressure*	*Total wall pressure*	*Leeward pressure*	*Pressures from Table G4-7*
0	36.6	16.0	20.6			
0 to 15				28.0	11.9	16.1
30	40.9	16.0	24.9	31.7	11.9	19.8
50	43.8	16.0	27.8	34.8	11.9	22.9
80	48.1	16.0	32.1	38.2	11.9	26.3
120	53.8	16.0	37.8	41.3	11.9	29.4
157	59.1	16.0	43.1	43.6	11.9	31.7

Another potentially useful comparative tool is to study the story moments along the height of the building. These moments have been determined by considering the total wall pressures (both windward and leeward) at each level. Thus, total wind pressure increases the resultant moment as the height decreases (see **Table G4-22**). The moment is determined by the following formula:

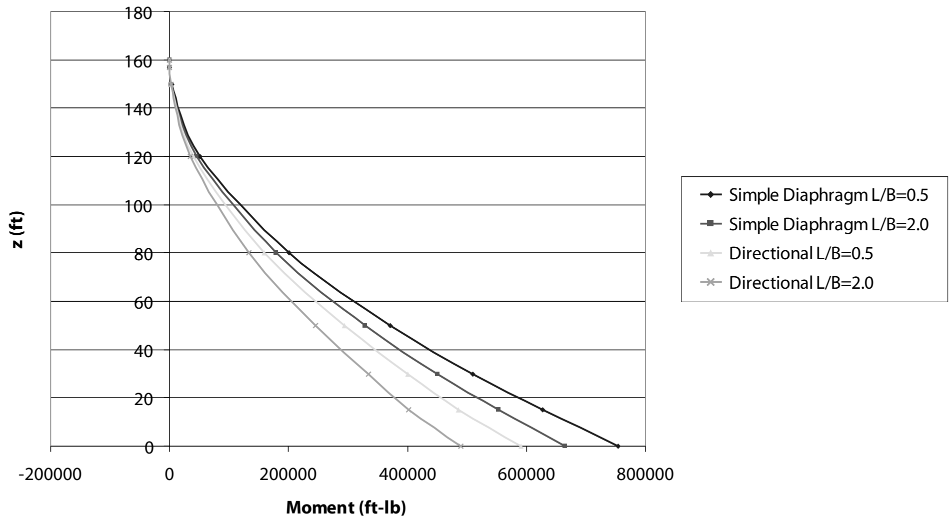
$$M_z = 0.5p_z(h - z)^2 + 0.33(p_h - p_z)(h - z)^2$$

Graphically the moments are shown in **Figure G4-6** as moment per foot of building width (ft-lb). The Simplified Procedure is clearly providing more conservative results than the Analytical (Directional) Procedure, especially at the lower heights where wind pressures are accumulating and causing more overturning moment.

Table G4-22 Comparison by Procedure of Moments at the Parapet (ft-lb)

	Simplified procedure		*Directional procedure*	
z (ft)	L/B = *0.5*	L/B = *2.0*	L/B = *0.5*	L/B = *2.0*
160	0	0	0	0
157	298	268	232	196
150	3,292	2,961	2,569	2,173
120	51,564	46,203	40,569	34,233
80	200,355	178,582	158,893	133,548
50	370,427	328,800	293,424	245,507
30	509,580	451,009	400,884	333,959
15	626,689	553,419	485,590	402,329
0	754,202	664,499	591,254	489,875

Fig. G4-6
Comparison of moments and heights for simplified procedure and analytical procedure



Wind Pressures on Components and Cladding

The determination of C&C wind pressures for building $h > 60$ ft is given in chapter 30, part 4. It uses the following expression:

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

where

p = C&C wind pressure for the component in question
p_{table} = wind pressure from Table 30.7-2 for Exposure C
EAF = exposure adjustment factor shown in Table 30.7-2 to account for exposure condition other than Exposure C
RF = reduction factor for effective wind area (EWA). The pressures in Table 30.7-2 are based on EWA of 10 ft^2. The RF modifies this EWA and is given in Table 30.7-2.
K_{zt} = topographic factor as defined in Section 26.8

As determined in Example 3, there are two C&C examples with their respective EWAs.

Wall mullions have EWA = 55 ft^2
Wall glazing panel has EWA = 27.5 ft^2

The RFs are taken from the chart in Table 30.7-2 and specify which reduction factor line to use in the graph of those lines (A–E) also shown in Table 30.7-2. The RF for flat roofs and wall Zones 4 and 5 are shown in **Table G4-23**, where bold type highlights this example. The RFs are then determined from **Fig. G4-7**.

Table G4-24 shows the reduction factors associated with the mullion and glazing panels. The C&C pressures are determined by using the pressures provided in Table 30.7-2 and interpolating between 160 ft and 150 ft to find the pressures at $h = 157$ ft (see **Table G4-25**).

The C&C wall pressures from Table 30.7-2 and the interpolated values from **Table G4-25** above are shown in **Table G4-26**. Note that the negative

Table G4-23 Reduction Factors for C&C According to Effective Wind Area

Roof form	*Sign pressure*	*Zone 1*	*Zone 2*	*Zone 3*	*Zone 4*	*Zone 5*
Flat	–	D	D	D	C	E
	+	NA	NA	NA	D	D
Gable, Mansard	–	B	C	C	C	E
	+	B	B	B	D	D
Hip	–	B	C	C	C	E
	+	B	B	B	D	D
Monoslope	+	A	B	D	C	E
	–	C	C	C	D	D
Overhangs	All	A	A	B	NA	NA

pressures are evaluated at *h* and thus are the same value throughout the height as the value at *h* (–64.1 psf at Zone 4 and –117.5 psf at Zone 5). This condition is described in Note 4 of Fig. 30.6-1.

The C&C wall pressures must be adjusted for Exposure B conditions (EAF), for effective wind area (RF) and for topographic conditions, K_{zt} (if required) using Eq. 30.7-1. The design pressures for mullions are shown in **Table G4-27**. The design pressures for glazing panels are shown in **Table G4-28.**

Design pressures for roof C&C elements are determined similarly by using the appropriate roof zone pressures and reduction factors for effective wind areas. There are no positive pressures for the three zones on flat roofs.

When the results of the Simplified Procedure shown in **Tables G4-27** and **G4-28** are compared with the Analytical (Directional) Procedure for

Fig. G4-7
Reduction factors in effective wind area

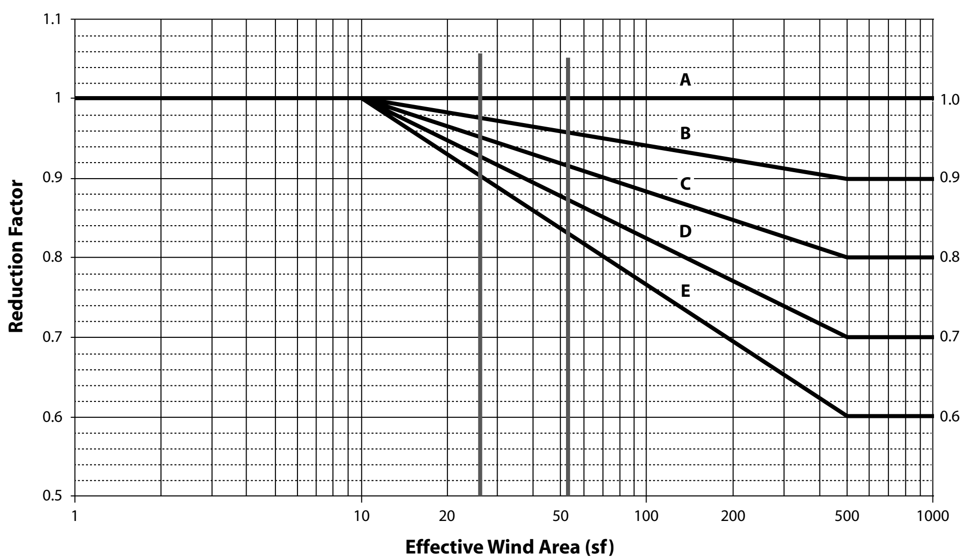


Table G4-24 Reduction Factor for Effective Wind Area for Mullions and Glazing Panels

	Mullion			*Glazing panel*		
Pressure sign	*EWA (sf)*	*Zone 4*	*Zone 5*	*EWA (sf)*	*Zone 4*	*Zone 5*
–	55	0.91	0.82	27.5	0.95	0.90
+	55	0.87	0.87	27.5	0.92	0.92

Table G4-25 Interpolation for C&C Wall Pressure for a Flat Roof ($V = 140$ mph)

	Wall pressure (psf)	
Flat roof	*Zone 4*	*Zone 5*
160 ft	–64.4	–118.0
	+64.4	+64.4
157 ft	**–64.1**	**–117.5**
	+64.1	**+64.1**
150 ft	–63.5	–116.4
	+63.5	+63.5

Note: This table only illustrates wall zones. Pressures are available for the three roof zones also. Bold type indicates interpolated figures.

Table G4-26 Wall Pressures at Different Heights (psf)

	Zone 4		*Zone 5*	
Height (ft)	*Negative*	*Positive*	*Negative*	*Positive*
15	–64.1	+39.1	–117.5	+39.1
30	–64.1	+45.2	–117.5	+45.2
50	–64.1	+50.4	–117.5	+50.4
80	–64.1	+55.6	–117.5	+55.6
120	–64.1	+60.6	–117.5	+60.6
157	–64.1	+64.1	–117.5	+64.1

Table G4-27 Design Pressures for Mullions (psf)

	Zone 4					*Zone 5*			
Height (ft)	*EAF*	*–RF*	*Negative pressure*	*+RF*	*Positive pressure*	*–RF*	*Negative pressure*	*+RF*	*Positive pressure*
15	0.808	0.91	–47.1	0.87	27.5	0.82	–77.8	0.87	27.5
30	0.808	0.91	–47.1	0.87	31.8	0.82	–77.8	0.87	31.8
50	0.808	0.91	–47.1	0.87	35.4	0.82	–77.8	0.87	35.4
80	0.808	0.91	–47.1	0.87	39.1	0.82	–77.8	0.87	39.1
120	0.808	0.91	–47.1	0.87	42.6	0.82	–77.8	0.87	42.6
157	0.808	0.91	–47.1	0.87	45.0	0.82	–77.8	0.87	45.0

Table G4-28 Design Pressures for Glazing Panels (psf)

	Zone 4					Zone 5			
Height (ft)	*EAF*	*–RF*	*Negative pressure*	*+RF*	*Positive pressure*	*–RF*	*Negative pressure*	*+RF*	*Positive pressure*
15	0.808	0.95	–49.2	0.92	29.1	0.90	–85.4	0.92	29.1
30	0.808	0.95	–49.2	0.92	33.6	0.90	–85.4	0.92	33.6
50	0.808	0.95	–49.2	0.92	37.5	0.90	–85.4	0.92	37.5
80	0.808	0.95	–49.2	0.92	41.3	0.90	–85.4	0.92	41.3
120	0.808	0.95	–49.2	0.92	45.0	0.90	–85.4	0.92	45.0
157	0.808	0.95	–49.2	0.92	47.6	0.90	–85.4	0.92	47.6

C&C pressures shown in **Tables G4-9** and **G4-10**, a reasonable agreement can be seen between the pressures for positive pressure with z = 15–50 ft. Because of the stated design condition in **Section 4.1** of an enclosed condition for the first 60 ft and a partially enclosed condition above that, the comparison between these C&C results is not as meaningful.

C&C Pressures for the Parapet

As stated in Section 30.7.1.2, there are two load cases for parapets as in the following examples:

Load Case A: For pressure at the top of the parapet assuming 10 ft^2 EWA influenced by the corner zone of the roof, $64.4 \times 0.808 + 201.4 \times 0.808 =$ **214.8 psf** (directed inward). (Note: the Standard is silent about zone 3; it can be zone 2 with 3 ft parapet)

Load Case B: For the same parameters as Load Case A, $64.4 \times 0.808 + 118.0 \times 0.808 =$ **147.4 psf** (directed outward).

In both load cases, there is no reduction taken for EWA, since the assumption was the EWA is 10 ft^2 and the application of the loads is at the corner of the building.

Chapter 5

Commercial/ Warehouse Metal Building

In this example, design wind pressures for a large, one-story commercial/ warehouse building are determined. **Fig. G5-1** shows the dimensions and framing of the building. The building data are in **Table G5-1**.

Procedure for Design Wind Loads

The Analytical (Directional) Procedure for buildings of any height is used for MWFRS in this example (see chapter 27 of the Standard). Alternate provisions of low-rise buildings are illustrated in **Section 5.2** of this guide.

For components & cladding (C&C), the Envelope Procedure for a low-rise building given in chapter 30, part 1 is used.

Building Classification

The building function is commercial-industrial. It is not considered an essential facility or likely to be occupied by 300 persons at one time. Risk Category II is appropriate; see Table 1.5-1.Wind speed map associated with this risk category is in Fig. 26.5-1A.

Basic Wind Speed

Selection of basic wind speed is addressed in Section 26.5.1 of the Standard. Memphis, Tennessee, is not located in the special wind region, nor is there any reason to suggest that winds at the site are unusual or require additional engineering attention. Therefore, the basic wind speed is V = 115 mph (see Fig. 26.5-1A of the Standard).

Exposure

The building is located on flat and open farmland. It does not fit Exposures B or D; therefore, Exposure C is used (Sections 26.7.2 and 26.7.3 of the Standard). Values of K_z are obtained from Table 27.3-1 for MWFRS and from

Fig. G5-1
Building characteristics of a commercial/warehouse metal building

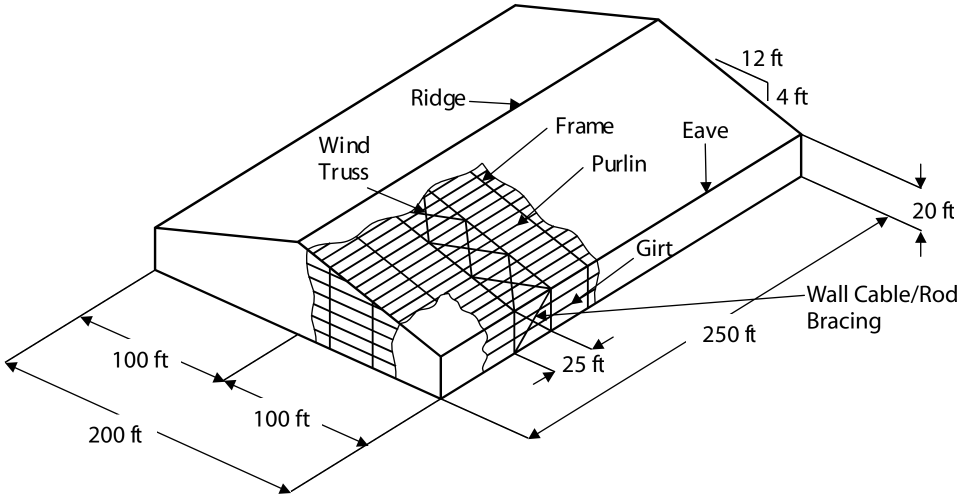


Table 30.3-1 for C&C. For Exposure C, values of K_z in these two tables are the same.

Wind Directionality

Wind directionality factor is given in Table 26.6-1 of the Standard. For MWFRS and C&C, the factor $K_d = 0.85$.

Velocity Pressures

The velocity pressures are computed using the following equation:

$$q_z = 0.00256\ K_z K_{zt} K_d V^2 \text{ psf} \qquad \text{(Eqs. 27.3-1 and 30.3-1)}$$

Table G5-1 Data for Commercial/Warehouse Metal Building

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

where

K_z = value obtained from Table 27.3-1 for MWFRS and from Table 30.3-1 for C&C of the Standard. Note that for C&C only K_h is needed

K_{zt} = 1.0 (no topographic effect)

K_d = 0.85

V = 115 mph

$$q_z = 0.00256\ K_z(1.0)(0.85)(115)^2 = 28.8\ K_z \text{ psf}$$

Values for K_z for Exposure C (see Table 27.3-1 of the Standard) are shown in **Table G5-2**. Mean roof height h = 36.7 ft.

5.1 Analytical Procedure

Design wind pressures for MWFRS of this building can be obtained using Section 27.4.1 of the Standard for buildings of all heights or Section 28.4.1 for low-rise buildings. Pressures determined in this example are using buildings of all height criteria, Section 27.4.1. Example 8 in Section 5.2 illustrates use of low-rise building criteria; see also Section 28.4.1:

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

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 enclosed buildings

G = gust effect factor

C_p = values obtained from Fig. 27.4-1 of the Standard

(GC_{pi}) = values obtained from Table 26.11-1 of the Standard

For this example, when the wind is normal to the ridge, the windward roof experiences both positive and negative external pressures. Combining these

Table G5-2 Velocity Pressures

	Height (ft)	K_z	q_z *(psf)*
Eave	0 to 15	0.85	24.5
	20	0.90	25.9
	30	0.98	28.2
h	36.7	1.02	29.4*
	40	1.04	29.9
	50	1.09	31.4
Ridge	53.3	1.10	31.7

*q_h = 29.4 psf.

external pressures with positive and negative internal pressures will result in four loading cases when wind is normal to the ridge.

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

Gust Effect Factor

This building meets the requirement of a low-rise building. Section 26.9.2 permits a low-rise building to be considered as rigid without calculating fundamental natural frequency.

For rigid structures, G can be calculated using Eq. 26.9-6 (Section 26.9.4 of the Standard) or alternatively taken as 0.85 (see Section 26.9.1). For simplicity, $G = 0.85$ is used in this example.

External Wall Pressure Coefficients

The pressure coefficients for the windward wall and for the side walls are 0.8 and –0.7 (see Fig. 27.4-1 of the Standard), respectively, for all L/B ratios.

The leeward wall pressure coefficient is a function of the L/B ratio. For wind normal to the ridge, $L/B = 200/250 = 0.8$; therefore, the leeward wall pressure coefficient is –0.5. For flow parallel to the ridge, $L/B = 250/200 = 1.25$; the value of C_p is obtained by linear interpolation. The wall pressure coefficients are summarized in **Table G5-3**.

External Roof Pressure Coefficients for Wind Normal to Ridge

The roof pressure coefficients for the MWFRS (**Table G5-4**) are obtained from Fig. 27.4-1 of the Standard. For the roof angle of 18.4°, linear interpolation is used to establish C_p. For wind normal to the ridge, $h/L = 36.7/200 = 0.18$; hence, only single linear interpolation is required. Note that interpolation is only carried out between values of the same sign.

Internal Pressure

Openings are uniformly distributed. An enclosed building can be assumed with the understanding that when a door or window fails in one wall, there may be extra uplift on the roof structure before openings in other walls can occur to relieve pressure. The Standard does not have guidance when a building is located outside of the wind-borne debris region. An enclosed building classification is assumed. The reduction factor of Section 26.11.1.1 is not applicable for enclosed buildings.

Table G5-3 External Wall Pressure Coefficient

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

Note: Bold indicates linear interpolation.

Table G5-4 Roof Pressure Coefficient for Wind Normal to Ridge

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

Note: Bold indicates linear interpolation.

Values for (GC_{pi}) for buildings are addressed in Section 26.11 and Table 26.11-1 of the Standard.

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

MWFRS Net Pressures

$$p = qGC_p - q_i(GC_{pi}) \qquad \text{(Eq. 27.4-1)}$$
$$p = q(0.85)C_p - 29.4(\pm 0.18)$$

where

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

Typical Calculation

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

$$p = 24.5(0.85)(0.8) - 29.4(\pm 0.18)$$
$$p = 11.4 \text{ psf with (+) internal pressure}$$
$$p = 21.9 \text{ psf with (–) internal pressure}$$

The net pressures for the MWFRS are summarized in **Table G5-5**.

Table G5-5 MWFRS Pressures for Wind Normal to Ridge

					Net pressure (psf) with	
Surface	z *(ft)*	q *(psf)*	G	C_p	*(+*GC_{pi}*)*	*(–*GC_{pi}*)*
Windward wall	0–15	24.5	0.85	0.8	11.4	21.9
	20	25.9	0.85	0.8	12.3	22.9
Leeward wall	All	29.4	0.85	–0.5	–17.8	–7.2
Side walls	All	29.4	0.85	–0.7	–22.8	–12.2
Windward	—	29.4	0.85	–0.36	–14.3	–3.7
Roof*				0.14	–1.8	8.8
Leeward roof	—	29.4	0.85	–0.57	–19.5	–8.9

Notes: q_h = 29.4 psf; $(GC_{pi}) = \pm 0.18$; $q_h(GC_{pi}) = \pm 5.3$ psf.

*Two loadings on windward roof and two internal pressures yield a total of four loading cases (see **Figs. G5-2** and **G5-3**).

Table G5-6 Roof Pressures for Wind Parallel to Ridge

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

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

External Roof Pressures for Wind Parallel to Ridge

For wind parallel to the ridge, $h/L = 36.7/250 = 0.147$ and $\theta < 10°$. The values of C_p for wind parallel to ridge are obtained from Fig. 27.4-1 of the Standard and are shown in **Tables G5-6** and **G5-7**.

Design Wind Load Cases

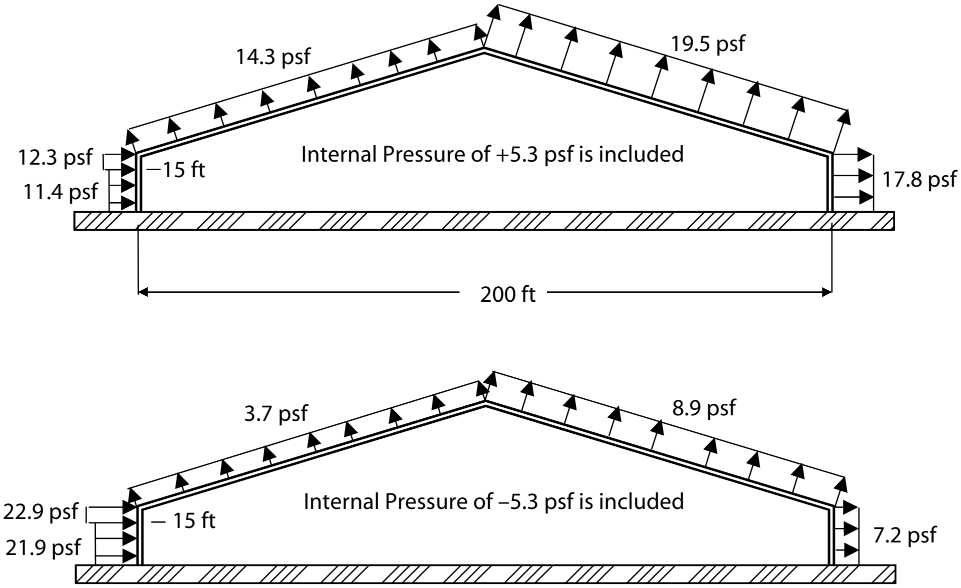
Section 27.4.6 of the Standard requires that any building whose wind loads have been determined under the provisions of Sections 27.4.1 and 27.4.2 shall be designed for wind load cases as defined in Fig. 27.4-8 of the Standard. Case 1 includes the loadings shown in **Figs. G5-2 through G5-5**. A combination of windward (P_W) and leeward (P_L) loads is applied for Load Cases 2, 3, and 4 as shown in Fig. 27.4-8 of the Standard. Section 27.4.6 of the Standard has an exception that if a building meets the requirements of

Table G5-7 MWFRS Pressures for Wind Parallel to Ridge

					Net pressure(psf) with	
Surface	z *(ft)*	q *(psf)*	G	C_p	*(+GC$_{pi}$)*	*(–GC$_{pi}$)*
Windward wall	0–15	24.5	0.85	0.8	11.4	21.9
	20	25.9	0.85	0.8	12.3	22.9
	30	28.2	0.85	0.8	13.9	24.5
	40	29.9	0.85	0.8	15.0	25.6
	53.3	31.7	0.85	0.8	16.3	26.8
Leeward wall	All	29.4	0.85	–0.45	–16.5	–5.9
Side walls	All	29.4	0.85	–0.7	–22.8	–12.2
Roof	0 to *h*	29.4	0.85	–0.9	–27.8	–17.6
	h to 2*h*	29.4	0.85	–0.5	–17.8	–7.2
	>2*h*	29.4	0.85	–0.3	–12.8	–2.2

Notes: Roof distances are from windward edge. $q_h = 29.4$ psf; $(GC_{pi}) = \pm 0.18$; $h = 36.7$ ft; $q_h(GC_{pi}) = \pm 5.3$ psf.

Fig. G5-2
Net design pressures for MWFRS when wind is normal to ridge with negative windward external roof pressure coefficient



Section D1.1 of Appendix D, only Load Cases 1 and 3 need to be considered. There is not enough structural information given in this example to assess flexibility of roof diaphragm. The structural designer will need to make a judgment for each building designed.

Design Pressures for Components and Cladding

Chapter 30, part 1, of the Standard is used to obtain the design pressures for components and cladding. The equation is

$$p = q_h[(GC_p) - (GC_{pi})] \qquad \text{(Eq. 30.4-1)}$$

Fig. G5-3
Net design pressures for MWFRS when wind is normal to ridge with positive windward external roof pressure coefficient

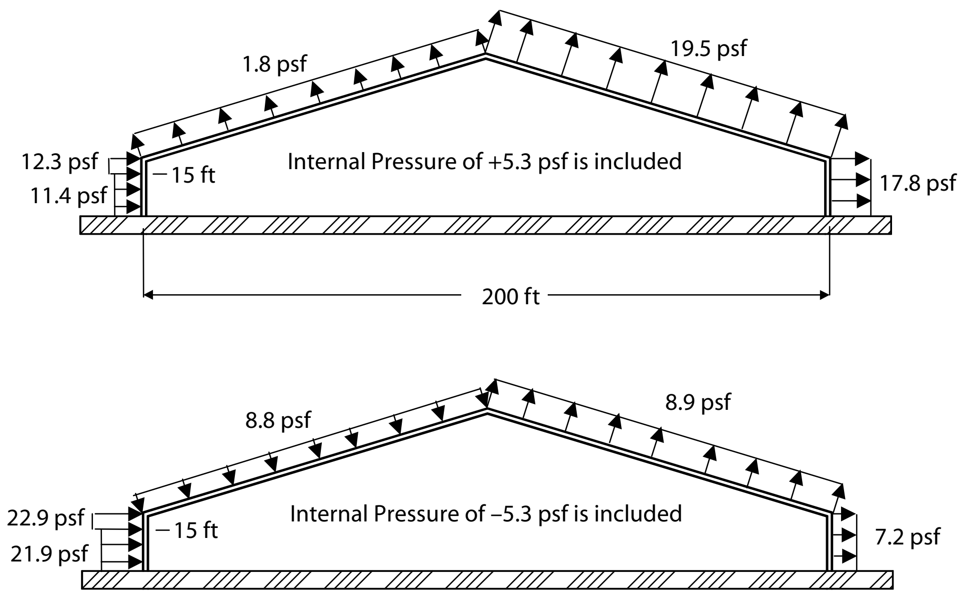
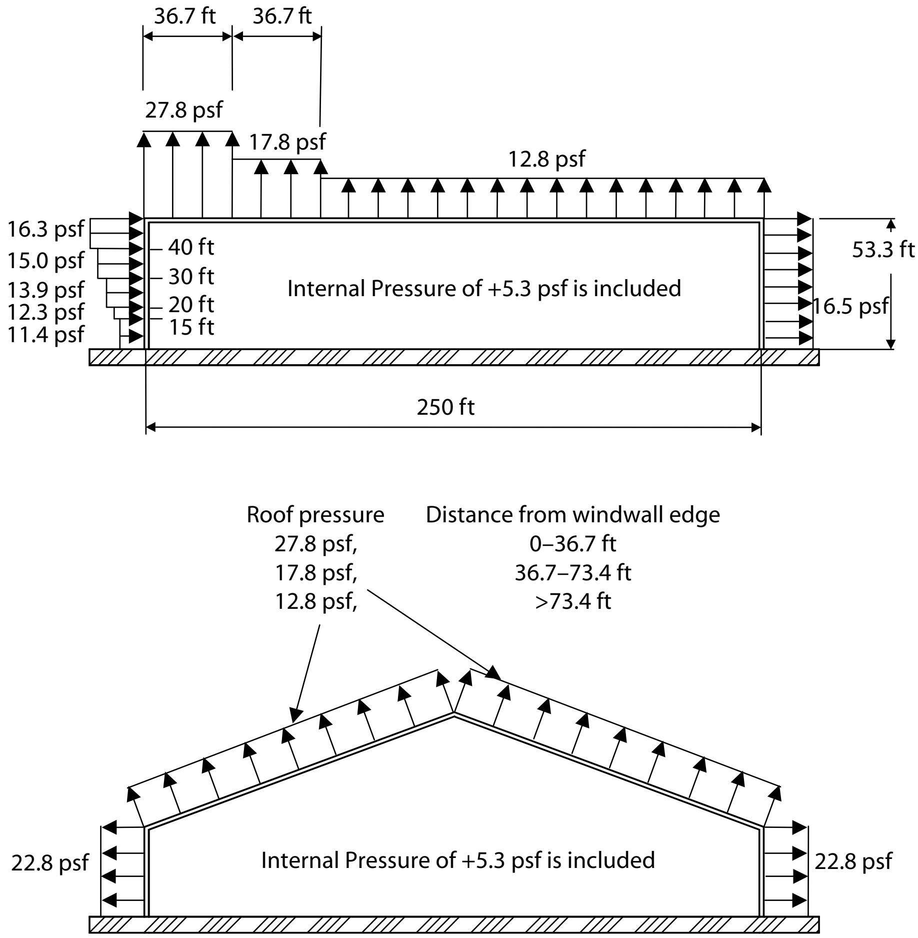


Fig. G5-4
Net design pressures for MWFRS when wind is parallel to ridge with positive internal pressure



where

q_h = 29.4 psf
(GC_p) = values obtained from Fig. 30.4-1 and 30.4-2B
(GC_{pi}) = ±0.18 for this building

Wall Pressures for C&C

The pressure coefficients (GC_p) (**Table G5-8**) are a function of effective wind area. The definitions of effective wind area for a component or cladding panel is the span length multiplied by an effective width that need not be less than one-third the span length; however, for a fastener it is the area tributary to an individual fastener.

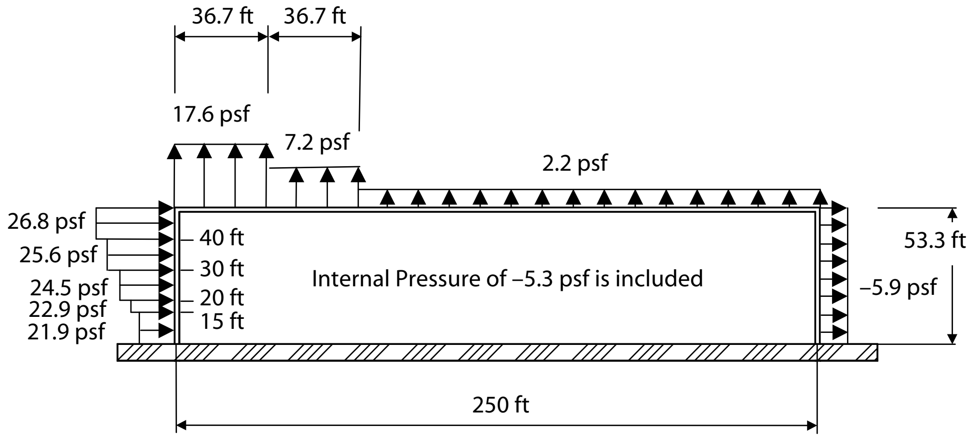
Girt

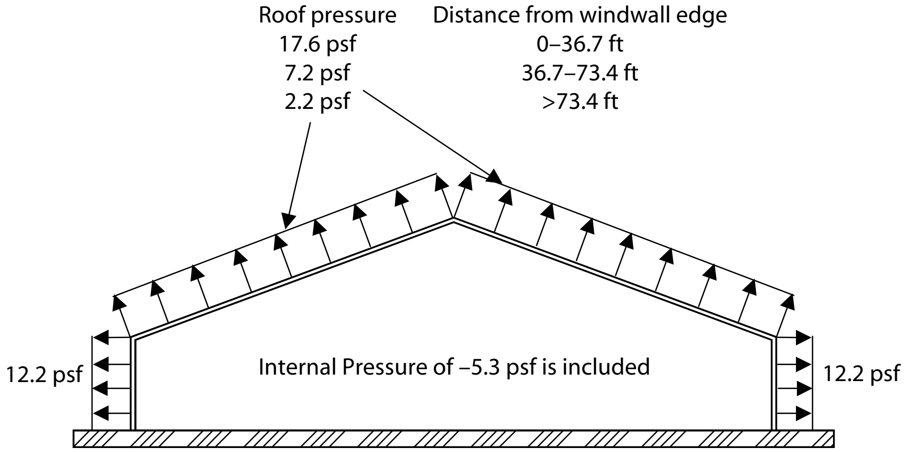
larger of $A = 25(6.67) = 167\ \text{ft}^2$
or $A = 25(25/3) = 208\ \text{ft}^2$ (controls)

Wall Panel

larger of $A = 6.67(2) = 13.3\ \text{ft}^2$
or $A = 6.67(6.67/3) = 14.8\ \text{ft}^2$ (controls)

Fig. G5-5
Net design pressures for MWFRS when wind is parallel to ridge with negative internal pressure





Fastener

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

Typical calculations of design pressures for girt in Zone 4 are shown as follows, and wall C&C pressures are summarized in **Table G5-9**.

Table G5-8 Wall Coefficients for Components and Cladding in Figure 30.4-1

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

Note: Coefficients from Fig. 30.4-1 of the Standard. (GC_p) values are obtained using equations in chapter 2 of this guide. "Other" C&C can be doors, windows, etc.

Table G5-9 Net Controlling Wall Component Pressures for Components and Cladding

	Controlling design pressures (psf)			
	Zone 4		Zone 5	
C&C	*Positive*	*Negative*	*Positive*	*Negative*
Girt	27.9	–30.9	27.9	–32.6
Panel	33.8	–36.8	33.8	–44.7
Fastener	34.7	–37.6	34.7	–46.4
$A \leq 10$ ft^2	34.7	–37.6	34.7	–46.4
$A \geq 500$ ft^2	25.9	–28.8	25.9	–28.8

For maximum negative pressure

$p = 29.4[(-0.87) - (\pm 0.18)]$
$p = -30.9$ psf with positive internal pressure (controls)
$p = -20.3$ psf with negative internal pressure

For maximum positive pressure

$p = 29.4[(0.77) - (\pm 0.18)]$
$p = 17.3$ psf with positive internal pressure
$p = 27.9$ psf with negative internal pressure (controls)

Roof Pressures for C&C

Effective wind areas of roof C&C (**Table G5-10**) are from Fig 30.4-2B of the Standard:

Purlin

larger of $A = 25(5) = 125$ ft^2
or $A = 25(25/3) = 208$ ft^2 (controls)

Panel

larger of $A = 5(2) = 10$ ft^2 (controls)
or $A = 5(5/3) = 8.3$ ft^2

Table G5-10 Roof Pressure Coefficients for Components and Cladding, $7° < \theta \leq 27°$

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

Note: "Other" C&C can be skylights, etc.

Fastener

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

Typical calculations of design pressures for a purlin in Zone 1 are as follows, and roof C&C pressures are summarized in **Table G5-11**.

For maximum negative pressure

$p = 29.4\ [(-0.8) - (\pm 0.18)]$
$p = -28.8$ psf with positive internal pressure (controls)
$p = -18.2$ psf with negative internal pressure

For maximum positive pressure

$p = 29.4\ [(0.3) - (\pm 0.18)]$
$p = 3.5$ psf with positive internal pressure
$p = 14.1$ psf with negative internal pressure
$p = 16$ psf minimum net pressure (controls) (Section 30.2.2 of the Standard)

Special Case of Girt that Transverses Zones 4 and 5

Width of Zone 5

smaller of	$a = 0.1(200) = 20$ ft
or	$a = 0.4(36.7) = 14.7$ ft (controls)
but not less than	$0.04(200) = 8$ ft
or	3 ft

Weighted average design pressure

$$p = \frac{14.7(-32.6) + 10.3(-30.9)}{25} = 31.9\ \text{psf}$$

This procedure of using a weighted average may be used for other C&C.

Table G5-11 Net Controlling Roof Component Pressures

	Controlling design pressures (psf)			
	Positive	*Negative*		
Component	*Zones 1, 2, and 3*	*Zone 1*	*Zone 2*	*Zone 3*
Purlin	16.0*	–28.8	–40.6	–64.1
Panel	20.0	–31.8	–55.3	–81.7
Fastener	20.0	–31.8	–55.3	–81.7
$A \le 10\ \text{ft}^2$	20.0	–31.8	–55.3	–81.7
$A \ge 500\ \text{ft}^2$	16.0*	–28.8	–40.6	–64.1

*Minimum net pressure controls (Section 30.2.2 of the Standard).

Special Case of Interior Strut Purlin

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

Component pressure

End bay purlin located in Zones 1 and 2
Width of Zone 2, $a = 14.7$ ft
Weighted average design pressure: $\frac{14.7(-40.6)+10.3(-28.8)}{25} = -35.6$ psf

(Purlins in Zones 2 and 3 will have higher pressures.)

MWFRS Load

Figure G5-3 shows design pressure on an end wall with wind parallel to the ridge with positive internal pressure (consistent with high uplift on the purlin). Assuming that the end wall is supported at the bottom and at the roof line, the effective axial load on an end bay purlin can be determined.

Combined Design Loads on Interior Strut Purlin

Figure G5-6 (top) shows combined design loads on the interior strut purlin. Note that many metal building manufacturers support the top of the wall panels with the eave strut purlin (see bottom of **Fig. G5-6**). For this case, the eave purlin also serves as a girt, and the negative wall pressures of Zones 5 and 4 would occur for the same wind direction as the maximum negative uplift pressures on the purlin (refer to Zones 3 and 2). Thus, in this instance, the correct load combination would involve biaxial bending loads based on C&C pressures combined with the MWFRS axial load.

Comment

The pressures determined are limit state design pressures for strength design. Section 2.3 of the Standard indicates load factor for wind load to be $1.0W$ for loads determined in this example. If allowable stress design is to be used, the load factor for wind load is $0.6W$ as shown in Section 2.4 of the Standard.

5.2 Simplified Method for Low-Rise Building

This example illustrates the use of the envelope procedure for low-rise buildings to determine design pressures for the MWFRS. For this purpose, the building used has the same dimensions as the building in **Section 5.1** of this guide. The design pressures on C&C also will be the same as in **Section 5.1** of this guide. The building is shown in **Fig. G5-1**. The building data are as in **Table G5-1**, except that the openings in the framing are uniformly distributed.

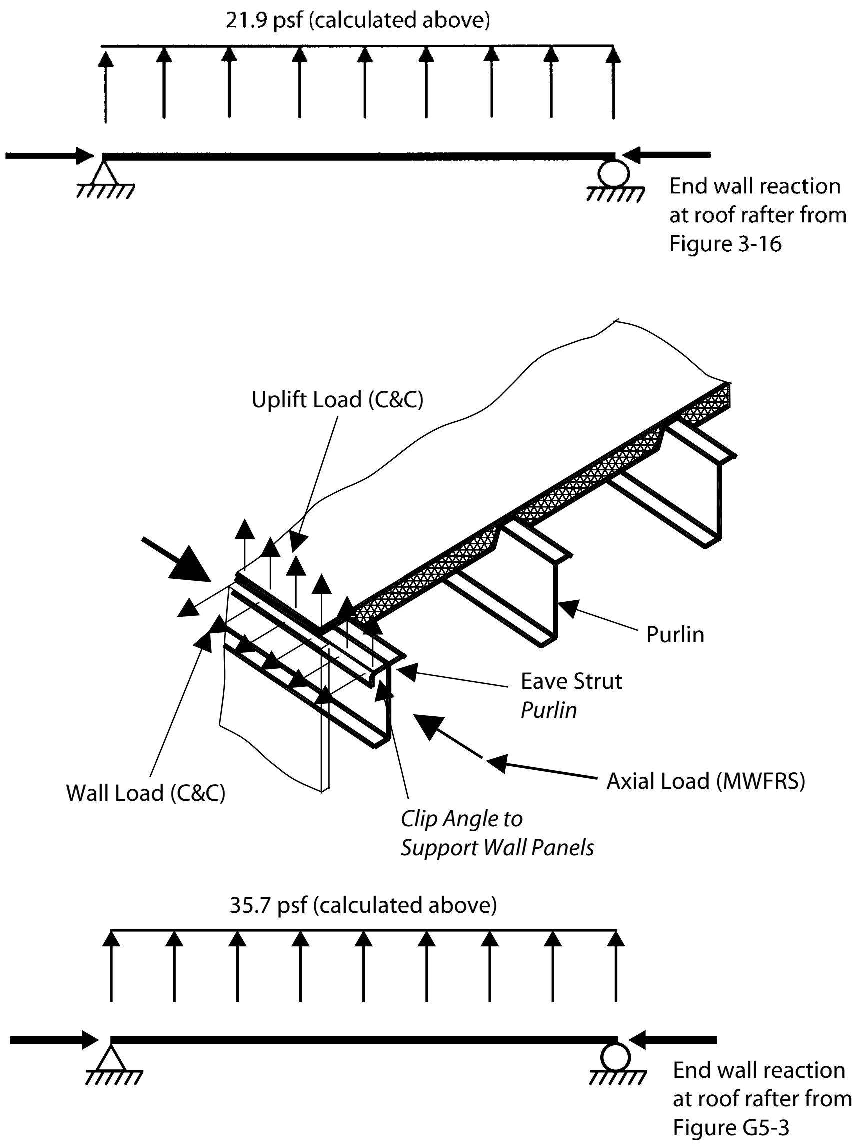


Fig. G5-6
Combined design loads on interior strut purlin. Top: eave strut purlin supporting roof and wall panels. Bottom: combined uplift and axial design loads

Low-Rise Building

Section 26.2 of the Standard specifies two requirements for a building to qualify as a low-rise building:

1. mean roof height has to be less than or equal to 60 ft, and
2. mean roof height does not exceed the least horizontal dimension.

The building of this example has h = 36.7 ft, and so it qualifies as a low-rise building and the alternate provisions of Section 28.4 may be used.

Exposure, Building Classification, and Basic Wind Speed

These are the same as for **Section 5.1** of this guide:

Exposure C
Category II
Enclosed building (openings uniformly distributed)
V = 115 mph

Velocity Pressure

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

Mean roof height $h = 36.7$ ft

The velocity pressures are computed using

$$q_h = 0.00256\ K_h K_{zt} K_d V^2 \text{ (psf)} \qquad \text{(Eq. 28.3-1)}$$

where

q_h = velocity pressure at mean roof height, h
K_h = 1.02 for Exposure C (see Table 28.3-1 of the Standard)
K_{zt} = 1.0 topographic factor (see Section 26.8.2 of the Standard)
K_d = 0.85 (see Table 26.6-1 of the Standard)
V = 115 mph basic wind speed (see Fig. 26.5-1A of the Standard)

Therefore

$$q_h = 0.00256(1.02)(1.0)(0.85)(115)^2 = 29.4 \text{ psf}$$

Design Pressures for the MWFRS

The equation for the determination of design wind pressures for MWFRS using the envelope procedure for low-rise buildings is given by Eq. 28.4-1 in Section 28.4.1 of the Standard:

$$p = q_h[(GC_{pf}) - (GC_{pi})] \qquad \text{(Eq. 28.4-1)}$$

where

q_h = velocity pressure at mean roof height calculated above
(GC_{pf}) = external pressure coefficients from Fig. 28.4-1 of the Standard
(GC_{pi}) = internal pressure coefficient from Table 26.11-1 of the Standard

The building must be designed for all wind directions using the eight loading patterns shown in Fig. 28.4-1 of the Standard. For each of these patterns, both positive and negative internal pressures must be considered, resulting in a total of 16 separate loading conditions. However, if the building is symmetrical, the number of separate loading conditions will be reduced to eight (two directions of MWFRS being designed for normal load and torsional load cases, or a total of four load cases, plus one windward corner, and two internal pressures). The load patterns are applied to each building corner in turn as the reference corner.

Table G5-12 Transverse Direction ($\theta = 18.4°$) by Building Surface

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

Pressure coefficient is calculated by linear interpolation.

External Pressure Coefficients

The roof and wall coefficients are functions of the roof slope, θ (see **Tables G5-12** and **G5-13**).

Width of end zone surface

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

Internal Pressure Coefficients

Openings are assumed to be evenly distributed in the walls, and since Memphis, Tennessee, is not located in a hurricane prone region, the building qualifies as an enclosed building (see Section 26.10 of the Standard). The internal pressure coefficients are given from Table 26.11-1 of the Standard as $(GC_{pi}) = \pm 0.18$.

Design Wind Pressure

Design wind pressures in the transverse and longitudinal directions are shown in **Tables G5-14** and **G5-15**.

Calculation for Surface 1

$$p = 29.4\ [0.52 - (\pm 0.18)] = +10.0 \text{ or } +20.6$$

Application of Pressures on Building Surfaces 2 and 3

Note 8 of Fig. 28.4-1 of the Standard states that when the roof pressure coefficient, GC_{pf}, is negative in Zone 2, it shall be applied in Zone 2 for a distance from the edge of the roof equal to 0.5 times the horizontal dimension of the building measured parallel to the direction of the MWFRS being designed or $2.5h$, whichever is less. The remainder of Zone 2 that extends

Table G5-13 Longitudinal Direction ($\theta = 0$) by Building Surface

	Building surface											
	1	*2*	*3*	*4*	*5*	*6*	*1E*	*2E*	*3E*	*4E*	*5E*	*6E*
GC_{pf}	–0.45	0.69	0.37	0.45	0.40	0.29	0.48	1.07	0.53	0.48	0.61	0.43

Table G5-14 Design Wind Pressures, Transverse Direction

Building surface	*(GC$_{pf}$)*	*Design pressure (psf)* (+GC$_{pi}$)	*Design pressure (psf)* (–GC$_{pi}$)
1	0.52	10.0	20.6
2	–0.69	–25.6	–15.0
3	–0.47	–19.1	–8.5
4	–0.42	–17.6	–7.0
1E	0.78	17.6	28.2
2E	–1.07	–36.8	–26.2
3E	–0.67	–25.0	–14.4
4E	–0.62	–23.5	–12.9

to the ridge line shall use the pressure coefficient GC_{pf} for Zone 3. Thus, the distance from the edge of the roof is the smaller of:

$$0.5(200) = 100 \text{ ft for transverse direction}$$
$$0.5(250) = 125 \text{ ft for longitudinal direction}$$
$$\text{or} \quad (2.5)(36.7) = 92 \text{ ft for both directions (controls)}$$

Therefore, Zone 3 applies over a distance of 100 – 92 = 8 ft in what is normally considered to be Zone 2 (adjacent to ridge line) for transverse direction and 125 – 92 = 33 ft for longitudinal direction.

Loading Cases

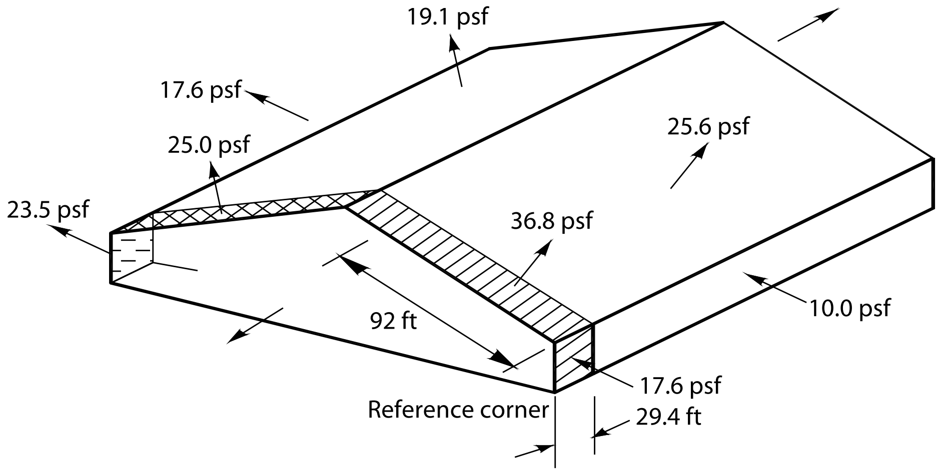
Because the building is symmetrical, the four loading cases provide all the required combinations provided the design is accomplished by applying

Table G5-15 Design Wind Pressures, Longitudinal Direction

Building surface	*(GC$_{pf}$)*	*Design pressure (psf)* (+GC$_{pi}$)	*Design pressure (psf)* (–GC$_{pi}$)
1	–0.45	–18.5	–7.9
2	–0.69	–25.6	–15.0
3	–0.37	–16.2	–5.6
4	–0.45	–18.5	–7.9
5	0.40	6.5	17.1
6	–0.29	–13.8	–3.2
1E	–0.48	–19.4	–8.8
2E	–1.07	–36.8	–26.2
3E	–0.53	–20.9	–10.3
4E	–0.48	–19.4	–8.8
5E	0.61	12.6	23.2
6E	–0.43	–17.9	–7.4

Fig. G5-7
Design pressures for transverse direction with positive internal pressure

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



loads for each of the four corners. The load combinations illustrated in **Figs. G5-7 through G5-10** are to be used to design the rigid frames, the "wind truss" spanning across the building in the 200-ft direction, and the rod/cable bracing in the planes of the walls (see **Fig. G5-1** in **Section 5.1** of this guide).

Torsional Load Cases

Since the mean roof height, h = 36.7 ft, is greater than 30 ft and if the roof diaphragm is assumed to be rigid, torsional load cases need to be considered (see exception in Note 5 in Fig. 28.4-1 of the Standard if building is designed with flexible diaphragm). Pressures in "T" zones are 25 percent of the full design pressures; the "T" zones are shown in Fig. 28.4-1 of the Standard. Other surfaces will have the full design pressures. The "T" zone pressures with positive and negative internal pressures for transverse and longitudinal directions are shown in **Tables G5-16** and **G5-17**, respectively.

Figures G5-11 through G5-14 show design pressure cases for one reference corner; these cases are to be considered for each corner.

Fig. G5-8
Design pressures for transverse direction with negative internal pressure

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

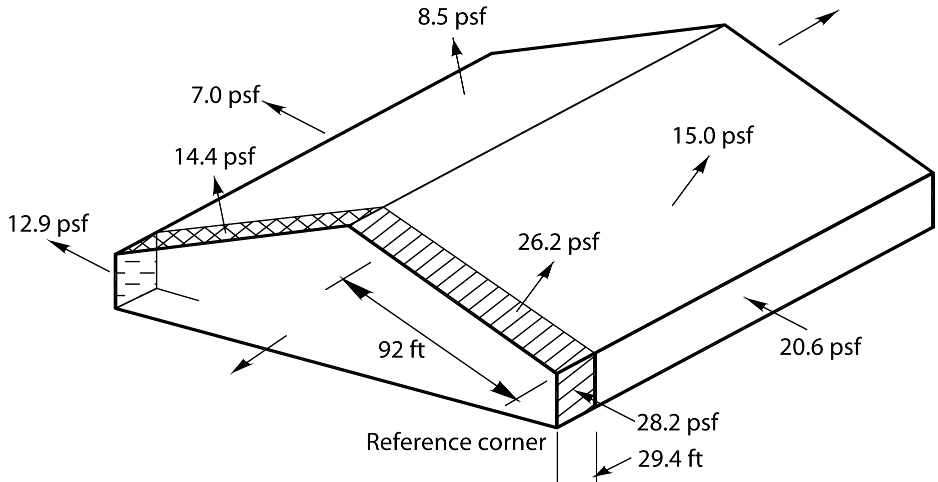


Fig. G5-9
Design pressures for longitudinal direction with positive internal pressure

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

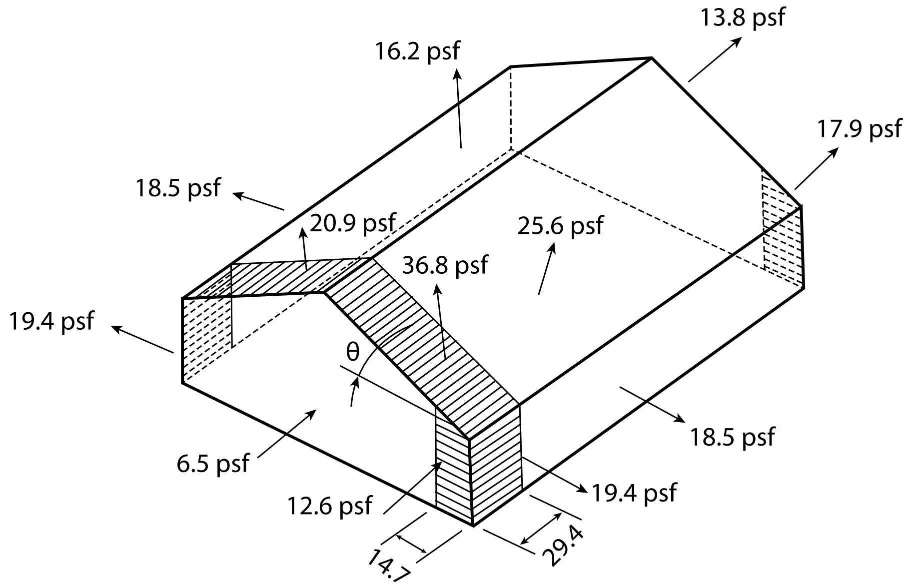


Fig. G5-10
Design pressures for longitudinal direction with negative internal pressure

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

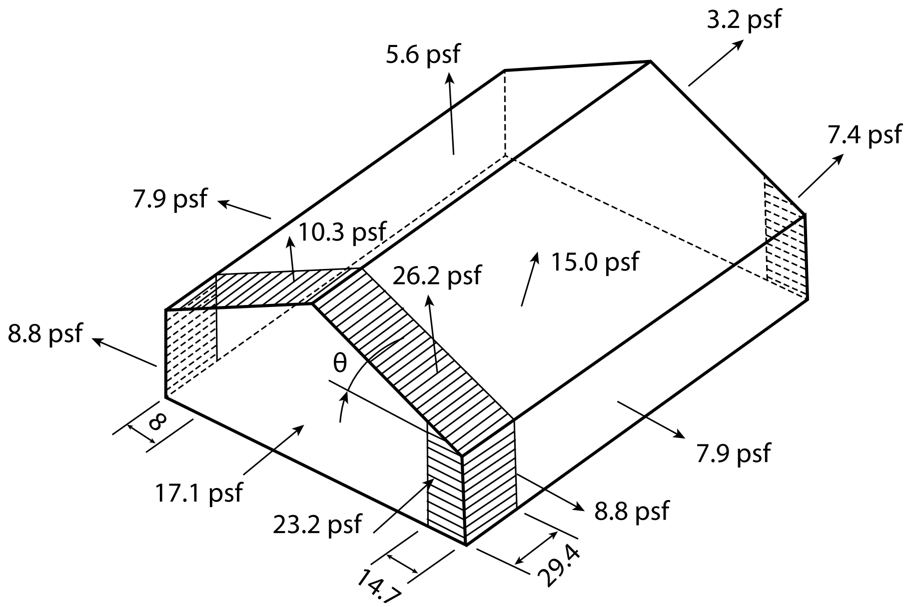


Table G5-16 Design Wind Pressure for Zone "T," Transverse Direction

	Design pressures (psf)	
Building surface	*(+GC$_{pi}$)*	*(–GC$_{pi}$)*
1T	2.5	5.2
2T	–6.4	–3.8
3T	–4.8	–2.1
4T	–4.4	–1.8

Table G5-17 Design Wind Pressure for Zone "T," Longitudinal Direction

	Design pressures (psf)	
Building surface	*(+GC_{pi})*	*(–GC_{pi})*
1T	–4.6	–2.0
2T	–6.4	–3.8
3T	–4.0	–1.4
4T	–4.6	–2.0
5T	1.6	4.3
6T	–3.4	–0.8

Fig. G5-11
Torsional load case for transverse direction with positive internal pressure

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

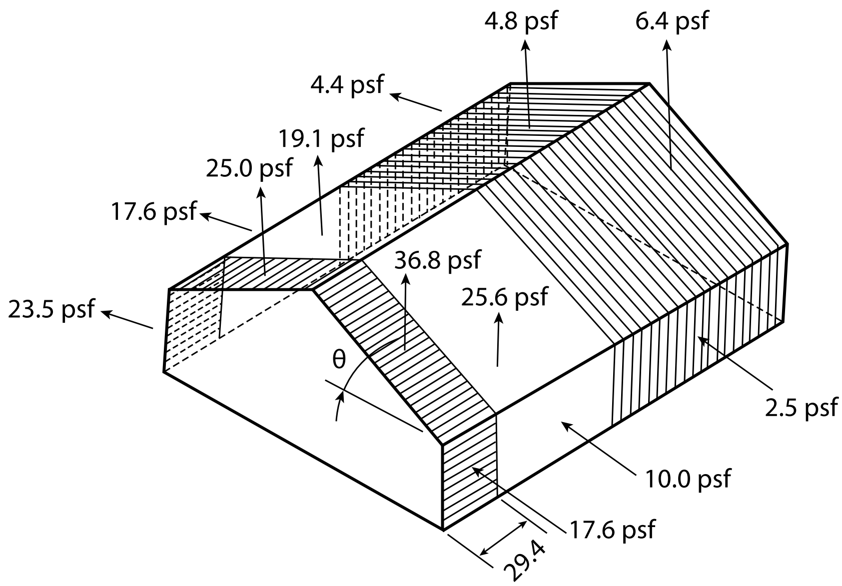


Fig. G5-12
Torsional load case for transverse direction with negative internal pressure

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

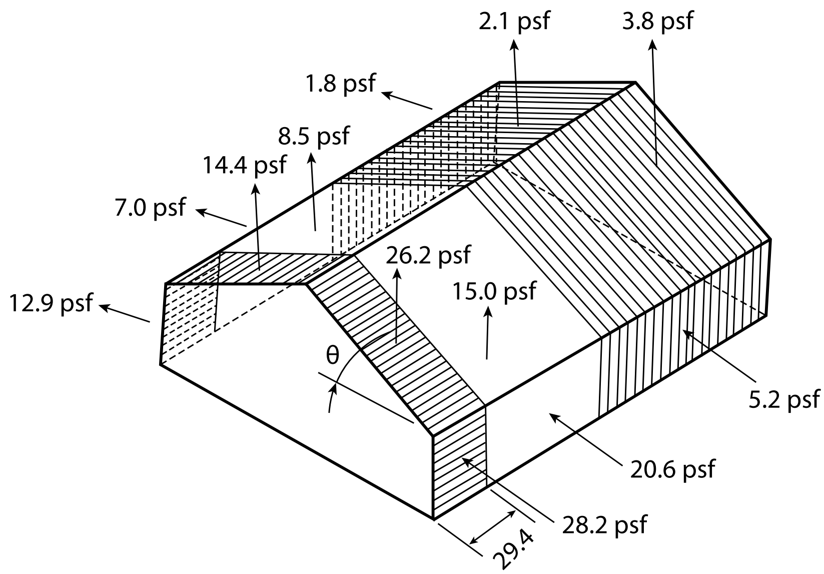


Fig. G5-13
Torsional load case for longitudinal direction with positive internal pressure

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

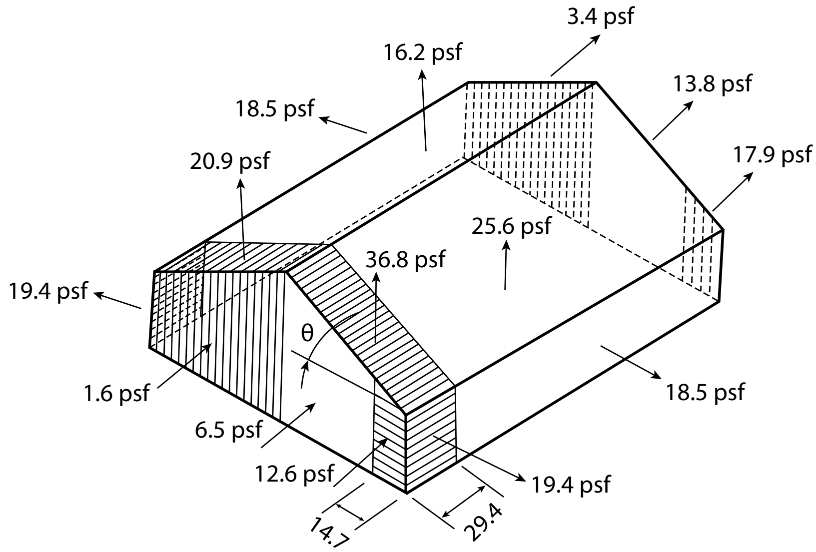
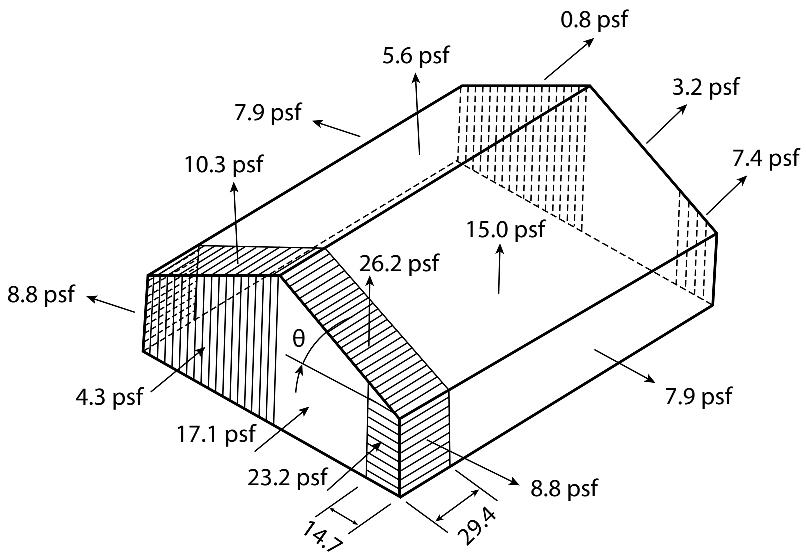


Fig. G5-14
Torsional load case for longitudinal direction with negative internal pressure

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



Design Pressures for Components and Cladding

The design pressures for components and cladding (C&C) are the same as shown in **Section 5.1** of this guide.

Comment

The pressures determined are limit state design pressures for strength design. Section 2.3 of the Standard indicates load factor for wind load to be 1.0*W* for loads determined in this example. If allowable stress design is to be used, the load factor for wind load is 0.6*W* as shown in Section 2.4 of the Standard.

Chapter 6

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 **Fig. G6-1**, and data are shown in **Table G6-1**.

The example in **Section 6.1** uses the MWFRS Directional Procedure and follows Table 27.2-1 of ASCE 7-10 for buildings of all heights. The same building is illustrated in **Section 6.2**, using part 2 of the Simplified Envelope Procedure in chapter 28 of ASCE 7-10.

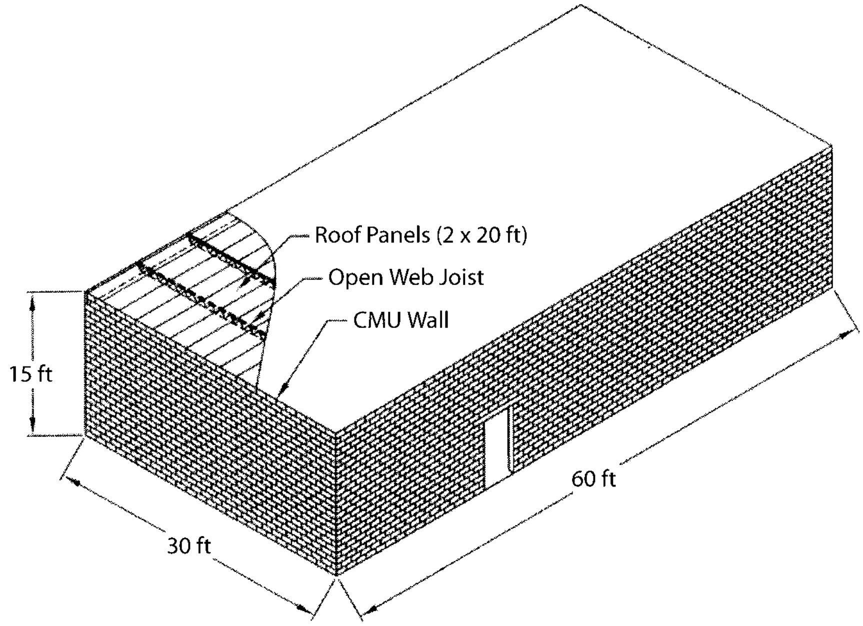
Building Classification

The building function is shops. It is not considered an essential facility. Building Category II is appropriate; see Table 1.5-1 of the Standard. The wind speed map to be used with this category of building is in Fig. 26.5-1A.

Basic Wind Speed

Selection of the basic wind speed is addressed in Section 26.5 of the Standard. Basic wind speed for Corpus Christi, Texas, is 148 mph (interpolating between isotachs on Fig. 26.5-1A of the Standard).

Fig. G6-1
Building characteristics for commercial building with concrete masonry unit walls



Exposure

The building is located on flat and open terrain. It does not fit Exposures B or D; therefore, use Exposure C (Section 26.7.3 of the Standard).

Velocity Pressure

The velocity pressures are computed using

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

Table G6-1 Data for Commercial Building with Concrete Masonry Unit Walls

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 panels 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
Rooftop equipment	1–10 ft (face normal to wind is assumed) × 5 ft × 5 ft high air handler
	1–3 ft diameter × 10 ft high smooth surface vent pipe

where

K_z = 0.85 from Table 27.3-1 of the Standard; for 0 to 15 ft, there is only one value: $K_z = K_h$
K_{zt} = 1.0 for homogeneous topography (see Section 26.8 of the Standard)
K_d = 0.85 for buildings (see Table 26.6-1 of the Standard)
V = interpolation yields 148 mph (see Fig. 26.5-1A of the Standard)

Therefore

$$q_z = 0.00256(0.85)(1.0)(0.85)(148)^2 = 40.5 \text{ psf}$$
$$q_h = 40.5 \text{ psf for } h = 15 \text{ ft}$$

Gust Effect Factor

The building is considered a rigid structure. Section 26.9.4 of the Standard permits use of $G = 0.85$.

If the detailed procedure for a rigid structure is used (Section 26.9.4 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 **Section 4.1** of this guide).

Use $G = 0.85$ for this example.

Internal Pressure Coefficient

The building is located in a hurricane prone area as well as in a wind-borne debris region (see definition in Section 26.2 and Section 26.10.3.1 of the Standard). The glazing protection requirements are given in Section 26.10.3.2 of the Standard.

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

Use $(GC_{pi}) = +0.18$ and -0.18 for enclosed buildings (see Table 26.11-1 of the Standard).

6.1 Analytical Procedure

Design Pressures for the MWFRS

Design wind pressures are determined using the equation

$$p = q\, G\, C_p - q_i\, (GC_{pi}) \qquad \text{(Eq. 27.4-1)}$$

where

q = q_z for windward wall (40.5 psf for this example)
q = q_h for leeward wall, side walls, and roof (40.5 psf for this example)
G = 0.85
C_p = values of external pressure coefficients
q_i = q_h for enclosed building (40.5 psf)
(GC_{pi}) = +0.18 and –0.18

The values of external pressure coefficients are obtained from Fig. 27.4-1 of the Standard.

Wall Pressure Coefficient

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 Pressure Coefficient

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 Sections 2.3 and 2.4 of the Standard. For brevity, loading for this value is not shown in this example.

MWFRS Pressures

Windward wall

$$p = 40.5(0.85)(0.8) - 40.5(\pm 0.18) = 27.5 \pm 7.3 \text{ psf}$$

Leeward wall

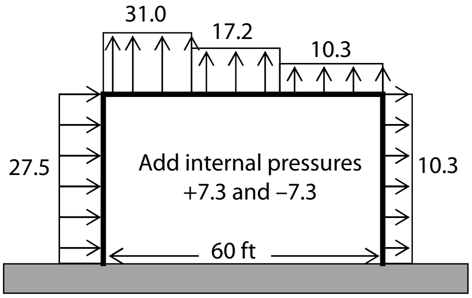
$p = 40.5(0.85)(-0.5) - 40.5(\pm 0.18) = -17.2 \pm 7.3$ psf for wind normal to 60 ft
$p = 40.5(0.85)(-0.3) - 40.5(\pm 0.18) = -10.3 \pm 7.3$ psf for wind normal to 30 ft

Roof – First value

$$\begin{aligned} p &= 40.5(0.85)(-0.9) - 40.5(\pm 0.18) \\ &= -31.0 \pm 7.3 \text{ psf for 0 to 15 ft} \\ &= -17.2 \pm 7.3 \text{ psf for 15 to 30 ft} \\ &= -10.3 \pm 7.3 \text{ psf for > 30 ft} \end{aligned}$$

The MWFRS design pressures for two directions are shown in **Figs. G6-2** and **G6-3**. 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.

Fig. G6-2
Design pressures for MWFRS when wind is normal to 30-ft wall



Design Wind Load Cases

According to Section 27.4.6 of the Standard, this building shall be designed for the wind load Cases 1 and 3 as defined in Fig. 27.4-8.

Load Case 1 has been considered as explained. **Figure G6-4** is for Load Case 3 where the windward and leeward pressures are taken as 75 percent of the specified values. Internal pressures cancel, so they are not shown.

Design Pressures for Components & Cladding

Design wind pressures for C&C are determined using the provisions of chapter 30, part 1 of the Standard. The equation is

$$p = q_h[(GC_p) - (GC_{pi})] \qquad \text{(Eq. 30.4-1)}$$

where

q_h = 40.5 psf
(GC_p) = values obtained from Fig. 30.4-1 and 30.4-2A of the Standard; they are a function of effective area and zone
(GC_{pi}) = +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 26.2 of the Standard: "an effective width that need not be less than one-third the span length."

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

Fig. G6-3
Design pressures for MWFRS when wind is normal to 60-ft wall

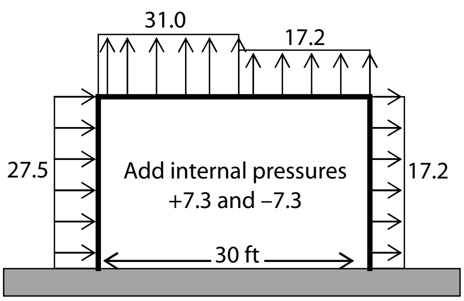
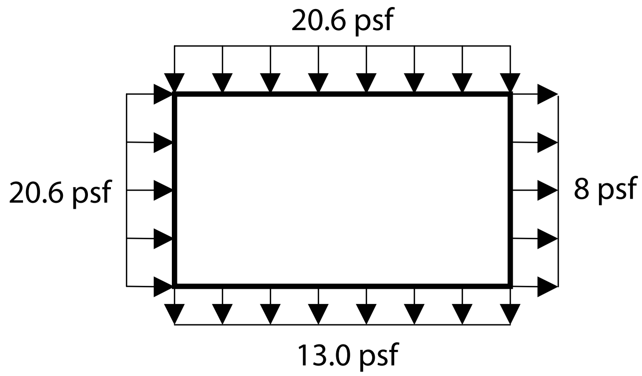


Fig. G6-4
Load Case 3



Pressure coefficients for C&C of walls, $A = 75\ \text{ft}^2$
$(GC_p) = +0.85$ and -1.09 for Corner Zone 5
$= +0.85$ and -0.95 for Interior Zone 4

In Fig. 30.4-1 of the Standard, Note 5 suggests that the pressure coefficient values for walls can be reduced by 10 percent for roof slope of 10° or less; thus calculations below take 90 percent of the calculated or graphed values. 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

$$p = 40.5[(-1.09)(0.9) - (\pm 0.18)] = -47.0 \text{ psf}$$
$$p = 40.5[(0.85)(0.9) - (\pm 0.18)] = +38.3 \text{ psf}$$

Interior Zone 4

$$p = 40.5[(-0.95)(0.9) - (\pm 0.18)] = -41.9 \text{ psf}$$
$$p = 40.5[(0.85)(0.9) - (\pm 0.18)] = +38.3 \text{ psf}$$

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 (Fig. 30.4-2A)

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^2
or $A = 30 \times (30/3) = 300$ ft^2 (controls)

The values of (GC_p) are obtained from Fig. 30.4-2A of the Standard or from equations of the graphs using effective area $A = 300$ ft^2.

Interior Zone 1

$p = 40.5[-0.9 \pm 0.18] = -43.7$ psf
$p = 40.5[+0.2 \pm 0.18] = +15.4$ psf

Eave Zone 2 and corner Zone 3

$p = 40.5[-1.1 \pm 0.18] = -51.8$ psf
$p = 40.5[+0.2 \pm 0.18] = +15.4$ 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 $A = 5 \times 2 = 10$ ft^2 (controls)
or $A = 5 \times (5/3) = 8$ ft^2 (width of Zones 2 and 3, $a = 3$ ft)

Interior Zone 1

$p = 40.5[-1.0 \pm 0.18] = -47.8$ psf
$p = 40.5[+0.3 \pm 0.18] = +19.4$ psf

Eave Zone 2

$p = 40.5[-1.8 \pm 0.18] = -80.2$ psf
$p = 40.5[+0.3 \pm 0.18] = +19.4$ psf

Corner Zone 3

$p = 40.5[-2.8 \pm 0.18] = -120.7$ psf
$p = 40.5[+0.3 \pm 0.18] = +19.4$ psf

Notes:

- Internal pressure coefficient of +0.18 or –0.18 is used to give critical pressures.
- The roof panel fastener design pressures will be the same as the roof metal panel since values of (GCp) are the same for wind effective areas less than 10 ft^2.

Design Forces on Rooftop Equipment

Design wind forces are determined from the provisions of Section 29.5.1 of the Standard.

Lateral Force

$$F_h = q_h(GC_r)A_f \qquad \text{(Eq. 29.5-2)}$$

where

q_h = 40.5 psf

GC_r = 1.9 for rooftop structures and equipment with $A_f < (0.1\ Bh)$ in accordance with Section 29.5.1 of the Standard

A_f = projected area normal to the wind

Rectangular air handler

$$A_f = 10 \text{ ft} \times 5 \text{ ft} = 50 \text{ ft}^2$$

A_f must be compared to 0.1 Bh in order to determine the magnitude of the magnification factor. B = the face of the building normal to the wind, therefore if the windward wall is 60 ft, 0.1 Bh = 0.1(60)(15) = 90 ft^2; therefore, $A_f < 0.1\ Bh$ and the magnification factor = 1.9. Thus, lateral force,

$$F_h = 40.5(1.9)(50) = 3{,}848 \text{ lb}$$

Round vent pipe

$$A_f = 3 \text{ ft} \times 10 \text{ ft} = 30 \text{ ft}^2$$

$A_f < 0.1\ Bh$, therefore the magnification factor = 1.9. Thus, lateral force,

$$F_h = 40.5(1.9)(30) = 2{,}308 \text{ lb}$$

Vertical Uplift Force

$$F_v = q_h(GC_r)\ A_r \qquad \text{(Eq. 29.5-3)}$$

Rectangular air handler

$$A_r = 10 \text{ ft} \times 5 \text{ ft} = 50 \text{ ft}^2 < 0.1\ Bh = 90 \text{ ft}^2\text{; hence } (GC_r) = 1.5$$

Thus, uplift force F_v = 40.5(1.5)(50) = 3,038 lb

Round vent pipe

$$A_r = 7.1 \text{ ft}^2 < 90 \text{ ft}^2\text{; hence } (GC_r) = 1.5$$

Thus uplift force F_v = 40.5(1.5)(7.1) = 431 lb

Comment

The pressures determined are limit state design pressures for strength design. Section 2.3 of the Standard indicates load factor for wind load to be 1.0W for loads determined in this example. If allowable stress design is to be used, the load factor for wind load is 0.6W as shown in Section 2.4 of the Standard.

6.2 Simplified Method for Low-Rise Buildings

In this example, design wind pressures for the low-rise CMU building are determined using the simplified method of Section 28.5, part 2, of the Standard. Data for the building are shown in **Table G6-1**.

In order to use the simplified procedure, all conditions of Section 28.6.2 of the Standard must be satisfied:

1. It is a simple diaphragm building.
2. The 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 should be designed to conform to the wind-borne debris provisions of Section 26.10.3 of the Standard.
4. It has a regular shape.
5. It is a rigid building because it meets the definition of low-rise building; see Section 26.2 for definition and Section 26.9.2 for "permitted to be considered rigid."
6. There is no expansion joint.
7. It has an approximately symmetrical cross section in each direction with a flat roof, and
8. The building is exempted from torsional load cases as indicated in Note 5 of Fig. 28.4-1 in the Standard.

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

Building Classification

Building Risk Category II is appropriate. Wind speed map in Fig. 26.5-1A is used.

Basic Wind Speed

Basic wind speed for Corpus Christi, Texas, is 148 mph (interpolating between isotachs on Fig. 26.5-1A 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 (Section 26.7.3 of the Standard). Note that wind pressure values given in Figs. 28.6-1 and 30.5-1 of the Standard are for Exposure B.

Height and Exposure Adjustment Coefficient λ

From Fig. 28.6-1 of the Standard, $\lambda = 1.21$.

Topographic Factor, K_{zt}

From Section 26.8.2 of the Standard, $K_{zt} = 1.0$.

Design Wind Pressures for MWFRS

See **Table G6-2** for design wind pressures for this example. This building has a flat roof, so only one Load Case A or B is needed (see Fig. 28.6-1 of the Standard).

$$p_s = \lambda\, K_{zt}\, p_{s30} = 1.21 \times 1.0 \times p_{s30} \qquad \text{(Eq. 28.6-1)}$$

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

Design Pressures for Components and Cladding

According to Section 30.5 of the Standard,

$$p_{\text{net}} = \lambda\, K_{zt}\, p_{\text{net30}} = 1.21 \times 1.0 \times p_{\text{net30}} \qquad \text{(Eq. 30.5-1)}$$

Wall Pressures

The effective wind area for a CMU wall is 75 ft^2 (see **Section 6.1** of this guide). Linear interpolation is permitted in Fig. 30.5-1 of the Standard. Linear interpolation is required for wind speed of 148 mph and effective wind area of 75 ft^2. $K_{zt} = 1.0$ and $(\lambda) = 1.21$.

Zone 4

$$p_{\text{net}} = 1.21 \times 1.0 \times 34.4 = 41.6 \text{ psf}$$
$$p_{\text{net}} = 1.21 \times 1.0 \times (-37.7) = -45.6 \text{ psf}$$

Zone 5

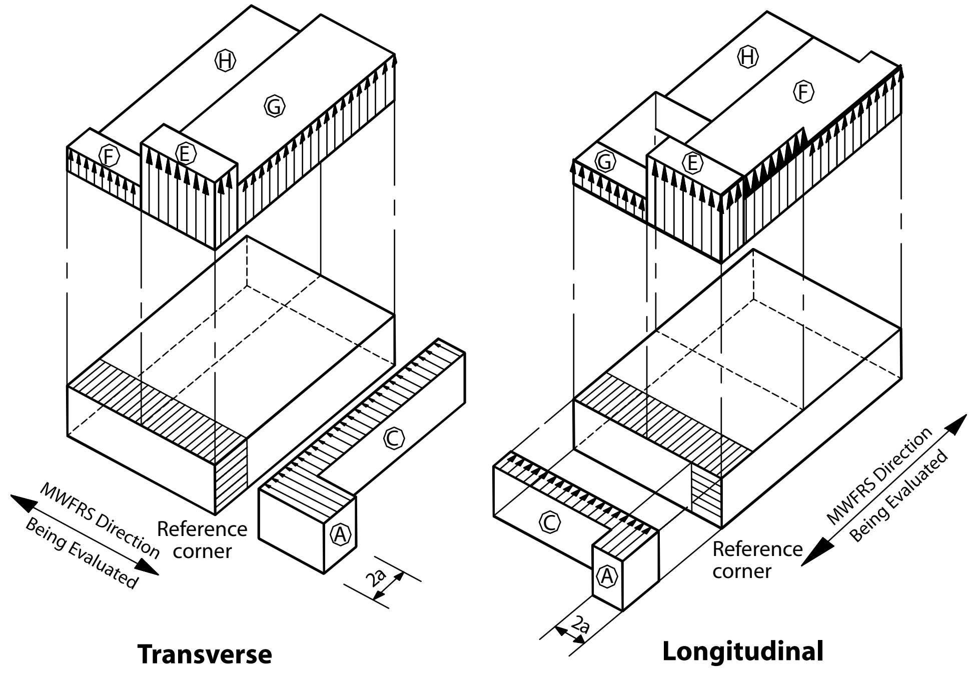
$$p_{\text{net}} = 1.21 \times 1.0 \times 34.4 = 41.6 \text{ psf}$$
$$p_{\text{net}} = 1.21 \times 1.0 \times (-42.8) = -51.8 \text{ psf}$$

Table G6-2 Design Wind Pressures, p_{s30}

	Zones					
Wind speed	*A*	*C*	*E*	*F*	*G*	*H*
140 mph	31.1	20.6	–37.3	–21.2	–26.0	–16.4
150 mph	35.7	23.7	–42.9	–24.4	–29.8	–18.9
148 mph	34.8	23.1	–41.8	–23.8	–29.0	–18.4
$p_s = 1.21 \times p_{s30}$ (psf)	42.1	28.0	–50.6	–28.8	–35.1	–22.3

Notes: Values of p_{s30} are obtained from Fig. 28.6-1 of the Standard. Zones are defined in Fig. 28.6-1 of the Standard: a = smaller of $0.1 \times 30 = 3$ ft (control) or $0.4 \times 15 = 6$ ft. The load patterns shown in **Fig. G6-5** shall be applied to each corner of the building in turn as the reference corner; see Note 2 of Fig. 28.6-1 of the Standard.

Fig. G6-5
Design wind pressure for transverse and longitudinal directions



Roof Joist Pressures

From Fig. 30.5-1 of the Standard, for $V = 148$ mph, for effective wind area of 300 ft^2, the design pressures are

Zone 1

$$p_{\text{net}} = 1.21 \times 1.0 \times 12.7 = 15.4 \text{ psf}$$
$$p_{\text{net}} = 1.21 \times 1.0 \times (-36.1) = -43.7 \text{ psf}$$

Zones 2 and 3

$$p_{\text{net}} = 1.21 \times 1.0 \times 12.7 = 15.4 \text{ psf}$$
$$p_{\text{net}} = 1.21 \times 1.0 \times 1.0 \times (-42.8) = -51.8 \text{ psf}$$

Roof Panel Pressures

Effective wind area for roof panel is 10 ft^2 (see Example 1 in Section 6.1). From Fig. 30.5-1 of the Standard, for $V = 148$ mph, for effective wind area of 10 ft^2, the design pressures are

Zone 1

$$p_{\text{net}} = 1.21 \times 1.0 \times 16.1 = 19.5 \text{ psf}$$
$$p_{\text{net}} = 1.21 \times 1.0 \times (-39.5) = -47.8 \text{ psf}$$

Zone 2

$$p_{\text{net}} = 1.21 \times 1.0 \times 16.1 = 19.5 \text{ psf}$$
$$p_{\text{net}} = 1.21 \times 1.0 \times (-66.2) = -80.1 \text{ psf}$$

Zone 3

$$p_{\text{net}} = 1.21 \times 1.0 \times 16.1 = 19.5 \text{ psf}$$
$$p_{\text{net}} = 1.21 \times 1.0 \times (-99.6) = -120.5 \text{ psf}$$

Comment

The pressures determined are limit state design pressures for strength design. Section 2.3 of the Standard indicates load factor for wind load to be $1.0W$ for loads determined in this example. If allowable stress design is to be used, the load factor for wind load is $0.6W$ as shown in Section 2.4 of the Standard.

Chapter 7

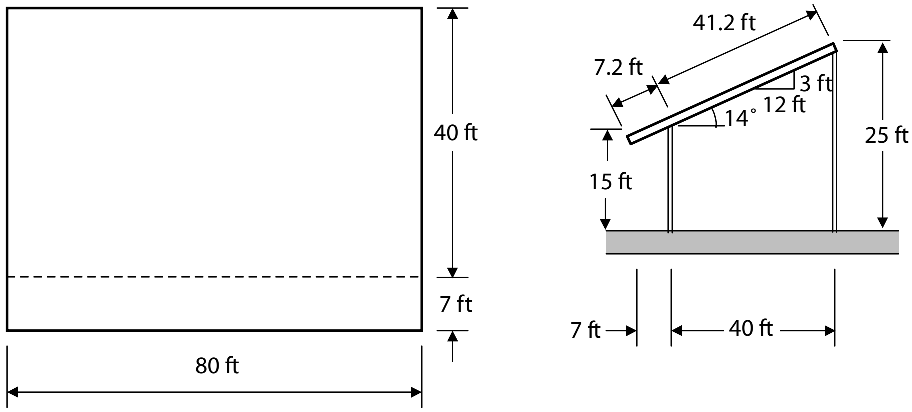
Commercial Building with Monoslope Roof and Overhang

In this example, design pressures for a typical retail store in a strip mall are determined. The building's dimensions are shown in **Fig. G7-1**. The building data are listed in **Table G7-1**.

7.1 Analytical Procedure

The building does not meet the requirements of chapter 27, part 2, of the Standard for a simple diaphragm building, because the roof is not flat, gable, or hip (see Fig. 27.6-2 of the Standard), so the simplified procedure cannot be used. If the roof is not gable, flat, or hip the low-rise building provisions of chapter 28, part 1, of the Standard should not be used. Therefore, chapter 27, part 1, of the Analytical Procedure, is used (see Section 27.2 of the Standard) for MWFRS. For components and cladding, chapter 30, part 1, for low-rise buildings is used.

Fig. G7-1
Building characteristics for commercial building with monoslope roof and overhang



Building Classification, Enclosure Classification, and Exposure Category

The building is not an essential facility, nor is it likely to be occupied by more than 300 persons at any one time. Use Category II (see Table 1.5-1 of the Standard). Wind speed map in Fig. 26.5-1A is appropriate for this category of building.

The building is sited in a suburban area and satisfies the criteria for Exposure B (see Section 26.7 of the Standard).

Since the building is sited within 1 mi of the coastal water, it is considered in a wind-borne debris region. It has glazing (that must be impact resistant) occupying 50 percent of a wall that receives positive pressure. The building could be classified as enclosed (see Sections 26.10.3 and 26.2 of the Standard). The standard does not require that the building be classified as partially enclosed if it is located in a wind-borne debris region; it just requires that the openings be protected with impact-resistant glazing or shutters. The wind-borne debris region is defined in Section 26.2.

Table G7-1 Data for Commercial Building with Monoslope Roof and Overhang

Location	Boston, Massachusetts, within 1 mi of the coastal mean high watermark
Topography	Homogeneous
Terrain	Suburban
Dimensions	40 ft × 80 ft in plan Monoslope roof with slope of 14° and overhang of 7 ft in plan Wall heights are 15 ft in front and 25 ft in rear $h = 20$ ft
Framing	Walls of CMU on all sides supported at top and bottom; steel framing in front (80-ft width) to support window glass and doors Roof joists span 41.2 ft with 7.2-ft overhang spaced at 5 ft on center
Cladding	Glass and door sizes vary; glazing is debris impact-resistant and occupies 50 percent of front wall (80 ft in width) Roof panels are 2 ft wide and 20 ft long

Basic Wind Speed

The wind speed contours of 130 and 140 mph traverse either side of Boston, Massachusetts (Fig. 26.5-1A of the Standard); use a basic wind speed of 135 mph (interpolating between the 130 and 140 mph isotachs).

Velocity Pressures

The velocity pressures for MWFRS (**Table G7-2**) are calculated using the following equation (see Section 27.3.2 of the Standard):

$$\begin{aligned} q &= 0.00256\ K_z K_{zt} K_d V^2 \text{ (psf)} \qquad \text{(Eq. 27.3-1)} \\ &= 0.00256\ K_z(1.0)(0.85)(135)^2 \\ &= 39.66\ K_z \text{ (psf)} \end{aligned}$$

where

K_z = value obtained from Table 27.3-1 of the Standard for MWFRS and Table 30.3-1 of the Standard for C&C

K_{zt} = 1.0 homogeneous terrain

K_d = 0.85, see Table 26.6-1 of the Standard

Since the Exposure Category is B for the building location, values of K_z are different for MWFRS and C&C.

Design Pressures for the MWFRS

The equation for rigid buildings of all heights is given in Section 27.4.1 of the Standard as follows:

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

where

q = q_z for windward wall

q_i = q_h for windward and leeward walls, side walls and roof for enclosed buildings

G = value determined from Section 26.9 of the Standard

C_p = value obtained from Figure 27.4-1 of the Standard

(GC_{pi}) = value obtained from Table 26.11-1 of the Standard

For positive internal pressure evaluation, the Standard permits q_i to be conservatively evaluated at height $h(q_i = q_h)$.

Table G7-2 Velocity Pressures

	MWFRS		*C&C*	
	Exposure B		*Exposure B*	
Height (ft)	K_z	q_z *(psf)*	K_z	q_h *(psf)*
0 to 15	0.57	22.6		
h = 20	0.62	24.6*	0.70	27.8
25	0.66	26.2		

Note: *q_h = 24.6 psf for MWFRS.

Gust Effect Factor

The building can be classified as a low-rise building, so the Standard permits it to be a rigid building without determining fundamental frequency (Section 26.9.2)

The gust effect factor for nonflexible (rigid) buildings is given in Section 26.9.1 of the Standard as $G = 0.85$.

Wall External Pressure Coefficients

The coefficients for the windward and side walls in **Table G7-3** are given in Fig. 27.4-1 of the Standard as $C_p = +0.8$ and -0.7, respectively. The values for the leeward wall depend on L/B; they are different for the two directions: wind parallel to roof slope (normal to ridge), and wind normal to roof slope (parallel to ridge).

Roof External Pressure Coefficients

Since the building has a monoslope roof, the roof surface for wind directed parallel to the slope (normal to ridge) may be a windward or a leeward surface. The value of $h/L = 0.5$ in this case, and the proper coefficients are obtained from linear interpolation for $\theta = 14°$ (see **Table G7-4**). When wind is normal to the roof slope (parallel to ridge), angle $\theta = 0$ and $h/L = 0.25$.

For the overhang, Section 27.4.4 of the Standard requires $C_p = 0.8$ for wind directed normal to a 15-ft wall. The Standard does not address the leeward overhang for the case of wind directed toward a 25-ft wall and perpendicular to roof slope (parallel to ridge). A $C_p = -0.5$ could be used (coefficient for leeward wall), but the coefficient has been conservatively taken as 0.

The building is sited in a hurricane prone region less than 1 mi from the coastal mean high-water level. The basic wind speed is 135 mph, and the glazing must be designed to resist wind-borne debris impact (or some other method of protecting the glazing must be installed, such as shutters). Thus, as noted earlier, the building is classified as enclosed for this example. The internal pressure coefficients, from Table 26.11-1 of the Standard, are as follows:

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

Typical Calculations of Design Pressures for MWFRS

For cases with wind parallel to the slope with a 15-ft windward wall (see **Table G7-5**).

Table G7-3 Wall Pressure Coefficients

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

Table G7-4 Roof Pressure Coefficients

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

*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.

Table G7-5 Design Pressures for MWFRS for Wind Parallel to Roof Slope, Normal to Ridge Line

Wind direction	*Surface*	z *(ft)*	q_z *(psf)*	*Gust effect*	*External* C_p	*Design pressure (psf)** (+GC_{pi})	(–GC_{pi})
Windward wall (15 ft)	Windward wall	0 to 15	22.6	0.85	0.80	10.9	19.8
	Leeward wall	0 to 25	24.6	0.85	–0.50	–14.9	–6.0
	Side wall	All	24.6	0.85	–0.70	–19.1	–10.2
	Roof	—	24.6	0.85	–0.74	–19.9	–11.0
	Overhang top	—	24.6	0.85	–0.74	–15.5**	–15.5**
	Overhang bottom	—	22.6	0.85	0.80	15.4**	15.4**
Windward wall (25 ft)	Windward wall	0 to 15	22.6	0.85	0.80	10.9	19.8
		15 to 20	24.6	0.85	0.80	16.7	21.2
		20 to 25	26.2	0.85	0.80	17.8	22.2
	Leeward wall	All	24.6	0.85	–0.50	–14.9	–6.0
	Side wall	All	24.6	0.85	–0.70	–19.1	–10.2
	Roof	—	24.6	0.85	–0.50	–14.9	–6.0
	Overhang top	—	24.6	0.85	–0.50	–10.4**	–10.4**
	Overhang bottom	—	—		—	0.0**	0.0**

*External pressure calculations include $G = 0.85$.

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

Pressure on Leeward Wall

$$p = q_h GC_p - q_h(\pm GC_{pi})$$
$$= 24.6(0.85)(-0.5) - (24.6)(+0.18) = -14.9 \text{ psf with positive internal pressure}$$
$$\text{and } = 24.6(0.85)(-0.5) - (24.6)(-0.18) = -6.0 \text{ psf with negative internal pressure}$$

Pressure on Overhang Top Surface

$$p = q_h GC_p = 24.6(0.85)(-0.74) = -15.5 \text{ psf}$$

Pressure on Overhang Bottom Surface

This is the same as windward wall external pressure.

$$p = q_z GC_p = 22.6(0.85)(0.8) = 15.4 \text{ psf}$$

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

Figures G7-2 and G7-3 illustrate the external, internal, and combined pressure for wind directed normal to the 15-ft wall. **Figures G7-4 and G7-5** illustrate combined pressure for wind directed normal to the 25-ft wall and perpendicular to slope (parallel to ridge line), respectively (see **Table G7-6**).

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 chapter 27 shall be designed for wind load cases as defined in Fig. 27.4-8 of the Standard. Case 1 includes the loadings shown in **Fig. G7-2 through Fig. G7-5**. The exception in Section 27.4.6 of the Standard indicates that buildings meeting the requirements of Section D1.1 of Appendix D need only be designed for Case 1 and Case 3 of Fig. 27.4-8.

Design Pressures for Components and Cladding

The design pressure equation for components and cladding (C&C) for a building with mean roof height $h \leq 60$ ft is given in Section 30.4.2 of the Standard:

$$P = q_h[(GC_p) - (GC_{pi})] \qquad \text{(Eq. 30.4-1)}$$

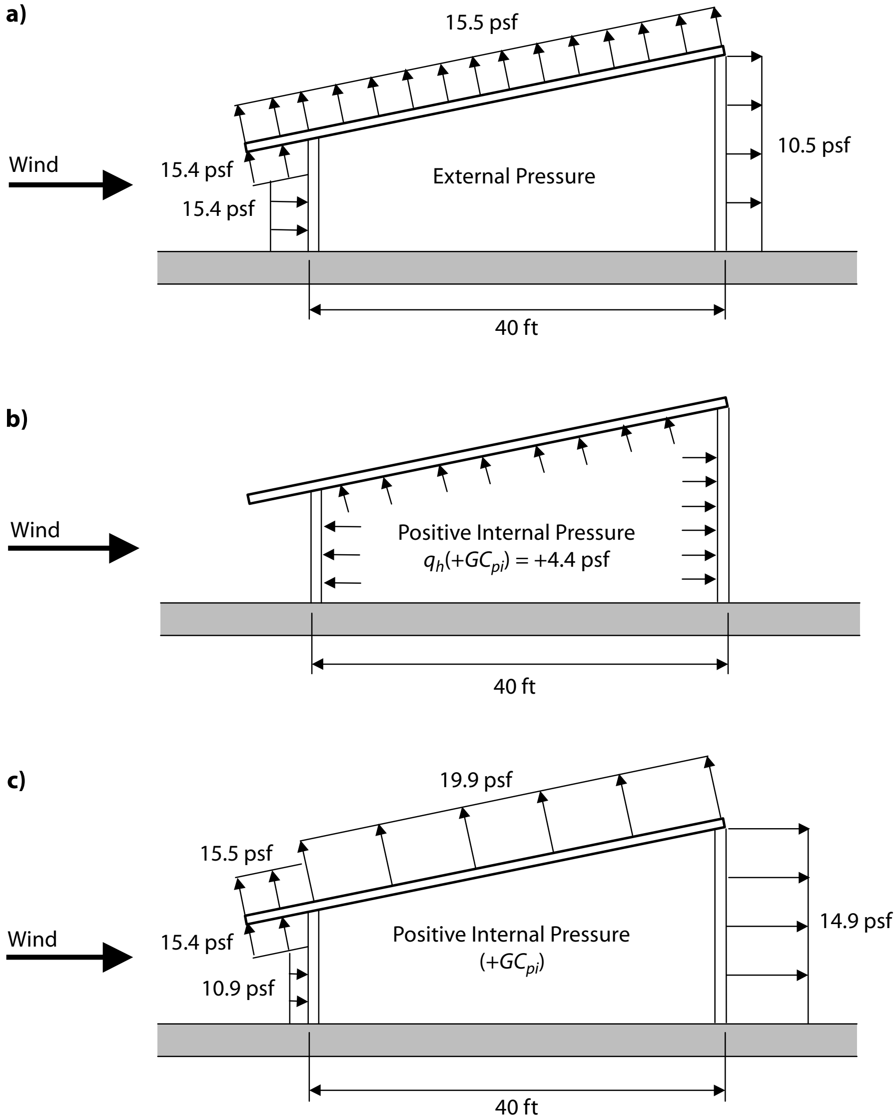
where

q_h = velocity pressure at mean roof height associated with Exposure B ($q_h = 27.8$ psf, previously determined)

(GC_p) = external pressure coefficients from Figs. 30.4-2A, 30.4-2B, and 30.4-2C of the Standard

(GC_{pi}) = +0.18 and −0.18, previously determined from Table 26.11-1 of the Standard

Fig. G7-2
Design pressures for MWFRS for wind parallel to roof slope, normal to 15-ft wall, and positive internal pressure: a) external pressures, b) positive internal pressure, and c) combined external and positive internal pressure



Wall Design Pressures

Wall external pressure coefficients are presented in **Table G7-7**. Since the CMU walls are supported at the top and bottom, the effective wind area will depend on the span length.

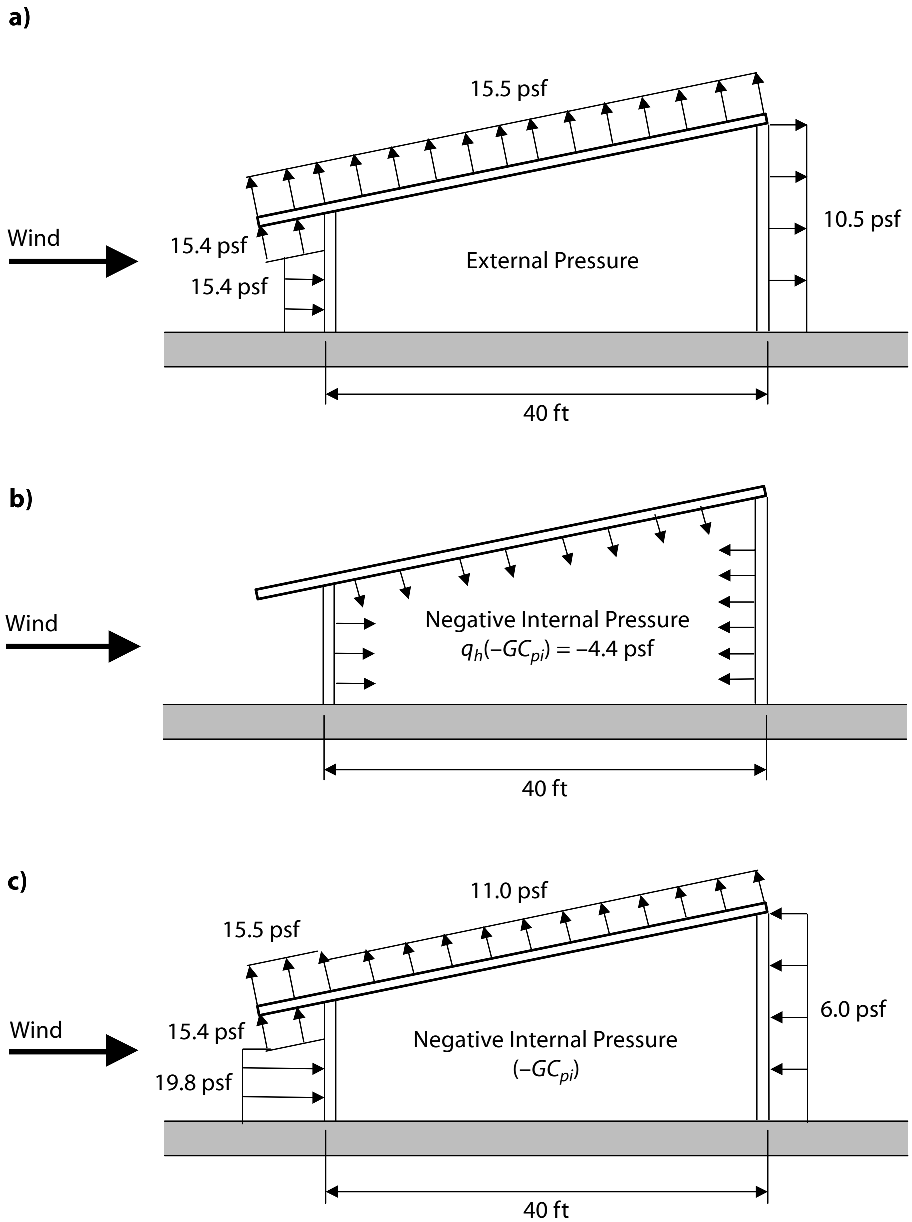
Effective wind area

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

Width of Zone 5 (Fig. 30.4-1)

smaller of	$a = 0.1(40) = 4$ ft (controls)
or	$a = 0.4(20) = 8$ ft
but not less than	$a = 0.4(40) = 1.6$ ft
or	$a = 3$ ft

Fig. G7-3
Design pressures for MWFRS for wind parallel to roof slope, normal to 15-ft wall, and negative internal pressure: a) external pressures, b) negative internal pressure, and c) combined external and negative internal pressure



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

Typical Calculations for Design Pressures for 15-ft Wall, Zone 4

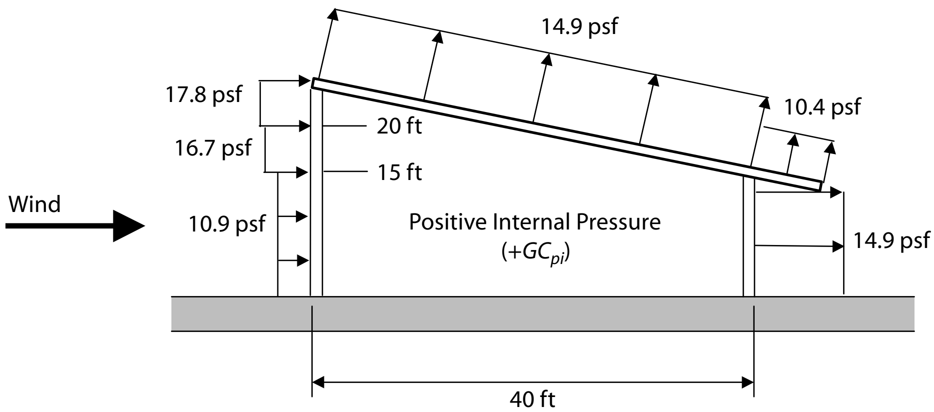
Wall design pressures are presented in **Table G7-8**.

$$
\begin{aligned}
p &= q_h[(GC_p) - (\pm GC_{pi})] \\
&= 27.8[(0.85) - (-0.18)] = 28.6 \text{ psf} \\
\text{and } &= 27.8[(-0.95) - (0.18)] = -31.4 \text{ psf}
\end{aligned}
$$

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

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

Fig. G7-4
Combined design pressures for MWFRS for wind parallel to roof slope, normal to 25-ft wall



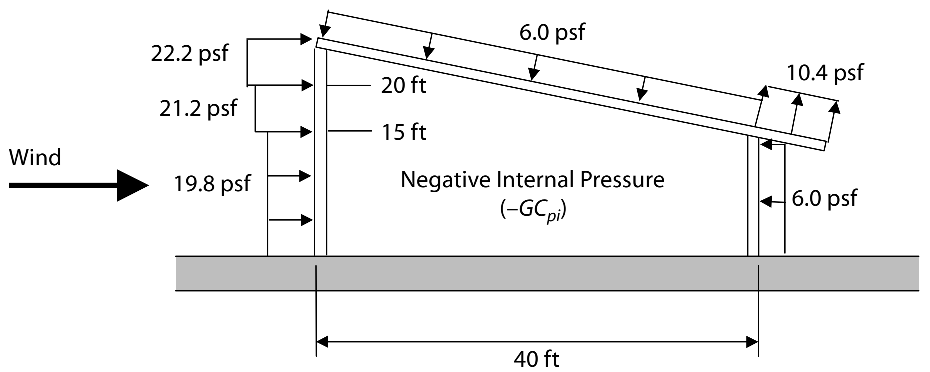
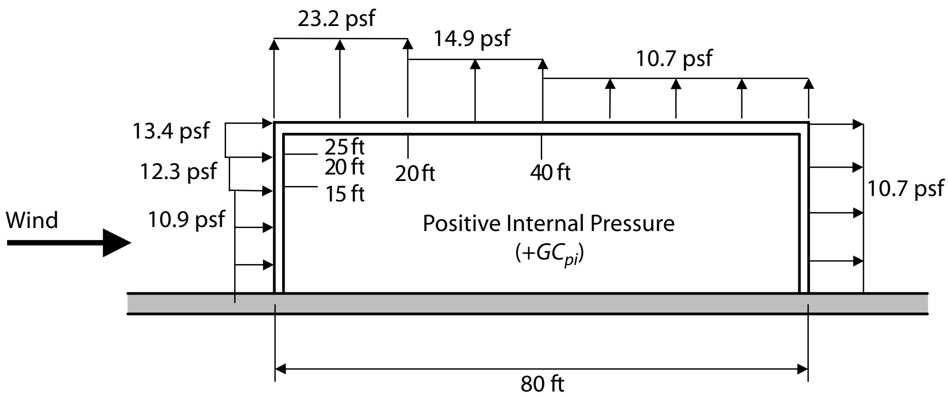


Fig. G7-5
Combined design pressures for MWFRS for wind perpendicular to roof slope, parallel to ridge line



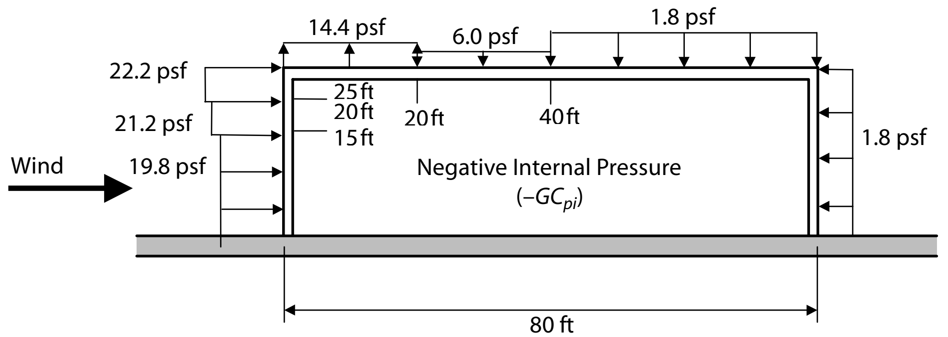


Table G7-6 Design Pressures for MWFRS for Wind Normal to Roof Slope, Parallel to Ridge Line

Surface	*z or distance (ft)*	q_z *(psf)*	*Gust effect, G*	C_p	*Design pressure (psf)* $+(GC_{pi})$	$-(GC_{pi})$
Windward wall	0–15	22.6	0.85	0.8	10.9	19.8
	15–20	24.6	0.85	0.8	12.3	21.2
	20–25	26.2	0.85	0.8	13.4	22.2
Leeward wall	All	24.6	0.85	–0.3	–10.7	–1.8
Side wall	All	24.6	0.85	–0.7	–19.1	–10.2
Roof	0–20	24.6	0.85	–0.9	–23.2	–14.4
	20–40	24.6	0.85	–0.5	–14.9	–6.0
	40–80	24.6	0.85	–0.3	–10.7	–1.8

Note: For design pressure, external pressure calculations include $G = 0.85$. Internal pressure, (GC_{pi}), is associated with $q_h = 24.6$ psf. For z, distance along roof is from leading windward edge. For the roof, pressure on the overhang is only external pressure (contribution on underside is conservatively neglected).

Table G7-7 Wall External Pressure Coefficients by Zone

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

Table G7-8 Wall Design Pressures by Zone

Wall height (ft)	*Design pressures (psf)* Zones 4 and 5 Positive	Zone 4 Negative	Zone 5 Negative
15	28.6	–31.4	–35.3
20	27.2	–30.0	–32.8
25	26.4	–29.2	–30.9

Note: $q_h = 27.8$ psf.

Roof Design Pressures

Roof external pressure coefficients are presented in **Table G7-9**; roof design pressures are presented in **Table G7-10**.

Effective wind area

Roof joist

$$A = (41.2)(5) = 206 \text{ ft}^2$$
$$\text{or} = (41.2)(41.2/3) = 566 \text{ ft}^2 \text{ (controls)}$$

Table G7-9 Roof External Pressure Coefficients for $\theta = 14°$, by Zone

Component	A *(ft²)*	*Zones 1, 2, and 3 (+GC$_p$)*	*Zone 1 (–GC$_p$)*	*Zone 2 (–GC$_p$)*	*Zone 3 (–GC$_p$)*
		Pressure coefficient, Fig. 30.4-5B			
Joist	566	0.3	–1.1	–1.2	–2.0
Panel	10	0.4	–1.3	–1.6	–2.9
		Pressure coefficient, Fig. 30.4-2B			
Joist	566	0.3	–0.8	–2.2*	–2.5*
Panel	10	0.5	–0.9	–2.2*	–3.7*

*Values are from overhang chart in Fig. 30.4-2B.

Roof panel

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

Section 30.10 of the Standard requires that pressure coefficients for components and cladding of roof overhangs be obtained from Fig. 30.4-2A to 30.4-2C. Note that the zones for roof overhangs in Fig. 30.4-2B are different from the zones for a monoslope roof in Fig. 30.4-5B.

Width of zone distance

smaller of $a = 0.1(40) = 4$ ft (controls)
or $a = 0.4(20) = 8$ ft
but not less than $a = 0.4(40) = 1.6$ ft
or $a = 3$ ft

The widths and lengths of Zones 2 and 3 for a monoslope roof are shown in Fig. 30.4-5B of the Standard (they vary from a to $4a$); for overhangs, widths and lengths are shown in Fig. 30.4-2B.

Similar to the determination of design pressures for walls, the critical design pressures for roofs are the algebraic sum of the external and internal

Table G7-10 Roof Design Pressures by Zone (psf)

Component	*Zones 1, 2, and 3 Positive*	*Zone 1 Negative*	*Zone 2 Negative*	*Zone 3 Negative*
Joist	13.3	–35.6	–38.4	–60.6
Joist overhang	13.3	–27.2	–66.2	–74.5
Panel	16.1	–41.1	–49.5	–85.6
Panel in overhang	18.9	–30.0	–66.2	–107.9

Notes: $q_h = 27.8$ psf. Zones for overhang are in accordance with Fig. 30.4-2B of the Standard. Section 30.2.2 of the Standard requires a minimum of 16 psf.

pressures. The design pressures for overhang areas are based on pressure coefficients obtained from Fig. 30.4-2B of the Standard.

Typical Calculations for Joist Pressures

Zone 2

$$p = q_h[(GC_p) - (\pm GC_{pi})]$$
$$= 27.8[(0.3) - (-0.18)] = 13.3 \text{ psf}$$
$$\text{and } = 27.8[(-1.2) - (0.18)] = -38.4 \text{ psf}$$

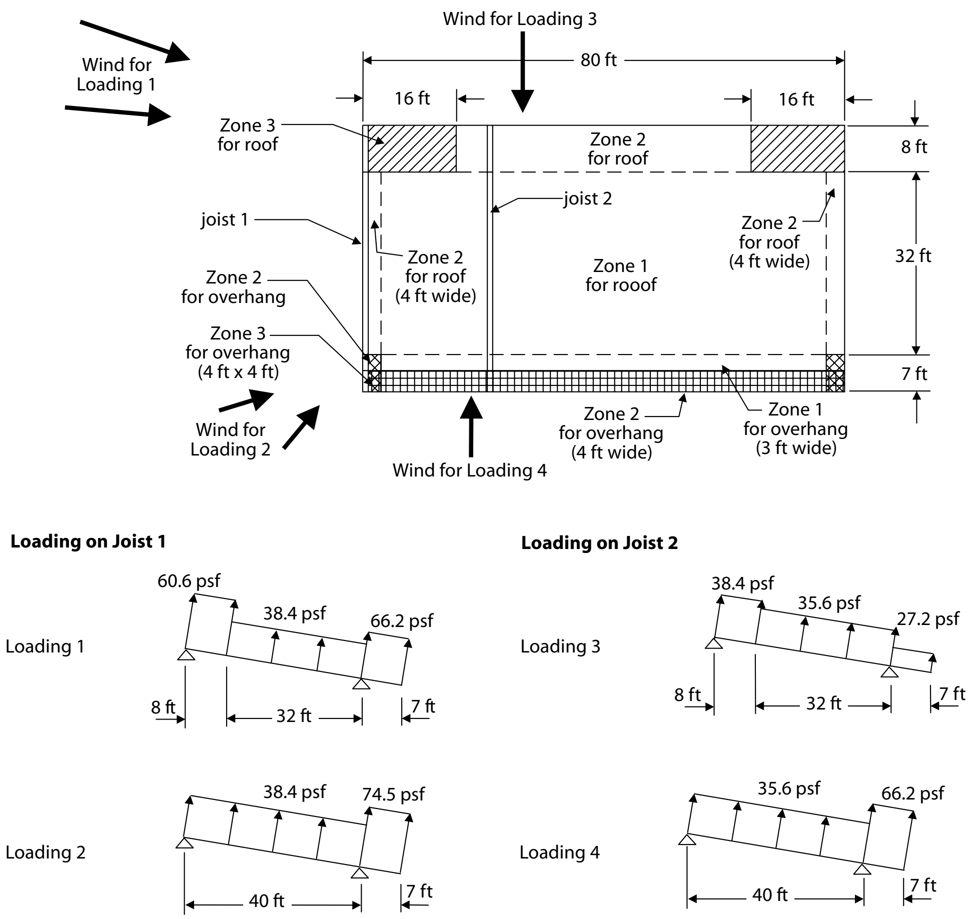
Zones for the monoslope roof and for overhangs are shown in **Fig. G7-6.** The panels are designed for the pressures indicated.

Roof joist design pressures need careful interpretation. The high pressures in corner or eave areas do not occur simultaneously at both ends. Two loading cases: wind loadings 1, 2 for joist 1 and wind loadings 3, 4 for joist 2, are shown in **Fig. G7-6** based on the following zones:

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

For simplicity, only one zone is used for overhang pressures in **Fig. G7-6**.

Fig. G7-6
Design pressures for typical joists and pressure zones for roof components and cladding



Comment

The pressures determined are limit state design pressures for strength design. Section 2.3 of the Standard indicates load factor for wind load to be $1.0W$ for loads determined in this example. If allowable stress design is to be used, the load factor for wind load is $0.6W$ as shown in Section 2.4 of the Standard.

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Chapter 8

L-Shaped 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 **Fig. G8-1**. The physical data are presented in **Table G8-1**.

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

8.1 Analytical Procedure

For MWFRS analytical directional procedure for building of any height given in chapter 27, part 1, is used to determine design wind pressure.

For C&C, analytical envelope procedure for low-rise (h < 60 ft) building given in chapter 30, part 1, is used to determine design wind pressure.

Building Classification

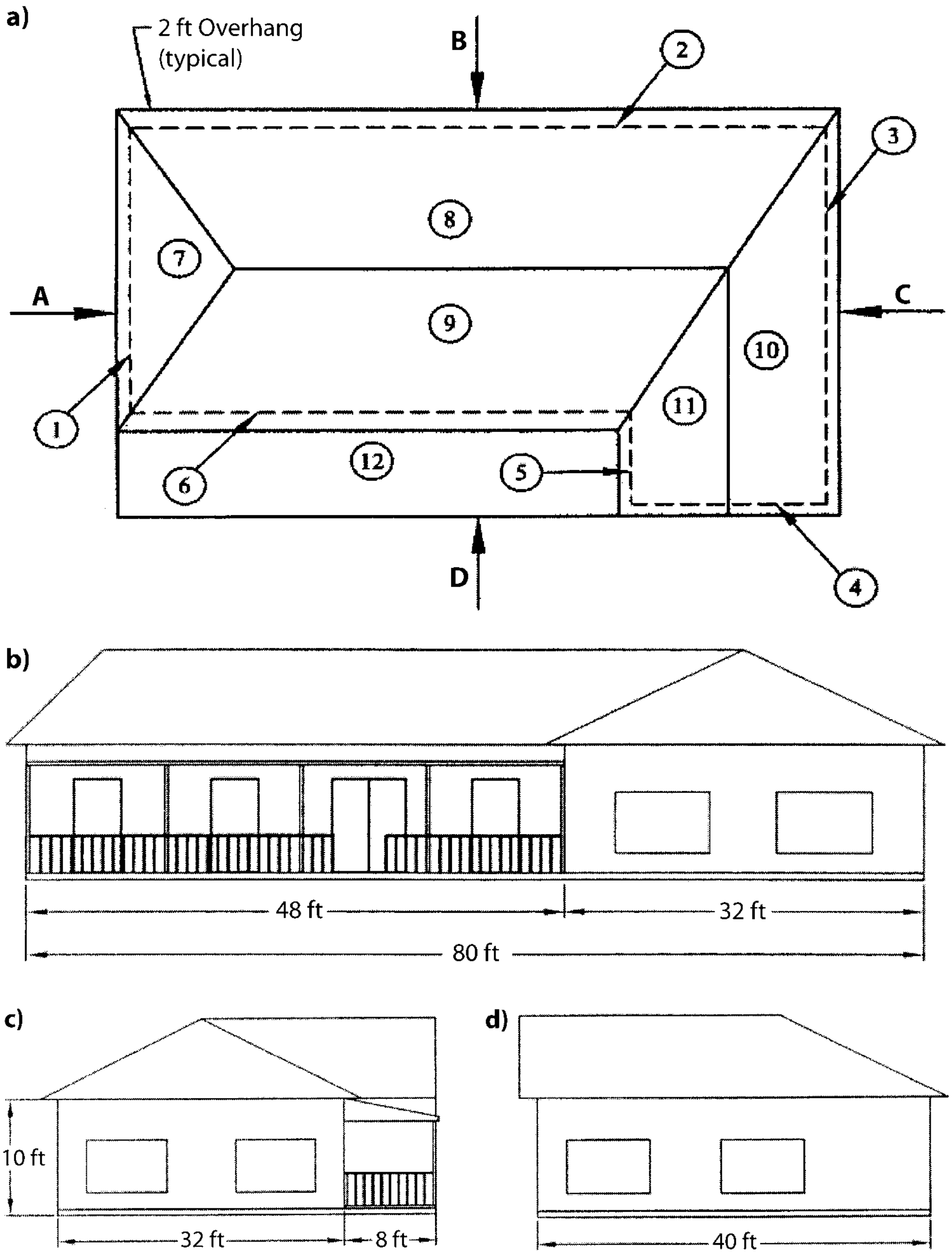
Residential building can be in Risk Category II according to Table 1.5-1 of the Standard. Wind speed map associated with this risk category is Fig. 26.5-1A of the Standard.

Wind Load Parameters

Wind speed V = 115 mph (Fig. 26.5-1A of the Standard)
Topography factor K_{zt} = 1.0 (Section 26.8 of the Standard)
Directionality factor K_d = 0.85 (for buildings) (Table 26.6-1 of the Standard)

The building is located in a suburban area; according to Section 26.7.2 and 26.7.3 of the Standard, Exposure B is used.

Fig. G8-1
Building characteristics for house with gable/hip roof: a) view of roof, b) view of front, c) view of side A, and d) view of side C



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

Since K_z is constant in the 0 to 15 ft region:

$K_z = K_h = 0.70$ for C&C (from Table 30.3-1)
$K_z = K_h = 0.57$ for MWFRS (from Table 27.3-1)

Velocity Pressures

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

For MWFRS, $q_z = q_h = 0.00256(0.57)(1.0)(0.85)(115)^2 = 16.4$ psf
For C&C, $q_z = q_h = 0.00256(0.7)(1.0)(0.85)(115)^2 = 20.1$ psf

Table G8-1 Data for L-Shaped House with Gabled/Hip Roof

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 θ = 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 2 ft on center Roof panels are 4 ft × 8 ft

Gust Effect Factor

This building meets the requirements of a low-rise building (Section 26.2 of the Standard); hence, it is permitted to be considered a rigid building (section 26.9.2)

$$G = 0.85 \qquad \text{(Section 26.9)}$$

Internal Pressures

Enclosure classification is a matter of judgment. Glazing is uniformly distributed. If it is designed to resist the pressures, it may be classified as an enclosed building.

$$(GC_{pi}) = +0.18 \text{ and } -0.18 \qquad \text{(Table 26.11-1)}$$

Design Pressure for the 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. The external pressure coefficients are from Fig. 27.4-1 of the Standard.

Wind Direction A

See **Fig. G8-2.**

$$h/L = 12.1/80 = 0.15 < 0.5$$

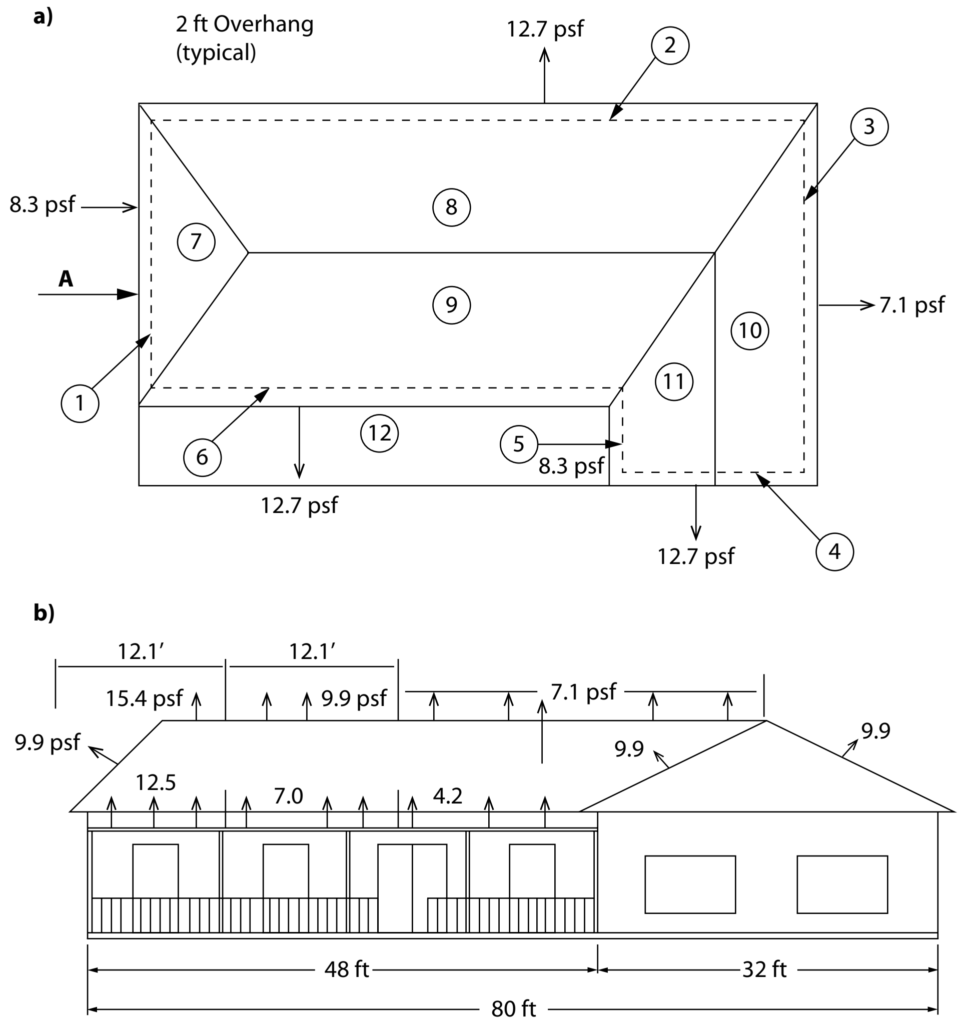
Wall pressures

Surface 1: $p = 16.4(0.85)(0.8) - 16.4(\pm 0.18) = +11.2 \pm 2.9$ psf (windward)

Surface 2: $p = 16.4(0.85)(-0.7) - 16.4(\pm 0.18) = -9.8 \pm 2.9$ psf (side)

Surface 3: $p = 16.4(0.85)(-0.3) - 16.4(\pm 0.18) = -4.2 \pm 2.9$ psf (leeward) (for $L/B = 80/40 = 2$; $C_p = -0.3$)

Fig. G8-2
MWFRS net pressures for positive internal pressure from Direction A



Surface 4: $p = -9.8 \pm 2.9$ psf (side)
Surface 5: $p = +11.2 \pm 2.9$ psf (windward)
Surface 6: $p = -9.8 \pm 2.9$ psf (side)

Roof pressures coefficient

Roof pressure coefficient, C_p, from Fig. 27.4-1 of the Standard is shown in **Table G8-2**.

Roof pressures calculation

Surface 7: $p = 16.4(0.85)(-0.5) - 16.4(\pm 0.18) = -7.0 \pm 2.9$ psf (windward)

Surface 8: for $\theta = 0°$; pressure varies along the roof
$p = 16.4(0.85)(-0.9) - 16.4(\pm 0.18) = -12.5 \pm 2.9$ psf; 0 to 6.1 ft
$p = 16.4(0.85)(-0.9) - 16.4(\pm 0.18) = -12.5 \pm 2.9$ psf; 6.1 to 12.1 ft
$p = 16.4(0.85)(-0.5) - 16.4(\pm 0.18) = -7.0 \pm 2.9$ psf; 12.1 to 24.2 ft

Table G8-2 Roof Pressure Coefficient for Wind Direction A ($h/L = 0.15$)

	Surface								
	7	8				9	10	11	12
		Horizontal distance from windward edge (ft)							
		0 to 6.1	6.1 to 12.1	12.1 to 24.2	24.2 to end				
C_p	0.0* −0.5	−0.9 −0.18*	−0.9 −0.18*	−0.5 −0.18*	−0.3 −0.18*	Same as surface 8	−0.5 −0.5*	−0.5 0.0*	Same as surface 8

*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 combination are given in Sections 2.3 and 2.4 of the Standard. For brevity, loading for this value is not shown here.

$p = 16.4(0.85)(-0.3) - 16.4(\pm 0.18) = -4.2 \pm 2.9$ psf; 24.2 ft to end

Surface 9: Same pressures as surface 8

Surface 10: $p = 16.4(0.85)(-0.5) - 16.4(\pm 0.18) = -7.0 \pm 2.9$ psf (leeward)

Surface 11: $p = 16.4(0.85)(-0.5) - 16.4(\pm 0.18) = -7.0 \pm 2.9$ psf (windward)

Surface 12: Same as surface 8 without internal pressure

Overhang pressures

From Section 27.4.4 of the Standard, at wall surfaces 1 and 5:

$$p = 16.4(0.85)(0.8) = +11.2 \text{ psf}$$

This pressure is to be combined with the top surface pressure. Internal pressure is of the same sign on all applicable surfaces.

Wind Direction B

Wall pressures

Surface 1: $p = -9.8 \pm 2.9$ psf (side)

Surface 2: $p = +11.2 \pm 2.9$ psf (windward)

Surface 3: $p = -9.8 \pm 2.9$ psf (side)

Surface 4: $p = 16.4(0.85)(-0.5) - 16.4(\pm 0.18) = -7.0 \pm 2.9$ psf (leeward) (for $L/B = 40/80 = 0.5$; $C_p = -0.5$)

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

Surface 6: Same pressure as surface 4

Roof pressures coefficient

$$h/L = 12.1/40 = 0.3;\ \theta = 15°$$

Roof pressure coefficient, C_p, from Fig. 27.4-1 of the Standard is shown in **Table G8-3**.

Table G8-3 Roof Pressure Coefficient for Wind Direction B ($h/L = 0.3$)

	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

Note: Bold type indicates value gained by interpolation.

For windward: $C_p = -0.54$ (interpolated between –0.5 and –0.7)
For leeward: $C_p = -0.5$
For parallel to ridge: $C_p = -0.9$(0 to 12.1 ft)
$C_p = -0.5$(12.1 to 24.2 ft)
$C_p = -0.3$(24.2 ft to end)

Roof pressures calculation

Surface 7: Same pressures as surface 8 for Wind Direction A
Surface 8: $p = 16.4(0.85)(-0.54) - 16.4(\pm 0.18) = -7.5 \pm 2.9$ psf (windward)
Surface 9: $p = 16.4(0.85)(-0.5) - 16.4(\pm 0.18) = -7.0 \pm 2.9$ 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 $2h$
$p = 16.4(0.85)(-0.3) = -4.2$ psf; no internal pressure

Overhang pressures

From Section 27.4.4 of the Standard, at wall surface 2:

$$p = 16.4(0.85)(0.8) = +11.2 \text{ psf}$$

This pressure is to be combined with the top surface pressure. Internal pressure is of the same sign on all applicable surfaces.

Wind Direction C

Wall pressures

Surfaces 1 and 5: $p = -7.0 \pm 2.9$ psf (leeward)
Surfaces 2, 4, and 6: $p = -9.8 \pm 2.9$ psf (side)
Surface 3: $p = +11.2 \pm 2.9$ psf (windward)

Roof pressures

Surfaces 7 and 11: $p = -7.0 \pm 2.9$ psf (leeward)
Surfaces 8 and 9: Pressures vary along the roof; same pressures as surface 8 for Wind Direction A
Surface 10: $p = -7.0 \pm 2.9$ psf (windward)

Surface 12: Same pressures as surface 9 without internal pressures

Overhang pressures

From Section 27.4.4 of the Standard, at wall surface 3:

$$p = 16.4\ (0.85)\ (0.8) = +11.2 \text{ psf}$$

This pressure is to be combined with the top surface pressure. Internal pressure is of the same sign on all applicable surfaces.

Wind Direction D

Wall pressures

Surfaces 1 and 3: $p = -9.8 \pm 2.9$ psf (side)
Surface 2: $p = -7.0 \pm 2.9$ psf (leeward)
Surfaces 4, 5, and 6: $p = +11.2 \pm 2.9$ 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: $p = -7.0 \pm 2.9$ psf (leeward)
Surface 9: $p = -7.0 \pm 2.9$ psf (windward)
Surface 12: This surface will see pressures on top and bottom surfaces; they will add algebraically.
For $\theta = 0°$, $h/L < 0.5$, $C_p = -0.9$
$p = 16.4(0.85)(-0.9) - 16.4(0.85)(+0.8) = -23.7$ psf uplift

Overhang pressures

From Section 27.4.4 of the Standard, at wall surfaces 4, 5, and 6

$$p = 16.4(0.85)(0.8) = +11.2 \text{ psf}$$

This pressure is to be combined with the top surface pressure. Internal pressure is of the same sign on all applicable surfaces.

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 chapter 27 shall be designed for wind load cases as defined in Fig. 27.4-8. Case 1 includes the loadings analyzed as noted. 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 the exception specified in Section 27.4.6 of the Standard. Only Load Cases 1 and 3 shown in Fig. 27.4-8 of the Standard need 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 Fig. 27.4-8 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 **Fig. G8-3**.
2. Because of the low roof slope, the wind load acting on the roof is negligible here.

Design Pressures for Components and Cladding

Design pressures, C_p, are in chapter 30, part 1 of the Standard.

Wall Component

Wall studs are 10 ft long and spaced 16 in. apart. The effective area is

larger of	$10 \times 1.33 = 13.3\ \text{ft}^2$
or	$10 \times 10/3 = 33.3\ \text{ft}^2$ (controls)

From Fig. 30.4-1 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

The distance a is

smaller of	$0.1(40) = 4$ ft (controls)
or	$0.4(12.1) = 4.8$ ft

Design pressure:

$p = 20.1(0.91 + 0.18) = +21.9$ psf (all walls)
$p = 20.1(-1.01 - 0.18) = -23.9$ psf (middle)
$p = 20.1(-1.22 - 0.18) = -28.1$ psf (corner)

Fig. G8-3
Combinations of wind loads (Arrows show the wind direction)

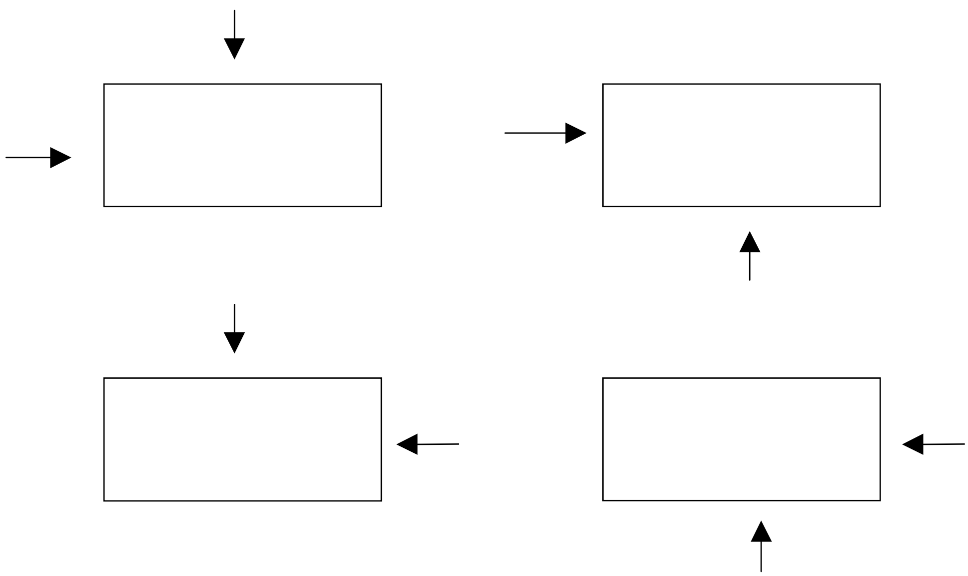
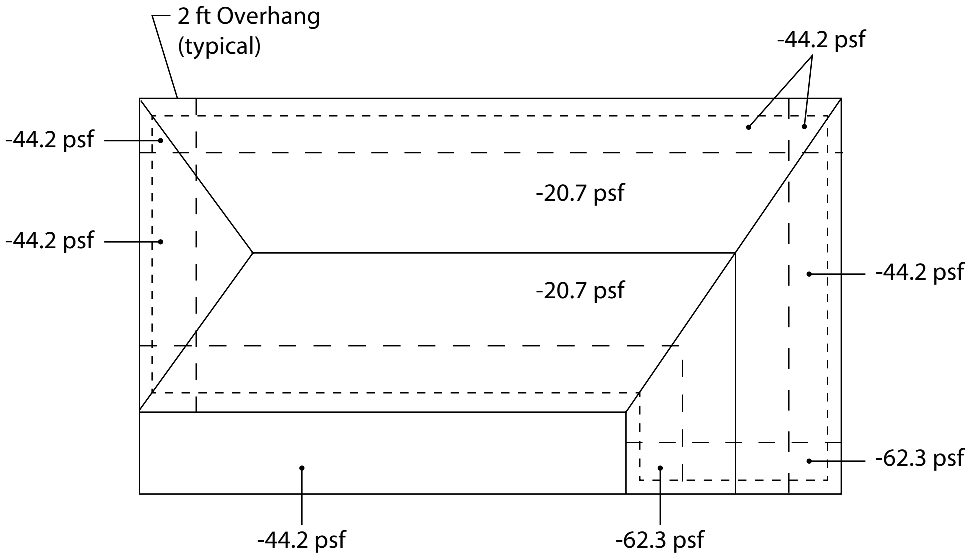


Fig. G8-4
Roof pressures for components and cladding



Roof Component

The distance a is

smaller of $0.1(40) = 4$ ft (controls)
or $0.4(12.1) = 4.8$ ft

Roof trusses are 32 ft long and spaced 2 ft apart. The effective area is

larger of $32 \times 2 = 64 \text{ ft}^2$
or $32 \times 32/3 = 341 \text{ ft}^2$ (controls)

From Fig. 30.4-2B of the Standard, for $\theta = 15°$

(GC_p) = +0.3 for Zones 1, 2, and 3 (Note: Zone 3 covers a very small area of truss)
(GC_p) = −0.8 for Zone 1
(GC_p) = −2.2 for Zone 2 (includes overhang area)

Design pressure (see Fig. G8-4)

$p = 20.1(0.3 + 0.18) = +9.6$ psf (all zones)
$p = 20.1(-0.8 - 0.18) = -19.7$ psf (Zone 1)
$p = 20.1 \times (-2.2\ ) = -44.2$ psf (Zone 2)

Overhang pressures

The overhang pressure to be used for reactions and anchorage is

$p = 20.1(-2.2) = -44.2$ psf (edge of roof)
$p = 20.1(-2.5) = -50.2$ psf (roof corners)

(MWFRS pressures should be used for the truss connection loads across the building span; C&C loads are used for the additional pressure created by the presence of the overhang.)

Roof Panels

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

From Fig. 30.4-2B of the Standard, for $\theta = 15°$ (Note: Zones 2 and 3 are regarded as overhangs and thus there is no internal pressure component.)

$GC_p = +0.4$ for Zones 1, 2, and 3
$GC_p = -0.85$ for Zone 1
$GC_p = -2.2$ for Zone 2 (with overhang)
$GC_p = -2.2$ for Zone 3 on hip roofs (with overhang)
$GC_p = -3.1$ for Zone 3 on gable roofs (with overhang)

Design pressures:

$p = 20.1(0.4 + 0.18) = +11.6$ psf (all zones)
$p = 20.1(-0.85 - 0.18) = -20.7$ psf (Zone 1)
$p = 20.1(-2.2) = -44.2$ psf (Zone 2)
$p = 20.1(-2.2) = -44.2$ psf (Zone 3 on hip roofs; Note 7)
$p = 20.1(-3.1) = -62.3$ psf (Zone 3 on gable roofs)

Fasteners

Effective area = 10 ft^2:

$GC_p = +0.5$ for Zones 1, 2, and 3
$GC_p = -0.9$ for Zone 1
$GC_p = -2.2$ for Zone 2 (with overhang)
$GC_p = -2.2$ for Zone 3 on hip roofs (with overhang)
$GC_p = -3.7$ for Zone 3 on gable roofs (with overhang)

Design pressures:

$p = 20.1(0.5 + 0.18) = +13.7$ psf (all zones)
$p = 20.1(-0.9 - 0.18) = -21.73$ psf (Zone 1)
$p = 20.1(-2.2) = -44.2$ psf (Zone 2)
$p = 20.1(-2.2) = -44.2$ psf (Zone 3 on hip roofs)
$p = 20.1(-3.7) = -74.4$ psf (Zone 3 on gable roofs)

Comment

The pressures determined are limit state design pressures for strength design. Section 2.3 of the Standard indicates load factor for wind load to be $1.0W$ for loads determined in this example. If allowable stress design is to be used, the load factor for wind load is $0.6W$ as shown in Section 2.4 of the Standard.

Chapter 9

U-Shaped Apartment Building

This example demonstrates calculation of wind loads for a U-shaped apartment building, shown in **Fig. G9-1**. Data for the building are provided in **Table G9-1**.

9.1 Analytical Procedure

The building is nonsymmetrical, and therefore the Analytical (Directional) Procedure of chapter 27, part 1, of the Standard is used for MWFRS. The building is less than 60 ft tall, so it is allowed to use Envelope low-rise provisions of Section 28.2 of the Standard. However, because U-, T-, and L-shaped buildings are not specifically covered, the adaptation of the low-rise "pseudo pressure" coefficients to buildings is outside the scope of the research and is not recommended. Therefore, use part 1 of the Directional Procedure of Section 27.2 of the Standard.

For components and cladding, provisions for the low-rise building ($h <$ 60 ft) of chapter 30, part 1, are appropriate to use.

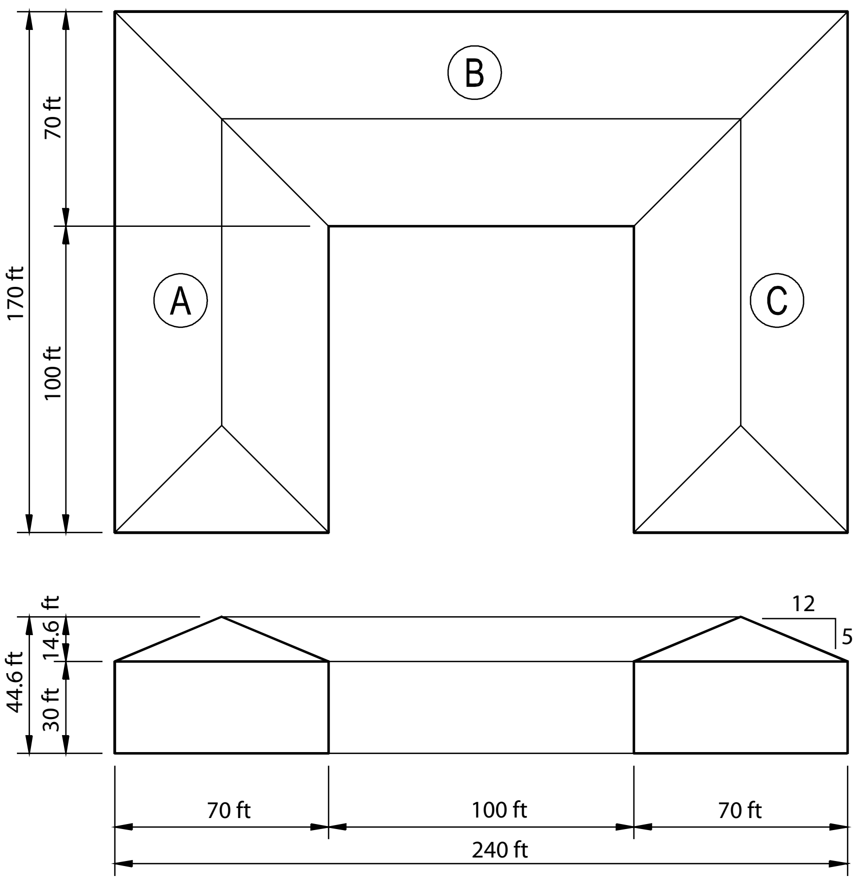
Building Classification

The building function is residential. It is not considered an essential facility, nor is it likely to be occupied by 300 persons in a single area at one time. Therefore, building Risk Category II is appropriate (see Table 1.5-1 of the Standard). Wind speed map of Fig. 26.5-1A of the Standard should be used.

Basic Wind Speed

Selection of the basic wind speed is addressed in Section 26.5 of the Standard. Birmingham, Alabama, is located just inside the 115-mph contour; therefore, the basic wind speed $V = 115$ mph (see Fig. 26.5-1A of the Standard).

Fig. G9-1
Building characteristics for U-shaped apartment building



Exposure

The building is located in a suburban area; according to Section 26.7 of the Standard, Exposure B is used.

Table G9-1 Data for U-Shaped Apartment Building

Location	Birmingham, Alabama
Topography	Homogeneous
Terrain	Suburban
Dimensions	170 ft × 240 ft overall in plan Roof eave height of 30 ft Hip roof with 5 on 12 pitch
Framing	Typical timber construction Wall studs are spaced at 16 in. on center, 10 ft tall Roof trusses are spaced at 24 in. on center, spanning 70 ft between exterior bearing walls Floor slab and roof sheathing provide diaphragm action
Cladding	Location is outside a wind-borne debris region, so no glazing protection is required. Window units are 3 ft × 4 ft

Enclosure

The building is designed to be enclosed. It is not located within a wind-borne debris region, so glazing protection is not required. The building is considered as an "enclosed" building for internal pressures.

Velocity Pressures

The velocity pressures for MWFRS and C&C are computed using the following equation:

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

where

K_z = value obtained from Table 27.3-1 of the Standard for MWFRS and Table 30.3-1 of the Standard for C&C

K_{zt} = 1.0 for homogeneous topography

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

V = 115 mph

$q_z = 0.00256\ K_z(1.0)(0.85)(115)^2 = 28.8\ K_z$ psf

Values for K_z and the resulting velocity pressures are given in **Table G9-2**. The mean roof height is the average of the eave and the peak:

$$h = 30 + (14.6/2) = 37.3 \text{ ft}$$

At the mean roof height, h = 37.3 ft; the velocity pressure is q_h = 21.6 psf.

Gust Effect Factor

Under the definition of Section 26.2 of the Standard, this building can be identified as a Low-Rise Building. In accordance with Section 26.9.2 of the Standard, a low-rise building can be considered as rigid; hence, gust effect factor can be taken as 0.85.

External Pressure Coefficients (C_p) for MWFRS

The values for the external pressure coefficients for the various surfaces (**Tables G9-3 through G9-6**) are obtained from Fig. 27.4-1 of the Standard for each of the surfaces in **Fig. G9-2**. The determination of certain pressure

Table G9-2 Velocity Pressures

	MWFRS		*C&C*	
Height (ft)	K_z	q_z *(psf)*	K_z	q_z *(psf)*
0 to 15	0.57	16.4	0.70	20.2
20	0.62	17.8	0.70	20.2
30	0.70	20.2	0.70	20.2
Mean roof h = 37.3	0.75	21.6	0.75	21.6

Table G9-3 External Pressure Coefficients for Wind Normal to Wall W2

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

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

Table G9-4 External Pressure Coefficients for Wind Normal to Wall W4

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

Table G9-5 External Pressure Coefficients for Wind Normal to Wall W3

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

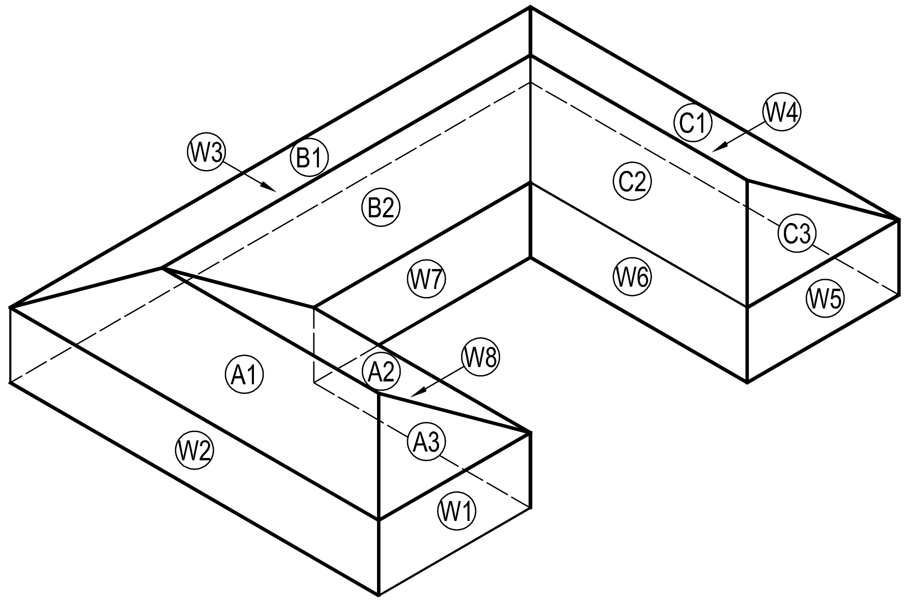
coefficients is based on aspect ratios. Even though this U-shaped building will be broken into pieces for the application of pressures, the overall dimensions have greater influence on the MWFRS pressure coefficients than the dimensions of the individual pieces. Therefore, the overall dimensions L and B are used.

When the wind is normal to wall W2, the wind blows over the "A" wing, crosses the courtyard in the middle of the U, and strikes the "C" wing. Although some reduction in the pressures on the "C" wing may occur

Table G9-6 External Pressure Coefficients for Wind Normal to Wall W1–W7–W5

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

Fig. G9-2
Surface designations for U-shaped apartment building



because of the shielding offered by "A," it is impossible to predict without a wind tunnel study. Therefore, the pressures on the "C" wing are taken as the same as on the "A" wing. There would also be reductions in pressures on the "C" wing if the "A" wing was taller, but again the amount of the reduction is not possible to predict without a wind tunnel study. If the wind impacts the "A" wing at an angle such that the wind would blow directly into the courtyard, then the "C" wing could still be affected by the full force of the wind; therefore, for this example, the judgment has been made that when the wind is normal to wall W2, wall W6 is also a windward wall (and likewise if the wind is normal to wall W4 where wall W8 is also treated as a windward wall).

For wind normal to surface W2 or W4

$L/B = 240/170 = 1.41$
$h/L = 37.3/240 = 0.16$
$\theta = 22.6°$ for a 5-in-12 slope

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

$L/B = 170/240 = 0.71$
$h/L = 37.3/170 = 0.22$
$\theta = 22.6°$ for a 5-in-12 slope

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

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

Design Wind Pressures for the MWFRS

The design pressures for this building are obtained by the equation

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

where

$q = q_z$ for windward wall at height z above ground
$q = q_h = 21.6$ psf for leeward wall, side walls, and roof
$q_i = q_h = 21.6$ psf for all surfaces since the building is enclosed
$G = 0.85$, the gust effect factor for rigid buildings and structures
C_p = external pressure coefficient for each surface as shown in **Tables G9-7 through G9-10**
$(GC_{pi}) = \pm 0.18$, the internal pressure coefficient for enclosed buildings

For windward walls

$$p = q_zGC_p - q_h(GC_{pi}) = q_z(0.85)C_p - 21.6(\pm 0.18) = 0.85q_zC_p \pm 3.9$$

For all other surfaces

$$p = q_hGC_p - q_h(GC_{pi}) = 21.6(0.85)C_p - 21.6(\pm 0.18) = 18.4C_p \pm 3.9$$

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

Table G9-7 External Pressures for Wind Normal to Wall W2

Surface type	*Surface designation*	*z or x (ft)*	*q (psf)*	C_p	*External pressure (psf)*	*Design pressures (psf)* $(+GC_{pi})$	*Design pressures (psf)* $(-GC_{pi})$
Walls	W2, W6	0 to 15	16.4	+0.80	+11.2	+7.3	+15.1
		20	17.8	+0.80	+12.1	+8.2	+16.0
		30	20.2	+0.80	+13.7	+9.8	+17.6
	W4, W8	0 to 30	21.6	−0.42	−7.7	−11.6	−3.8
	W1, W3, W5, W7	0 to 30	21.6	−0.70	−12.8	−16.7	−8.9
Roofs (⊥ to ridge)	A1, C2		21.6	−0.25	−4.6	−8.5	−0.7
			21.6	+0.25	+4.6	+0.7	+8.5
	A2, C1		21.6	−0.60	−11.0	−14.9	−7.1
Roofs (‖ to ridge)	A3, C3	0 to 37.3	21.6	−0.90	−16.5	−20.4	−12.6
		37.3 to 70	21.6	−0.50	−9.2	−13.1	−5.3
	B1, B2	0 to 37.3	21.6	−0.90	−16.5	−20.4	−12.6
		37.2 to 74.6	21.6	−0.50	−9.2	−13.1	−5.3
		74.6 to 240	21.6	−0.30	−5.5	−9.4	−1.6

Note: $q_h = 21.6$ psf; $G = 0.85$.

Table G9-8 External Pressures for Wind Normal to Wall W4

Surface type	*Surface designation*	*z or x (ft)*	*q (psf)*	C_p	*External pressure (psf)*	*Design pressures (psf)* ($+GC_{pi}$)	($-GC_{pi}$)
Walls	W4, W8	0 to 15	16.4	+0.80	+11.2	+7.3	+15.1
		20	17.8	+0.80	+12.1	+8.2	+16.0
		30	20.2	+0.80	+13.7	+9.8	+17.6
	W6, W2	0 to 30	21.6	−0.42	−7.7	−11.6	−3.8
	W1, W3, W5, W7	0 to 30	21.6	−0.70	−12.8	−16.7	−8.9
Roofs (⊥ to ridge)	C1, A2		21.6	−0.25	−4.6	−8.5	−0.7
			21.6	+0.25	+4.6	+0.7	+8.5
	C2, A1		21.6	−0.60	−11.0	−14.9	−7.1
Roofs (‖ to ridge)	A3, C3	0 to 37.3	21.6	−0.90	−16.5	−20.4	−12.6
		37.3 to 70	21.6	−0.50	−9.2	−13.1	−5.3
	B1, B2	0 to 37.3	21.6	−0.90	−16.5	−20.4	−12.6
		37.2 to 74.6	21.6	−0.50	−9.2	−13.1	−5.3
		74.6 to 240	21.6	−0.30	−5.5	−9.4	−1.6

Note: $q_h = 21.6$ psf; $G = 0.85$.

Table G9-9 External Pressures for Wind Normal to Wall W3

Surface type	*Surface designation*	*z (ft)*	*q (psf)*	C_p	*External pressure (psf)*	*Design pressures (psf)* ($+GC_{pi}$)	($-GC_{pi}$)
Walls	W3	0 to 15	16.4	+0.80	+11.2	+7.3	+15.1
		20	17.8	+0.80	+12.1	+8.2	+16.0
		30	20.2	+0.80	+13.7	+9.8	+17.6
	W1, W7, W5	0 to 30	21.6	−0.50	−9.2	−13.1	−5.3
	W2, W4, W6, W8	0 to 30	21.6	−0.70	−12.8	−16.7	−8.9
Roofs (⊥ to ridge)	B1		21.6	−0.25	−4.6	−8.5	−0.7
			21.6	+0.25	+4.6	+0.7	+8.5
	A3, B2, C3		21.6	−0.60	−11.0	−14.9	−7.1
Roofs (‖ to ridge)	A1, A2, C1,C2	0 to 37.3	21.6	−0.90	−16.5	−20.4	−12.6
		37.3 to 74.6	21.6	−0.50	−9.2	−13.1	−5.3
		74.6 to 170	21.6	−0.30	−5.5	−9.4	−1.6

Note: $q_h = 21.6$ psf; $G = 0.85$.

Table G9-10 External Pressures for Wind Normal to Wall W1–W7–W5

Surface type	*Surface designation*	*z (ft)*	*q (psf)*	C_p	*External pressure (psf)*	*Design pressures (psf)* ($+GC_{pi}$)	($-GC_{pi}$)
Walls	W1, W7, W5	0 to 15	16.4	+0.80	+11.2	+7.3	+15.1
		20	17.8	+0.80	+12.1	+8.2	+16.0
		30	20.2	+0.80	+13.7	+9.8	+17.6
	W3	0 to 30	21.6	−0.50	−9.2	−13.1	−5.3
	W2, W4, W6, W8	0 to 30	21.6	−0.70	−12.8	−16.7	−8.9
Roofs (⊥ to ridge)	A3, B2, C3		21.6	−0.25	−4.6	−8.5	−0.7
			21.6	+0.25	+4.6	+0.7	+8.5
	B1		21.6	−0.60	−11.0	−14.9	−7.1
Roofs (‖ to ridge)	A1, A2, C1, C2	0 to 37.3	21.6	−0.90	−16.5	−20.4	−12.6
		37.3 to 74.6	21.6	−0.50	−9.2	−13.1	−5.3
		74.6 to 170	21.6	−0.30	−5.5	−9.4	−1.6

Note: $q_h = 21.6$ psf; $G = 0.85$.

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 **Tables G9-7 through G9-10**. A combination of windward (P_W) and leeward (P_L) loads are applied for Load Cases 2, 3, and 4 as shown in **Fig. G9-3**.

For Load Case 2, there are two loading conditions shown; both of them need to be checked independently. The eccentricities are calculated as follows:

$$e_x = 0.15B_x = 0.15(170) = 25.5 \text{ ft}$$

and

$$e_y = 0.15B_y = 0.15(240) = 36 \text{ ft}$$

Design Pressures for Components and Cladding

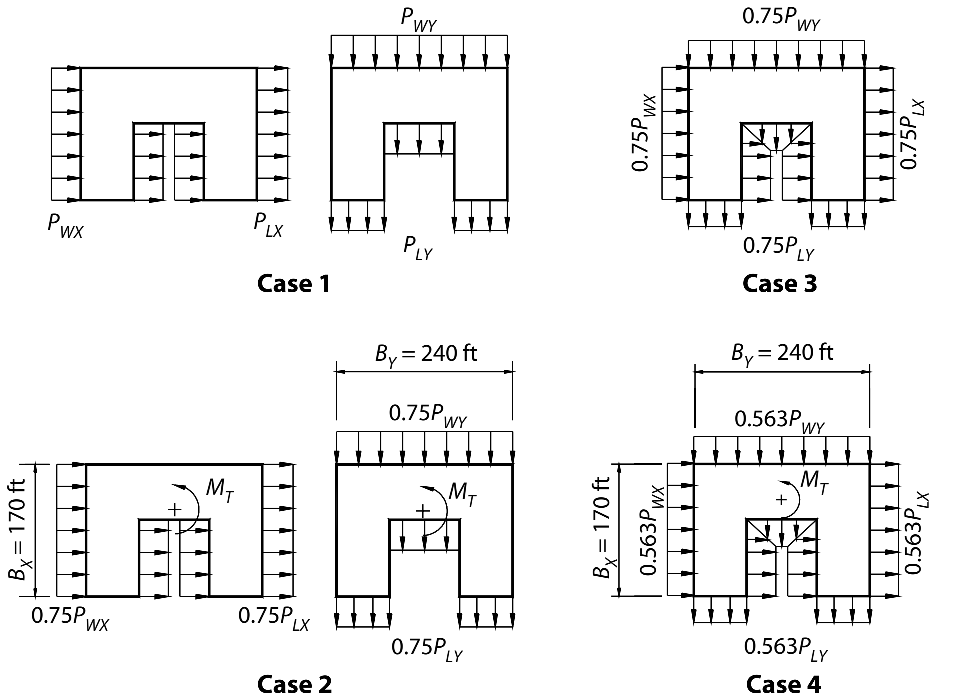
Design pressure for C&C is obtained by the provisions of chapter 30, part 1 of the Standard. The equation is:

$$p = q_h[(GC_p) - (GC_{pi})] \qquad \text{(Eq. 30.4-1)}$$

where

q_h = 21.6 psf
(GC_p) = external pressure coefficient (see Fig. 30.4-1 of the Standard)
(GC_{pi}) = ±0.18, the internal pressure coefficient for enclosed buildings

Fig. G9-3
Design wind load cases for wind normal to walls W2 and W3



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

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

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

Wall Design Pressures

The pressure coefficients (GC_p) are a function of effective wind area (**Table G9-11**). 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 as follows:

Window Unit

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

Wall Stud

larger of	$A = 10(1.33) = 13.3 \text{ ft}^2$
or	$A = 10(10/3) = 33.3 \text{ ft}^2$ (controls)

Table G9-11 Wall Design Pressures for Components and Cladding

Component	A *(ft²)*	*Zones 4 and 5* (+GC_p)	*Zone 4* (–GC_p)	*Zone 5* (–GC_p)
Window	12	+0.99	−1.09	−1.37
Wall stud	33.3	+0.91	−1.01	−1.22

Width of Corner Zone 5

smaller of $a = 0.1(170) = 17$ ft
or $a = 0.1(240) = 24$ ft
or $a = 0.4(37.3) = 14.9$ ft (controls)

but not less than the smaller of

$a = 0.04(170) = 6.8$ ft
or $a = 0.04(240) = 9.6$ ft

and not less than

$a = 3$ ft

Typical Design Pressure Calculations

Controlling design pressures for wall components are presented in **Table G9-12**.

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

$= 21.6[(-1.09) - (\pm 0.18)]$
$= -27.4$ psf (positive internal pressure controls)

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

$= 21.6[(+0.99) - (\pm 0.18)]$
$= 25.3$ psf (negative internal pressure controls)

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

The edges called Zone 5 for the walls are arranged at exterior corners, as shown in **Fig. G9-4**.

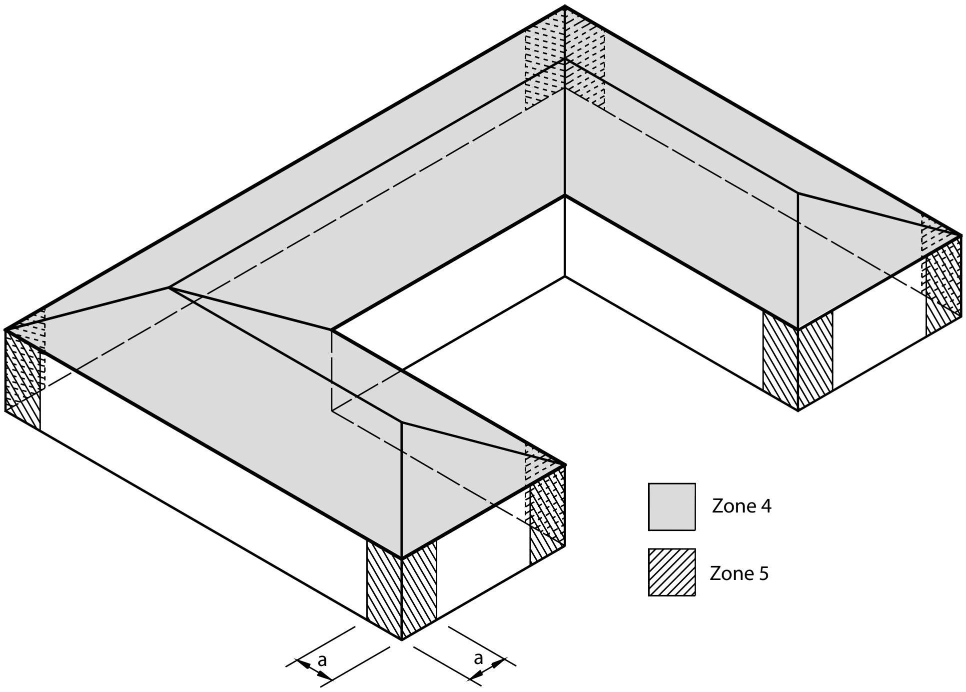
Roof Design Pressures

The C&C roof pressure coefficients for $\theta = 22.6°$ are given in Fig. 30.4-2B of the Standard and presented in **Tables G9-13 and G9-14**. The pressure coefficients are a function of the effective wind area. The definition of effective wind area for a component or cladding panel is the span length multiplied by an effective width that need not be less than one-third the span length (see Section 26.2 of the Standard). The effective wind areas, A, for the roof trusses are as follows:

Table G9-12 Controlling Design Pressures for Wall Components

	Design pressure (psf)			
	Zone 4		*Zone 5*	
Component	*Positive*	*Negative*	*Positive*	*Negative*
Window unit	+25.3	−27.4	+25.3	−33.5
Wall stud	+23.5	−25.7	+23.5	−30.2

Fig. G9-4
Component and cladding wall pressure zones



Roof Truss Top Chord

larger of $A = 70(2.0) = 140\ \text{ft}^2$
or $A = 70(70/3) = 1{,}633\ \text{ft}^2$ (controls)

Note 7 of Fig. 30.4-2B of the Standard says that for hip roofs with $\theta \le 25°$, Zone 3 may be treated as Zone 2.

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

The edges Zone 2 for the hip roof are arranged as shown in **Fig. G9-5**.

Comment

The pressures determined are limit state design pressures for strength design. Section 2.3 of the Standard indicates load factor for wind load to be $1.0W$ for loads determined in this example. If allowable stress design is to be used, the load factor for wind load is $0.6W$ as shown in Section 2.4 of the Standard.

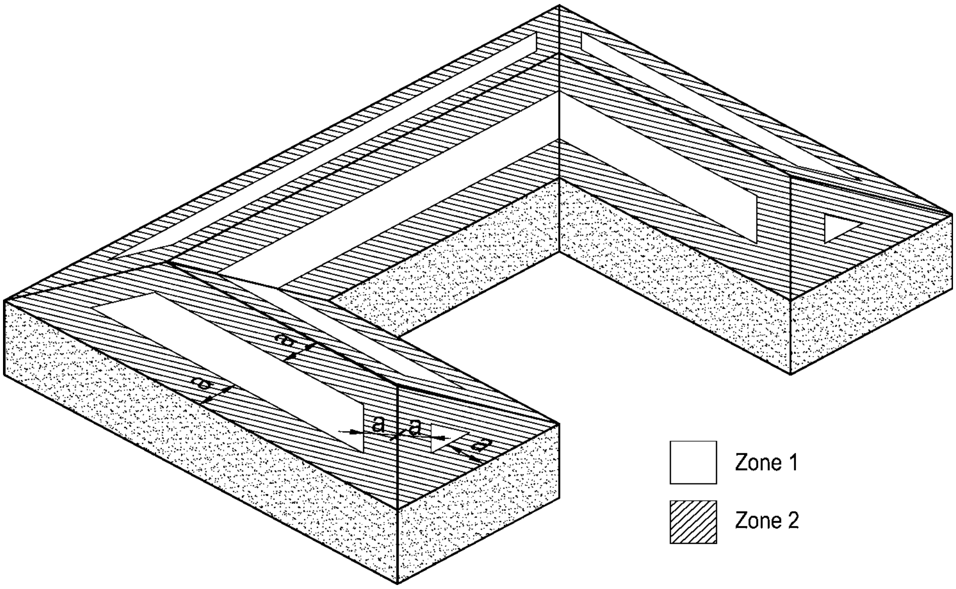
Table G9-13 Roof External Pressure Coefficients

	Positive	*Negative*	
	Zones 1, 2	*Zone 1*	*Zone 2*
A *(ft²)*	GC_p	GC_p	$-GC_p$
1,633	+0.3	–0.80	–1.2

Table G9-14 Roof Design Pressures

	Design pressure (psf)		
	Zones 1, 2	*Zone 1*	*Zone 2*
Component	*Positive*	*Negative*	*Negative*
Roof truss	+10.4	−21.2	−29.8

Fig. G9-5
Component and cladding roof pressure zones



Chapter 10

Open Building with Gable Roof

Wind pressures for a typical open storage building are determined. **Fig. G10-1** shows the dimensions and framing of the open buildings. The physical data are as listed in **Table G10-1.**

Procedure

Wind loads on MWFRS are obtained using the Analytical (Directional) Procedure of chapter 27, part 1, because the example building is an open building. Even though the building height is $h < 60$ ft and qualifies as a low-rise building by definition, adaptation of the low-rise "pseudo pressure" coefficients of chapter 28, part 1, is not appropriate for this example building. Therefore, the Directional Procedure of Section 27.4.1 is used.

C&C wind loads are obtained using provisions of chapter 30, part 5, for open buildings.

Building Risk Classification

A storage building of this type has a low risk to human life; Risk Category I is appropriate for this building. The wind speed map associated with Risk Category I is in Fig. 26.5-1C of the Standard.

Basic Wind Speed

Basic wind speed in any location in Oklahoma is 105 mph; see Fig. 26.5-1C of the Standard.

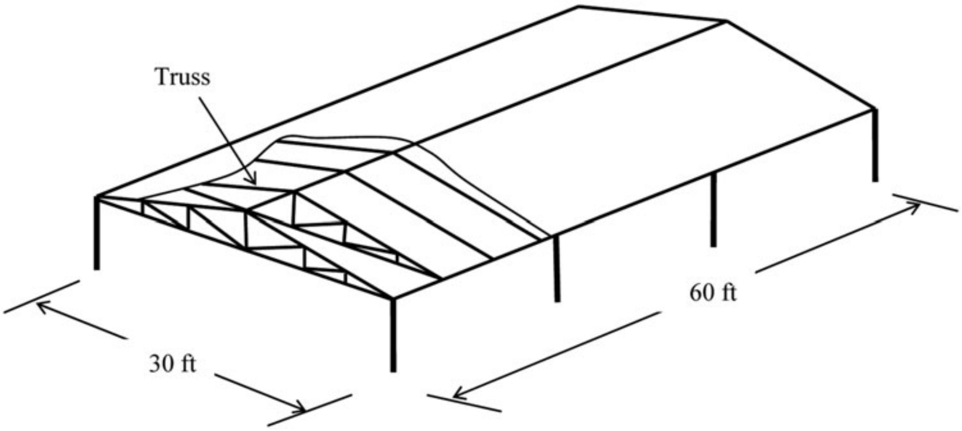
Exposure

The building is located in a wooded area. Section 26.7.2 of the Standard defines terrain roughness as Exposure B.

Enclosure Classification

The building has all walls open; it qualifies as an open building. Value of $(GC_{pi}) = 0.0$ (see Table 26.11-1 in the Standard).

Fig. G10-1
Building characteristics for open building with gable roof



Velocity Pressures

Velocity pressure is determined using equations from the Standard: Eq. 27.3-1 for MWFRS and Eq. 30.3-1 for C&C. The equation is the same for MWFRS and C&C.

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

where

q_z = velocity pressure at height z
K_z = exposure coefficient at height z and K_h at mean roof height
K_{zt} = 1.0 for homogeneous terrain (Section 26.8 of the Standard)
K_d = 0.85 for buildings (Table 26.6-1 of the Standard)
V = 105 mph (Fig. 26.5-1C of the Standard)

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

Table G10-1 Data for Open Building with Gable Roof

Location	Tulsa, Oklahoma
Topography	Homogeneous
Terrain	Suburban, wooded
Dimensions	30 ft × 60 ft in plan Roof eave height is 10 ft Roof gable θ = 15°
Framing	Typical metal construction Roof trusses spanning 30 ft are spaced 4 ft on center Roof panels are 2 ft × 8 ft
Storage	Clear flow; blockage less than 50 percent

Since K_z is constant in the 0 to 15 ft region, from Table 27.3-1 of the Standard for MWFRS and Table 30.3-1 for C&C:

For MWFRS	$K_z = K_h = 0.57$
For C&C	$K_z = K_h = 0.70$
For MWFRS	$q_h = 0.00256\ (0.57)\ (1.0)\ (0.85)\ (105)^2 = 13.7$ psf
For C&C	$q_h = 0.00256\ (0.7)\ (1.0)\ (0.85)\ (105)^2 = 16.8$ psf

Design Wind Pressure for MWFRS

The equation for open buildings is given in Section 27.4.3 of the Standard:

$$p = q_h G C_N \qquad \text{(Eq. 27.4-3)}$$

where

q_h = velocity pressure evaluated at mean roof height h

G = gust effect factor

C_N = net pressure coefficient value obtained from Fig. 27.4-4 through Fig. 27.4-7 of the Standard.

Gust Effect Factor

This building fits the definition of a low-rise building, so according to Section 26.9.2 of the Standard it can be considered rigid. The gust effect factor for nonflexible (rigid) buildings is given in Section 26.9.1 of the Standard as $G = 0.85$.

Roof Net Pressure Coefficients

Because the building is open and has a pitched roof, there are two wind directions to be considered: wind direction parallel to the slope (normal to ridge), $\gamma = 0°$ or 180°, and wind normal to the roof slope (parallel to ridge), $\gamma = 90°$. For $\gamma = 0°$ or 180°, the net pressure coefficients are obtained from Fig. 27.4-5 of the Standard. For $\gamma = 90°$, the net pressure coefficients are obtained from Fig. 27.4-7. The roof net pressure coefficients are presented in **Table G10-2**.

Clear wind flow is assumed; blockage is less than 50 percent.

Table G10-2 Roof Net Pressure Coefficients for Two Cases

Wind direction	$\theta°$	*Distance from windward edge*	*Case A*	*Case B*
Normal to ridge $\gamma = 0°$ or 180°; Fig. 27.4-5	15		1.1 (C_{NW})	0.1 (C_{NW})
			–0.4 (C_{NL})	–1.1 (C_{NL})
Parallel to ridge $\gamma = 90°$; Fig. 27.4-7	0	0 to 12 ft	–0.8	0.8
		12 to 24 ft	–0.6	0.5
		24 to 60 ft	–0.3	0.3

Wind Pressure for MWFRS

Calculated wind pressures for MWFRS are summarized in **Table G10-3**.

$$p = q_h G C_N = 13.7 \times 0.85 \times C_N$$

Figures G10-2 and G10-3 illustrate the design pressures (Case A and Case B) for wind direction $\gamma = 0°$ or $180°$ and $\gamma = 90°$, respectively.

Minimum Design Wind Loadings

Section 27.4.7 of the Standard requires that the design wind load for MWFRS of open buildings shall not be less than 16 psf (net horizontal) multiplied by area projected on a plane normal to the wind direction. Depending on the projected area of the roof and supporting structure, this minimum loading could govern and should be checked. Load Case A satisfies this minimum requirement.

Design Pressures for Roof Trusses and Roof Panels

The design pressure equation for C&C for open buildings with pitched roofs is given in Section 30.8 of the Standard:

$$p = q_h G C_N \qquad \text{(Eq. 30.8-1)}$$

where

q_h = velocity pressure evaluated at mean roof height h

G = gust effect factor value determined from Section 26.9 of the Standard

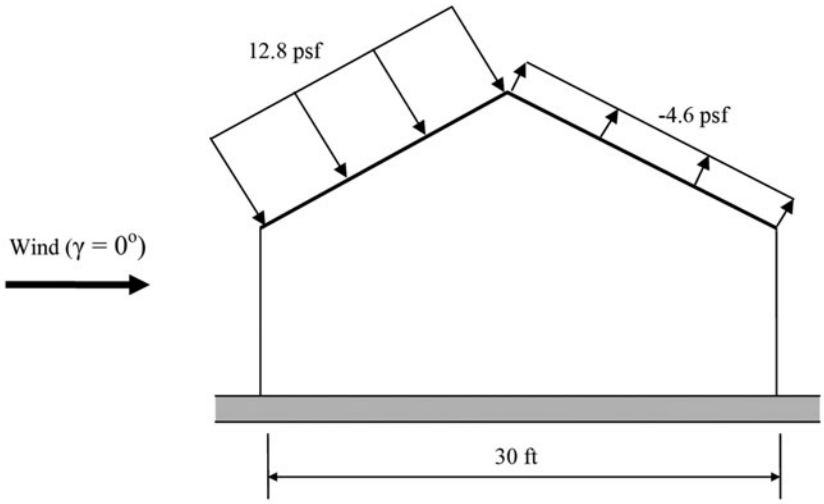
C_N = net pressure coefficient value obtained from Fig. 30.8-2 of the Standard

Table G10-3 Design Wind Pressures for MWFRS for Two Cases (psf)

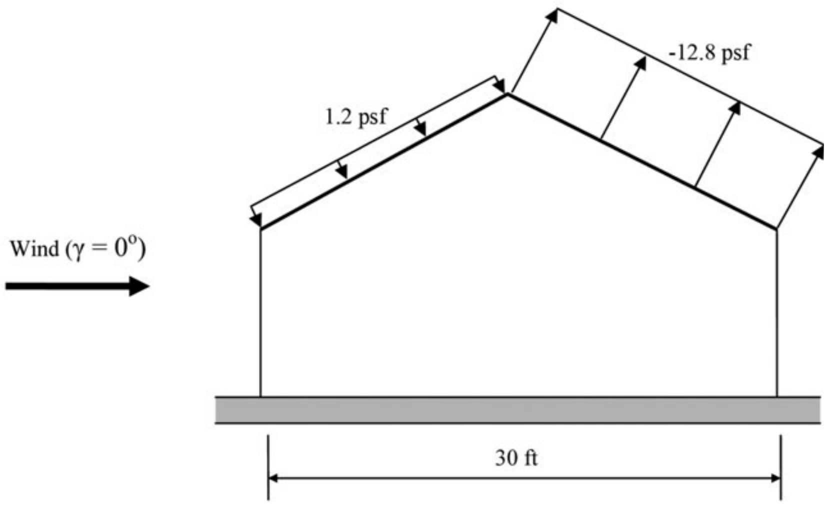
Wind direction, γ	*Distance from windward edge*	*Case A*	*Case B*
Normal to ridge			
$\gamma = 0°$ or $180°$			
Windward		12.8	1.2
Leeward		–4.6	–12.8
Parallel to ridge			
$\gamma = 90°$	0 to 12 ft	–9.3	9.3
	12 to 14 ft	–7.0	5.8
	24 to 60 ft	–3.5	3.5

Note: Positive numbers mean toward the surface; negative numbers mean away from the surface.

Fig. G10-2
Design pressures for MWFRS for wind direction $\gamma = 0°$ or $180°$



Case A



Case B

Roof Design Pressures

Roof net design coefficients for components and cladding are presented in **Table G10-4,** using Fig. 30.8-2 of the Standard.

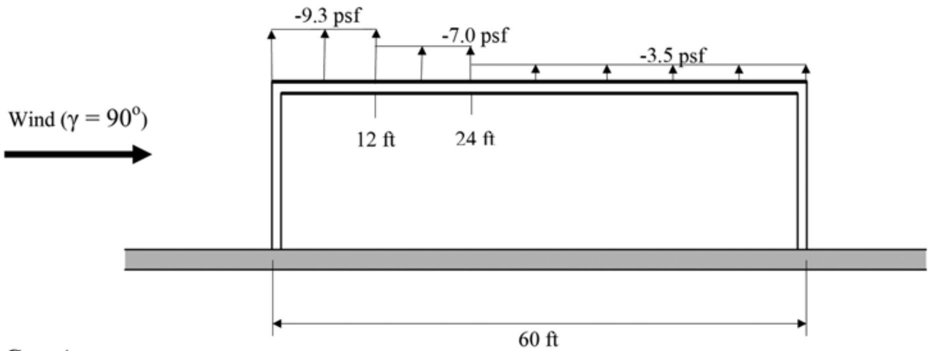
Width of Zone 2 and Zone 3

Smaller of	$a = 0.1(30) = 3$ ft (controls)
or	$a = 0.4(12) = 4.8$ ft
but not less than	$a = 0.04(30) = 1.2$ ft
or	$a = 3$ ft
	$a^2 = 9$ ft^2

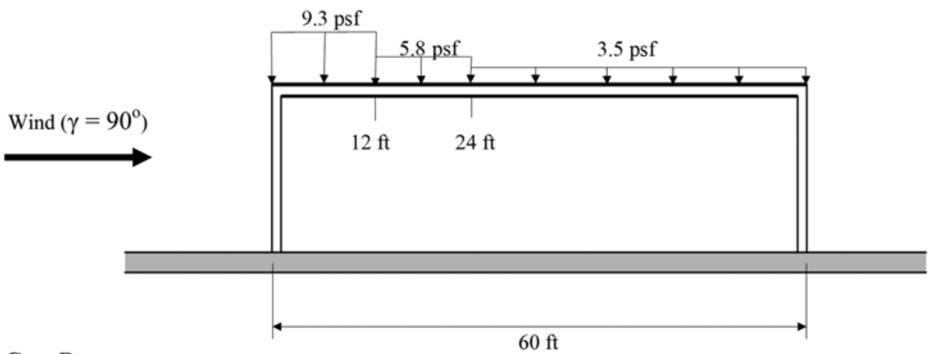
Effective wind area

Roof panel:	$A = 4 \times 2 = 8$ ft^2 (controls)
or	$A = 4 \times (4/3) = 5.3$ ft^2
Roof truss:	$A = 4 \times 30 = 120$ ft^2
or	$A = 30 \times (30/3) = 300$ ft^2 (controls)

Fig. G10-3
Design pressures for MWFRS for wind direction $\gamma = 90°$. The pressures are normal to roof surface



Case A



Case B

Wind Pressure for Trusses and Roof Panels

Calculated wind pressures for trusses and roof panels are summarized in **Table G10-5**.

$$p = q_h GC_N = 16.8 \times 0.85 \times C_N$$

Zones for the pitched roof of this open building are shown in **Fig. G10-4**. The panels and trusses are designed for the pressures indicated.

For trusses, two loading combinations need to be considered. The two loading cases are shown in **Fig. G10-5**. Loading Case 1 is for Wind Directions 1 and 2 while Loading Case 2 is for Wind Direction 2. The loadings shown for trusses are used for the design of truss and individual members. For anchorage of the truss to frame support members, you may use MWFRS loading.

Table G10-4 Roof Net Pressure Coefficients for Components, $\theta = 15°$

Component	*Effective Wind Area (sq ft)*	*Zone 3*		*Zone 2*		*Zone 1*	
Panel	8 ($\leq a^2$)	2.2	−2.2	1.7	−1.7	1.1	−1.1
Truss	300 ($>4a^2$)	1.1	−1.1	1.1	−1.1	1.1	−1.1

Note: Coefficients are from Fig. 30.8-2 of the Standard.

Table G10-5 Roof Component Design Pressure by Zone (psf)

Component	*Zone 3*		*Zone 2*		*Zone 1*	
Panel	31.4	–31.4	24.3	–24.3	15.7	–15.7
Truss	15.7	–15.7	15.7	–15.7	15.7	–15.7

Note: Positive numbers mean toward the surface; negative numbers mean away from the surface. Note that Section 30.2.2 requires that a minimum pressure of 16 psf be used.

Fig. G10-4
Pressure zones for panels and trusses

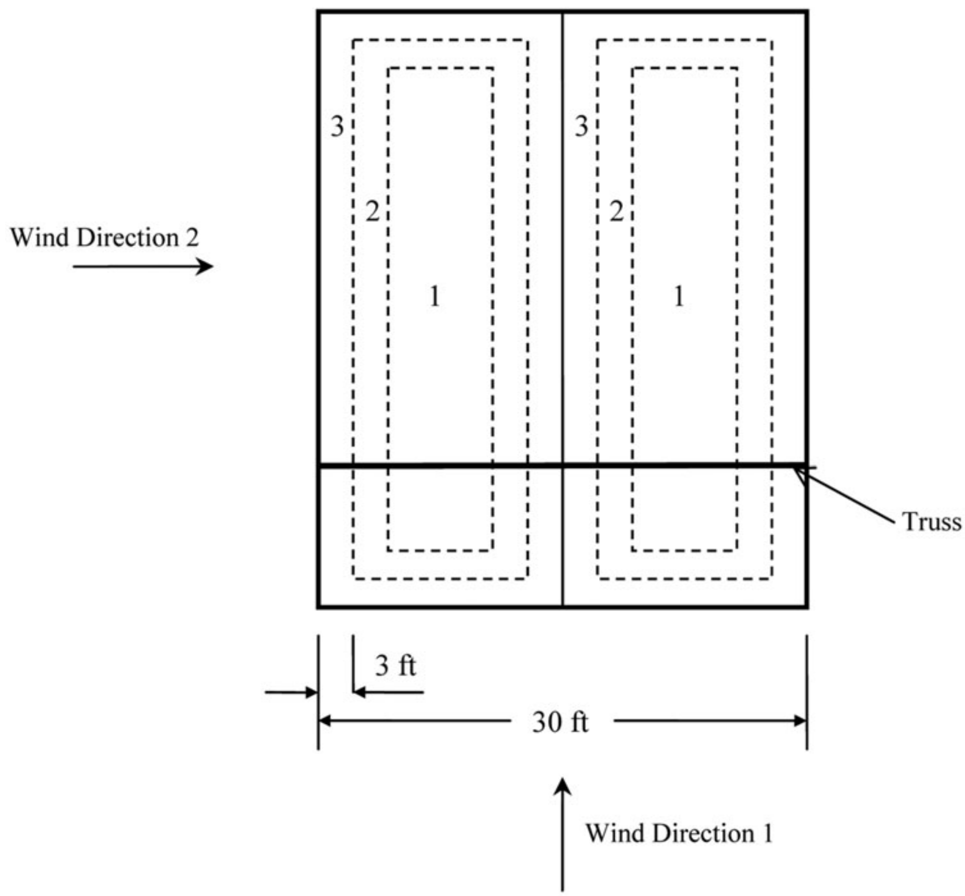
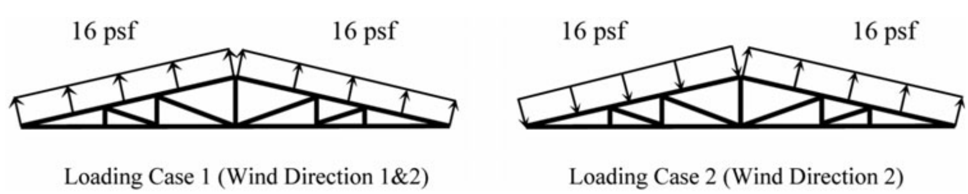


Fig. G10-5
Loading cases for room trusses



Comment

The pressures determined are limit state design pressures for strength design. Section 2.3 of the Standard indicates load factor for wind load to be $1.0W$ for loads determined in this example. If allowable stress design is to be used, the load factor for wind load is $0.6W$ as shown in Section 2.4 of the Standard.

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Chapter 11

Domed Roof Building

Figure G11-1 illustrates the domed roof building used for a church in this example. Building data are as listed in **Table G11-1**.

11.1 Analytical Procedure

Domed roofs are outside the scope of the Envelope Procedure of ASCE 7-10 since the roof shape does not comply with the restrictions of that procedure, therefore the Directional Procedure of chapter 27, part 1, is used.

Building Classification

The building is a church, so it will have more than 300 people congregating in one area. While Table 1.5-1 of the Standard is not conclusive on the classification, the commentary section C1.5.1 does suggest that with the assembly use, risk Category III is appropriate.

Wind speed map for this Category building is in Fig. 26.5-1B of the Standard.

Basic Wind Speed

Selection of the basic wind speed is addressed in Section 26.5.1 of the Standard, and the wind map for Category III buildings is Fig. 26.5-1B. Baton Rouge, Louisiana, is located between the 130-mph and 140-mph contours; therefore, the basic wind speed $V = 135$ mph (see Fig. 26.5-1B of the Standard).

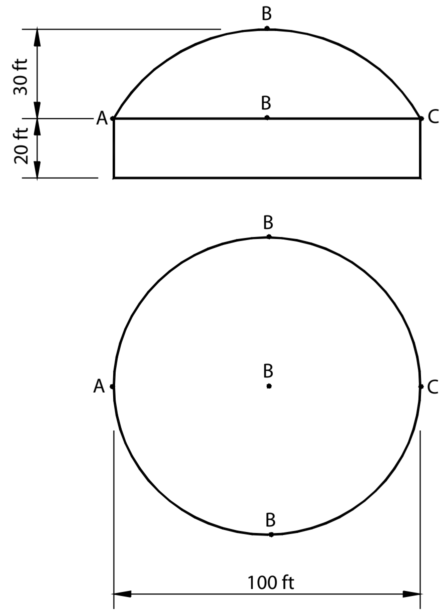
Exposure

The building is located in an open terrain area; according to Section 26.7 of the Standard, Exposure C is used.

Enclosure

The building is designed to be enclosed. It is not located within a wind-borne debris region, so glazing protection is not required. Because this is a Category III building, the wind-borne debris region is defined as being located in an

Fig. G11-1
Building characteristics for domed roof structure



area where the wind speed exceeds 130 mph and is within a mile of coastal mean high water or anywhere where the wind speed exceeds 140 mph. This building location is close to meeting this definition and while not required, the owner might want to consider wind-borne debris protection.

Velocity Pressures

The velocity pressures are computed using the following equation:

$$q_z = 0.00256\ K_z K_{zt} K_d V^2 \text{ psf}$$ (Eq. 27.3-1 for MWFRS and Eq. 30.3-1 for C&C)

where

K_z = value obtained from Table 27.3-1 of the Standard for MWFRS and from Table 30.3-1 of the Standard for C&C of the Standard; note that for Exposure C the values are the same

Table G11-1 Data for Domed Roof Building

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

K_{zt} = 1.0 for homogeneous topography
K_d = 0.95 for round tanks and similar structures (see Table 26.6-1 of the Standard)
V = 135 mph

$$q_z = 0.00256\ K_z(1.0)(0.95)(135)^2 = 44.3\ K_z \text{ psf}$$

Values for K_z and the resulting velocity pressures are given in **Table G11-2**. Wall pressures will be evaluated at midheight = 20 ft/2 = 10 ft – 15 ft; use q_z at 15 ft.

Design Wind Pressures for the MWFRS

Gust Effect Factor

Section 26.9.2 of the Standard permits consideration of the building as rigid if the building fits in the definition of a low-rise building. The example building is a low-rise building, so it can be considered a rigid building, and the value of the gust effect factor of 0.85 is used.

Wall Pressures

The walls of a round building are not specifically covered by the Standard. The values for the force coefficients for round tanks and chimneys from Fig. 29.5-1 of the Standard is used to determine the effect of the wall pressures on the MWFRS. The values of the force coefficients for round tanks vary with the aspect ratio of height to diameter and with the surface roughness.

The value of q_z varies from 37.6 psf at the ground to 39.9 psf at the eave line. Therefore, the ratio $D\sqrt{q_z}$ varies from $100\sqrt{37.6} = 613$ to $100\sqrt{39.9} = 632$, both of which are much greater than 2.5; therefore, the first set of values for C_f for round tanks in Fig. 29.5-1 of the Standard is used. Any projections on the exterior skin of the building are assumed to be less than 2 ft; therefore, D'/D would be less than 2 ft/100 ft = 0.02, so the building is considered moderately smooth. The height of the entire structure (h = 50 ft) is used for the aspect ratio, since the wind has to travel over the dome. Therefore, h/D = 50 ft/100 ft = 0.5, which is less than 1, resulting in C_f = 0.5.

The force on the walls represents the total drag of the wind on the walls of the building, both windward and leeward. Since it is not the typical

Table G11-2 Velocity Pressures

	MWFRS		*C&C*	
Height (ft)	K_z	q_z *(psf)*	K_z	q_z *(psf)*
0 to 15	0.85	37.6	0.85	37.6
Eave height = 20	0.90	39.9	0.90	39.9
Top of dome = 50	1.09	48.3	1.09	48.3

pressures applied normal to the wall surfaces, ignore internal pressures, as they cancel out in the net drag calculation.

Total drag force on walls = $F = q_z G C_f A_f$ (Eq. 29.5-1)

where

q_z = q at the centroid of A_f – centroid of A_f is at wall midheight = 20 ft/2 = 10 ft

q = 37.6 psf (at 10 ft)

G = 0.85, the gust effect factor for rigid structures

C_f = 0.5

A_f = 100 ft × 20 ft = 2,000 ft^2

Total drag force on walls = F = 37.6(0.85)(0.5)(2,000) = 31,960 lb

Domed Roof Pressures

The roof pressure coefficients for a domed roof are taken from Fig. 27.4-2 of the Standard. The height from the ground to the spring line of the dome, h_D = 20 ft. The height of the dome itself from the spring line to the top of the dome, f = 30 ft. Determine C_p for a rise to diameter ratio, f/D = 30/100 = 0.30; and a base height to diameter ratio, h_D/D = 20/100 = 0.20. Interpolation from Fig. 27.4-2 of the Standard is required. Pressure coefficient values for f/D = 0.30 for points A, B, and C on the dome are given in **Table G11-3**.

Two load cases are required for the MWFRS loads on domes: Cases A and B. Case A is based on linear interpolation of C_p values from point A to B and from point B to C (see **Fig. G11-1** of this guide for the locations of points A, B, and C). Case B uses the pressure coefficient at *A* for the entire front area of the dome up to an angle θ = 25°, then interpolates the values for the rest of the dome as in Case A.

Case A

For design purposes, interpolate the pressure coefficients at points at 10-ft intervals along the dome (see **Table G11-4**).

Case B

Determine the point on the front of the dome at which θ = 25°. The point is 36.2 ft from the center of the dome, therefore 13.8 ft from point *A*. The pressure coefficient at *A* shall be used for the section from *A* to an arc 13.8 ft

Table G11-3 Roof Pressure Coefficients for Domed Roof at f/D = 0.30

Point on dome in Figure G11-1	h_D/D = *0*	h_D/D = *0.20*	h_D/D = *0.25*	h_D/D = *0.50*
A	+0.5	–0.04	–0.18	—
B	–0.78	–0.97	—	–1.26
C	0	–0.20	—	–0.50

Table G11-4 Interpolated Domed Roof Pressure Coefficients, Case A

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

from *A*. The remainder of the dome pressures is based on linear interpolation between the 25° point and point B; and then from point B to C (see **Table G11-5**).

Internal Pressure Coefficient for Domed Roof

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

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

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

For enclosed buildings:

$$GC_{pi} = \pm 0.18 \qquad \text{(Table 26.11-1)}$$

q_i is taken as $q_{(hD+f)} = 48.3$ psf

Design internal pressure

$$q_i(GC_{pi}) = 48.3(\pm 0.18) = \pm 8.7 \text{ psf}$$

Design Wind Pressures for Domed Roof

The design pressures for this building (shown in **Fig. G11-2**) are obtained by the equation

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

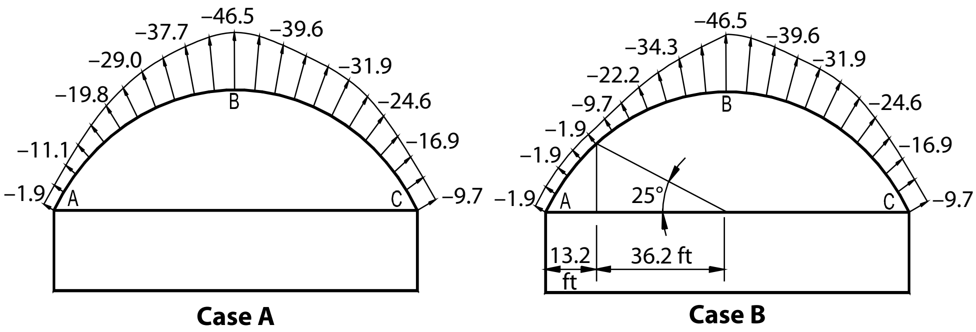
where

$q = q_{(hD+f)} = 48.3$ psf (see Note 2 of Fig. 27.4-2 of the Standard)
$G = 0.85$, the gust effect factor for rigid buildings and structures

Table G11-5 Interpolated Domed Roof Pressure Coefficients, Case B

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

Fig. G11-2
MWFRS external pressures for domed roof. (Internal pressure of ±8.7 psf to be added)



C_p = external pressure coefficient
q_i = q_h for all surfaces since the building is enclosed
GC_{pi} = ±0.18, the internal pressure coefficient for enclosed buildings

$$p = 48.3(0.85)C_p - 48.3(\pm 0.18) = 41.0C_p \pm 8.7$$

Values of design pressures for MWFRS are show in **Table G11-6**.

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

Design Pressures for Components and Cladding

Design pressure for C&C (**Fig. G11-3**) is obtained by

$$p = q_h[(GC_p) - (GC_{pi})] \qquad \text{(Eq. 30.4-1)}$$

where

$q_h = q_{(h_D+f)}$ = 48.3 psf for all domed roofs calculated at height h_D+f
$q_i = q_{(h_D+f)}$ = 48.3 psf for positive and negative internal pressure
(GC_p) = external pressure coefficient (see Fig. 30.4-7 of the Standard)
(GC_{pi}) = ±0.18 for internal pressure coefficient (see Table 26.11-1 of the Standard)

Wall Design Pressures

The Standard does not address component and cladding wall loads for round buildings. The designer might consider segmenting these walls into small widths and using the roof coefficients for the walls. The designer could also use the Figs. 30.4-1 or 30.6-1 (depending on h), and apply the coefficients for wall Zone 4. These values will come close to the domed roof coefficients of ±0.9.

Table G11-6 Design Pressures for MWFRS for Domed Roof

Surface	*Location (ft)*	C_p	*External pressure (psf)*	*Design pressures (psf)* $(+GC_{pi})$	*Design pressures (psf)* $(-GC_{pi})$
Domed roof Case A	Point A – 0 ft	–0.04	–1.9	–10.6	6.8
	10	–0.23	–11.1	–19.8	–2.4
	20	–0.41	–19.8	–28.5	–11.1
	30	–0.60	–29.0	–37.7	–20.3
	40	–0.78	–37.7	–46.4	–29.0
	Point B – 50 ft	–0.97	–46.5	–55.2	–37.8
	60	–0.82	–39.6	–48.3	–30.9
	70	–0.66	–31.9	–40.6	–23.2
	80	–0.51	–24.6	–33.3	–15.9
	90	–0.35	–16.9	–25.6	–8.2
	Point C – 100 ft	–0.20	–9.7	–18.4	–1.0
Domed roof Case B	Point A – 0 ft	–0.04	–1.9	–10.6	6.8
	θ = 25°; 13.8 ft	–0.04	–1.9	–10.6	6.8
	20	–0.20	–9.7	–18.4	–1.0
	30	–0.46	–22.2	–30.9	–13.5
	40	–0.71	–34.3	–43.0	–25.6
	Point B – 50 ft	–0.97	–46.5	–55.2	–37.8
	60	–0.82	–39.6	–48.3	–30.9
	70	–0.66	–31.9	–40.6	–23.2
	80	–0.51	–24.6	–33.3	–15.9
	90	–0.35	–16.9	–25.6	–8.2
	Point C – 100 ft	–0.20	–9.7	–18.4	–1.0

Fig. G11-3
Component design pressures for domed roof

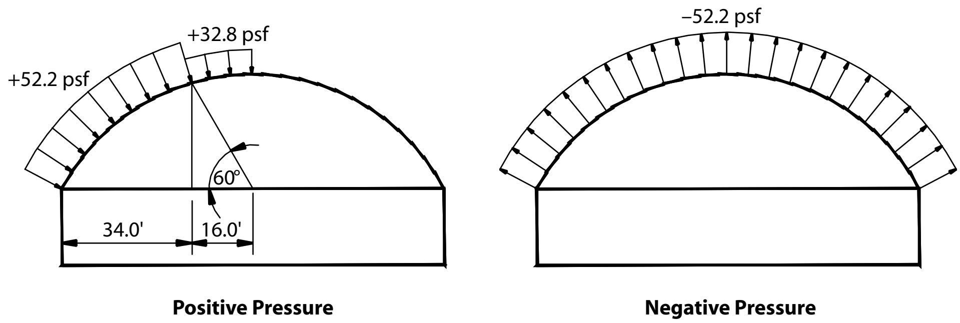


Table G11-7 Roof External Pressure Coefficient for C&C (from Figure 30.4-7 of the Standard)

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

Domed Roof Design Pressures

The C&C domed roof pressure coefficients (**Table G11-7**) are given in Fig. 30.4-7 of the Standard. This figure is valid only for domes of certain geometric parameters. The base height to diameter ratio, $h_D/D = 20/100 = 0.20$, which is in the range of 0 to 0.5 for Fig. 30.4-7. The rise to diameter ratio, $f/D = 30/100 = 0.30$, which is in the range of 0.2 to 0.5 for Fig. 30.4-7. Therefore, it is valid to use Fig. 30.4-7 for this dome.

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

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

$$p = 48.3\ GC_p - 48.3(\pm 0.18) = 48.3\ GC_p \pm 8.7$$

Design pressures are summarized in **Table G11-8**.

These pressures are for the front half of the dome. The back half would experience only the negative value of –52.2 psf. However, since all wind directions must be taken into account, and since each element would at some point be considered to be in the front half of the dome, each element must be designed for both positive and negative values.

Comment

The pressures determined are limit state design pressures for strength design. Section 2.3 of the Standard indicates load factor for wind load to be $1.0W$ for loads determined in this example. If allowable stress design is to be used, the load factor for wind load is $0.6W$ as shown in Section 2.4 of the Standard.

Table G11-8 Roof Design Pressures

	Design pressure (psf)	
Zone	*Positive*	*Negative*
0° to 60°	+52.2	–52.2
60° to 90°	+32.8	–52.2

Chapter 12

Unusually Shaped Building

This example demonstrates calculation of wind loads for an unusually shaped building, as shown in **Fig. G12-1**. Building data are as shown in **Table G12-1**.

12.1 Analytical Procedure

Nonsymmetrical buildings are outside the scope of part 2, chapter 28, Envelope Procedure, of ASCE 7-10. Therefore, Chapter 27, part 1, Directional Procedure, is used. The building is less than 60 ft tall, so it is possible to use low-rise provisions of part 1, Chapter 28. However, because unusually shaped buildings are not specifically covered, the adaptation of the low-rise "pseudo pressure" coefficients to buildings outside the scope of the research is not recommended. Therefore, the Directional Procedure of Section 27.4.1 of the Standard is used.

Building Classification

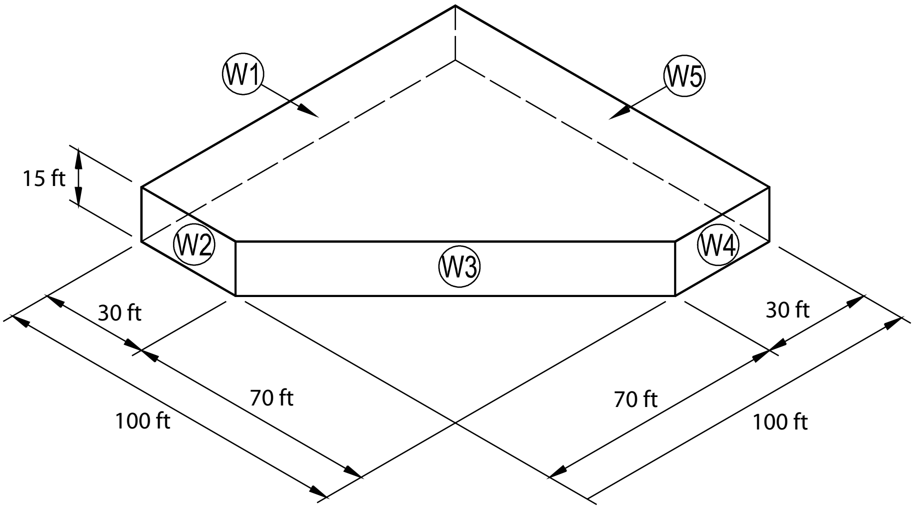
The building is an office building. It is not considered an essential facility, nor is it likely to be occupied by 300 people in a single area at one time. Therefore, building Risk Category II is appropriate (see Table 1.5-1 of the Standard).

Wind speed map for this building risk category is Fig. 26.5-1A of the Standard.

Basic Wind Speed

Selection of the basic wind speed is addressed in Section 26.5.1 of the Standard. San Francisco, California, is located in the 110-mph zone; therefore, the basic wind speed $V = 110$ mph (see Fig. 26.5-1A of the Standard).

Fig. G12-1
Building characteristics for unusually shaped building



Exposure

The building is located in a suburban terrain area; according to Section 26.7 of the Standard, Exposure B is used.

Enclosure

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

Velocity Pressures

The velocity pressures are computed using the following equation:

$$q_z = 0.00256\ K_z K_{zt} K_d V^2 \text{ psf}$$ (Eq. 27.3-1 for MWFRS and Eq. 30.3-1 for C&C)

where

K_z = value obtained from Table 27.3-1 of the Standard for MWFRS and Table 30.3-1 of the Standard for C&C

Table G12-1 Data for Unusually Shaped Building

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

K_{zt} = 1.0 for homogeneous topography
K_d = 0.85 for buildings (see Table 26.6-1 of the Standard)
V = 110 mph

$$q_z = 0.00256\ K_z(1.0)(0.85)(110)^2 = 26.3\ K_z \text{ psf}$$

The mean roof height for a flat roof is the eave height h = 15 ft. Values for K_z and the resulting velocity pressures for MWFRS and C&C are shown in **Table G12-2**.

Gust Effect Factor

Section 26.9.2 of the Standard permits consideration of the building as rigid if the building fits in the definition of a low-rise building. The example building is a low-rise building, so it can be considered rigid and the value of the gust effect factor of 0.85 is used.

External Pressure Coefficients for MWFRS

The values for the external pressure coefficients for the various surfaces are obtained from Fig. 27.4-1 of the Standard for each of the surfaces of the building shown in **Fig. G12-1** of this guide. The determination of certain pressure coefficients is based on aspect ratios. The overall dimensions for L and B are used.

L/B = 100/100 = 1.00
h/L = 15/100 = 0.15
$\theta = 0°$

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

The roof pressure coefficient C_p comes from the "wind parallel to a ridge" portion of Fig. 27.4-1 of the Standard. For these flat roofs, C_p varies with h/L and with distance from the leading edge of the roof. For h/L = 0.15 ≤ 0.5, C_p = –0.9, –0.5, or –0.3, depending on the distance from the leading edge. Figure 27.4-1 also includes the –0.18 case for all roofs; however, this case causes critical loading when combined with transient loads such as snow load or live load. For brevity, the case is not shown.

External pressure coefficients are summarized in **Tables G12-3 through G12-6**.

Table G12-2 Velocity Pressures for Unusually Shaped Building

	MWFRS		C&C	
Height (ft)	K_z	q_z *(psf)*	K_z	q_z *(psf)*
0 to 15	0.57	15.0	0.70	18.4
Eave height = 15	0.57	15.0	0.70	18.4

Table G12-3 External Pressure Coefficients for Wind Normal to Wall W1

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

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

Table G12-4 External Pressure Coefficients for Wind Normal to Wall W5

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

Table G12-5 External Pressure Coefficients for Wind Normal to Wall W4

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

Table G12-6 External Pressure Coefficients for Wind Normal to Wall W2

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

Design Wind Pressures for the MWFRS

The design pressures for this building are obtained by the equation

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

where

$q = q_z = 15.0$ psf for windward wall at height $z = 15$ ft and below
$q = q_h = 15.0$ psf for leeward wall, side walls, and roof
$q_i = q_h = 15.0$ psf for all surfaces since the building is enclosed
$G = 0.85$, the gust effect factor for rigid buildings and structures
C_p = external pressure coefficient for each surface, as shown in **Fig. 12-1** of this guide
$(GC_{pi}) = \pm 0.18$, the internal pressure coefficient for enclosed buildings

For windward walls

$$p = q_zGC_p - q_h(GC_{pi}) = 15.0(0.85)C_p - 15.0(\pm 0.18) = 12.8C_p \pm 2.7$$

For all other surfaces

$$p = q_hGC_p - q_h(GC_{pi}) = 15.0(0.85)C_p - 15.0(\pm 0.18) = 12.8C_p \pm 2.7$$

Design pressures are summarized in **Tables G12-7 through G12-10**. The external roof pressures and their prescribed zones are shown in **Fig. G12-2.**

Minimum Design Wind Pressures

Section 27.4.7 of the Standard requires that the MWFRS be designed for not less than 16 psf applied to the projection of the building in each orthogonal direction on a vertical plane. This is checked as a separate load case. The application of this load is shown in **Fig. G12-3**.

Table G12-7 Design Pressures for Wind Normal to Wall W1

Surface type	*Surface designation*	z *or* x *(ft)*	q *(psf)*	C_p	*External Pressure (psf)*	*Design pressures (psf)* ($+GC_{pi}$)	($-GC_{pi}$)
Walls	W1	0 to 15	15.0	+0.80	+10.2	+7.5	+12.9
	W3, W4	0 to 15	15.0	−0.50	−6.4	−9.1	−3.7
	W2, W5	0 to 15	15.0	−0.70	−8.9	−11.6	−6.2
Roof		0 to 15	15.0	−0.90	−11.5	−14.2	−8.8
		15 to 30	15.0	−0.50	−6.4	−9.1	−3.7
		30 to 100	15.0	−0.30	−3.8	−6.5	−1.1

Note: $q_h = 15.0$ psf; $G = 0.85$.

Table G12-8 Design Pressures for Wind Normal to Wall W5

Surface type	*Surface designation*	z *or* x *(ft)*	q *(psf)*	C_p	*External pressure (psf)*	*Design pressures (psf)* ($+GC_{pi}$)	($-GC_{pi}$)
Walls	W5	0 to 15	15.0	+0.80	+10.2	+7.5	+12.9
	W2, W3	0 to 15	15.0	−0.50	−6.4	−9.1	−3.7
	W1, W4	0 to 15	15.0	−0.70	−8.9	−11.6	−6.2
Roof		0 to 15	15.0	−0.90	−11.5	−14.2	−8.8
		15 to 30	15.0	−0.50	−6.4	−9.1	−3.7
		30 to 100	15.0	−0.30	−3.8	−6.5	−1.1

Note: $q_h = 15.0$ psf; $G = 0.85$.

Table G12-9 Design Pressures for Wind Normal to Wall W4

Surface type	*Surface designation*	z *or* x *(ft)*	q *(psf)*	C_p	*External pressure (psf)*	*Design pressures (psf)* ($+GC_{pi}$)	($-GC_{pi}$)
Walls	W4, W3	0 to 15	15.0	+0.80	+10.2	+7.5	+12.9
	W1	0 to 15	15.0	−0.50	−6.4	−9.1	−3.7
	W2, W5	0 to 15	15.0	−0.70	−8.9	−11.6	−6.2
Roof		0 to 15	15.0	−0.90	−11.5	−14.2	−8.8
		15 to 30	15.0	−0.50	−6.4	−9.1	−3.7
		30 to 100	15.0	−0.30	−3.8	−6.5	−1.1

Note: $q_h = 15.0$ psf; $G = 0.85$.

Table G12-10 Design Pressures for Wind Normal to Wall W2

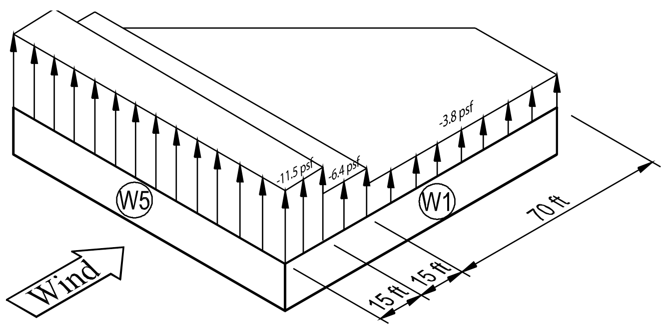
Surface type	*Surface designation*	*z or x (ft)*	*q (psf)*	C_p	*External pressure (psf)*	*Design pressures (psf)* $(+GC_{pi})$	$(-GC_{pi})$
Walls	W2, W3	0 to 15	15.0	+0.80	+10.2	+7.5	+12.9
	W5	0 to 15	15.0	–0.50	–6.4	–9.1	–3.7
	W1, W4	0 to 15	15.0	–0.70	–8.9	–11.6	–6.2
Roof		0 to 15	15.0	–0.90	–11.5	–14.2	–8.8
		15 to 30	15.0	–0.50	–6.4	–9.1	–3.7
		30 to 100	15.0	–0.30	–3.8	–6.5	–1.1

Note: $q_h = 15.0$ psf; $G = 0.85$.

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. Several exceptions are noted that require only the use of Load Case 1, the full orthogonal wind case, and Load Case 3, the diagonal wind case approximated by applying 75 percent of the loads to adjacent faces simultaneously. The exceptions are building types that meet the requirements of Section D1.1 of Appendix D.

Fig. G12-2
External roof pressure zones for MWFRS



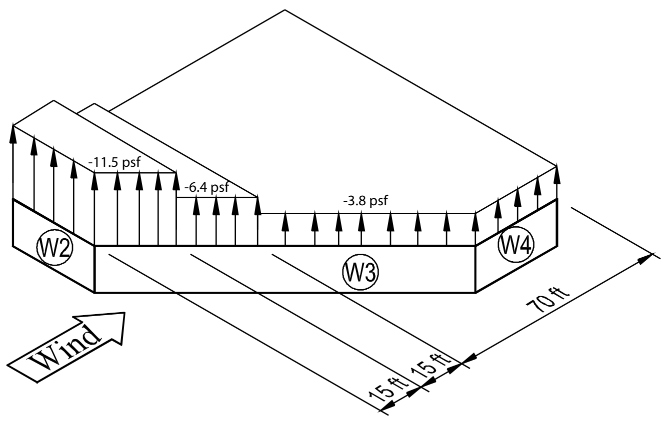
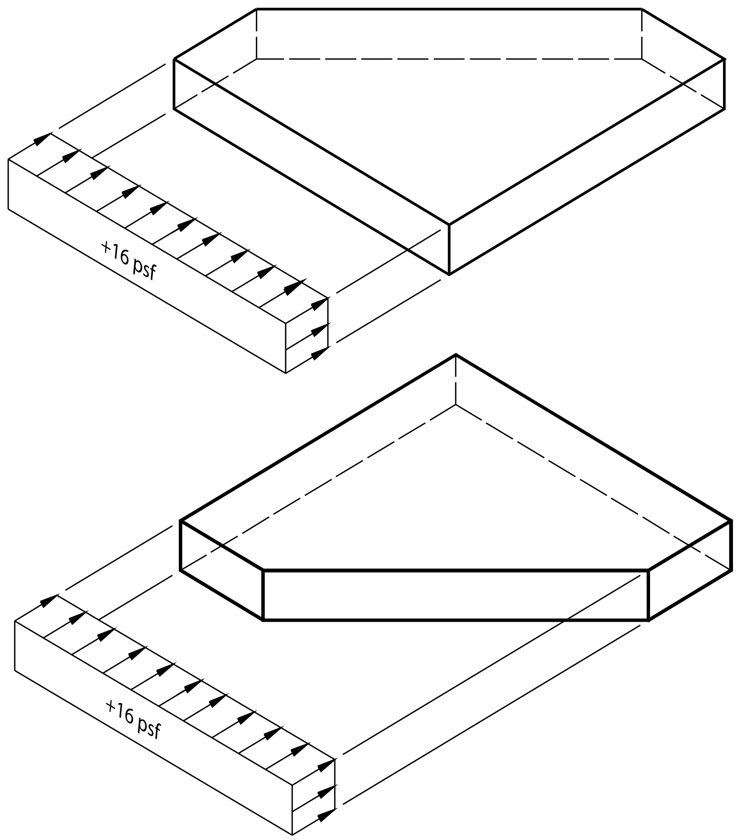


Fig. G12-3
Application of 16 psf minimum load case



This exception is for one-story buildings less than 30 ft in height, so this example meets that exception and is required only to meet Load Cases 1 and 3. Load Case 1 is calculated above and shown applied in each orthogonal direction in **Fig. G12-4**. Load Case 3 is the diagonal wind load case, applied in each of four directions as shown in **Fig. G12-5**.

Design Pressures for Components and Cladding

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

$$p = q_h[(GC_p) - (GC_{pi})] \qquad \text{(Eq. 30.4-1)}$$

where

q_h = 18.4 psf from **Table G12-2**
(GC_p) = external pressure coefficient (see Figs. 30.4-1 and 30.4-2 of the Standard)
(GC_{pi}) = ±0.18, the internal pressure coefficient for enclosed buildings

Wall Design Pressures

The pressure coefficients are a function of effective wind area. Since specific components of the walls are not identified, design pressures are given for various effective wind areas in **Table G12-11**. These values have been reduced

Fig. G12-4
Application of Load Case 1 from each orthogonal direction

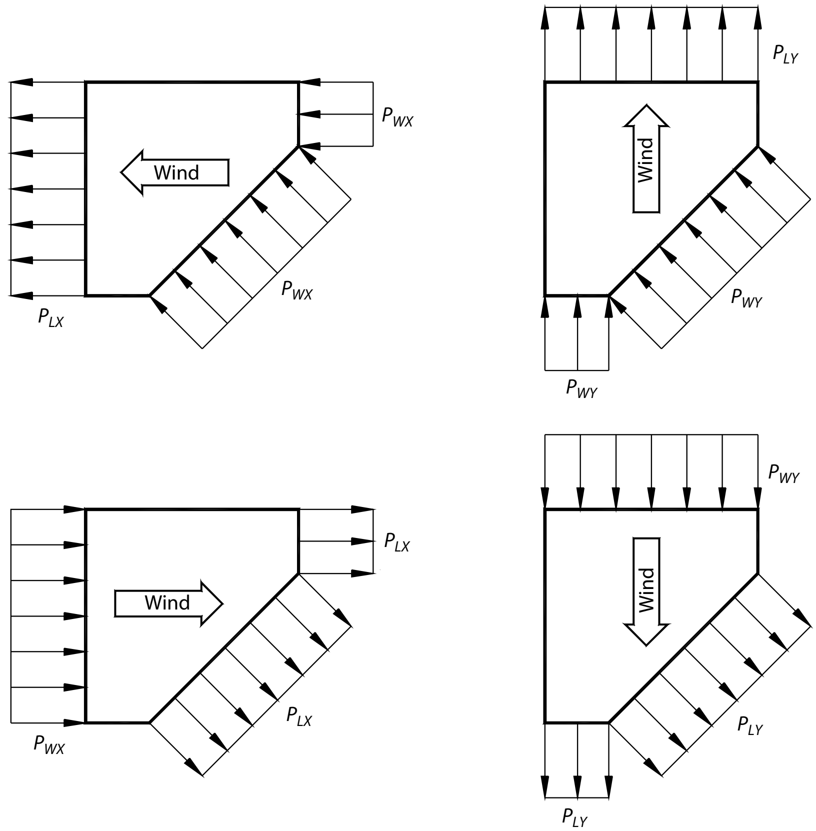


Fig. G12-5
Application of Load Case 3 from each diagonal direction

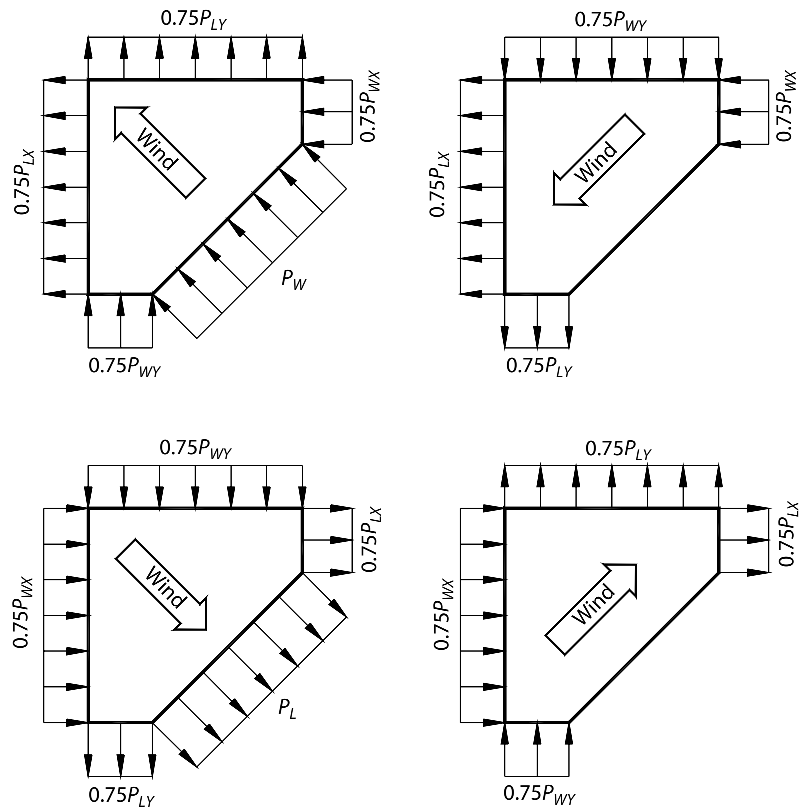


Table G12-11 Wall Component Pressure Coefficients by Zone, from Figure 30.4-1

A *(ft²)*	*Zones 4 and 5* $(+GC_p)$	*Zone 4* $(-GC_p)$	*Zone 5* $(-GC_p)$
≤10	+0.90	−0.99	−1.26
50	+0.79	−0.88	−1.04
100	+0.74	−0.83	−0.95
>500	+0.63	−0.72	−0.72

Note: GC_p values have been reduced by 10 percent since $\theta \le 10°$.

by 10 percent as allowed by Note 5 in Fig. 30.4-1 of the Standard for roof angle $\theta \le 10°$.

Width edge zone

smaller of	$a = 0.1(100) = 10$ ft
or	$a = 0.4(15) = 6.0$ ft (controls)
but not less than	$a = 0.04(100) = 4.0$ ft
or	$a = 3$ ft

The design pressures are the algebraic sum of external and internal pressures. Controlling negative pressure is obtained with positive internal pressure, and controlling positive pressure is obtained with negative internal pressure. The controlling design pressures are given in **Table G12-12**. The edge zones for the walls are arranged at exterior corners, as shown in **Fig. G12-6**.

Roof Design Pressures

The pressure coefficients are a function of effective wind area. Since specific components of the roof are not identified, design pressures are given for various effective wind areas in **Table G12-13**. The design pressures in **Table G12-14** are the algebraic sum of external and internal pressures. Controlling negative pressure is obtained with positive internal pressure, and controlling positive pressure is obtained with negative internal pressure. The edge zones for the roof are arranged as shown in **Fig. G12-7**.

Table G12-12 Controlling Design Pressures for Wall Components by Zone (psf)

	Zone 4		*Zone 5*	
Area	*Positive*	*Negative*	*Positive*	*Negative*
≤10	+19.9	−21.5	+19.9	−26.5
50	+17.8	−19.5	+17.8	−22.4
100	+16.9	−18.6	+16.9	−20.8
>500	+14.9*	−16.6	+14.9*	−16.6

Note: Design pressures include internal pressure of 3.3 psf.

*Section 30.2.2 of the Standard requires that C&C pressures be not less than ±16 psf.

Fig. G12-6
Wall pressure zones for components and cladding

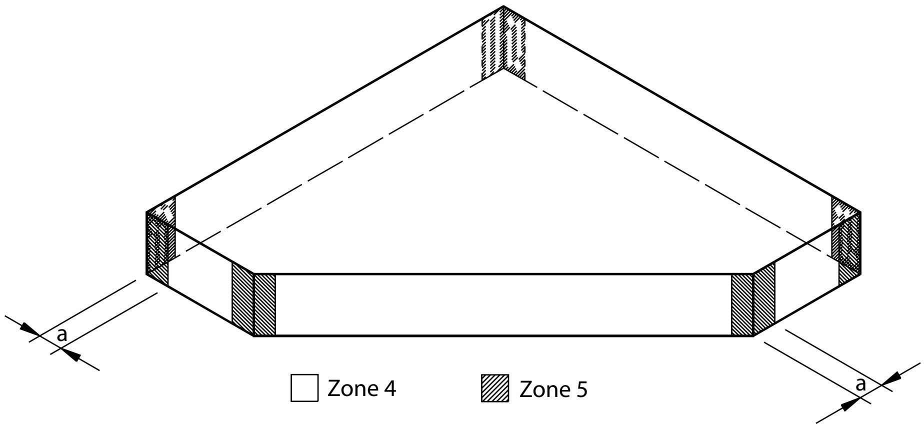


Table G12-13 Roof External Pressure Coefficients by Zone, from Figure 30.4-2A

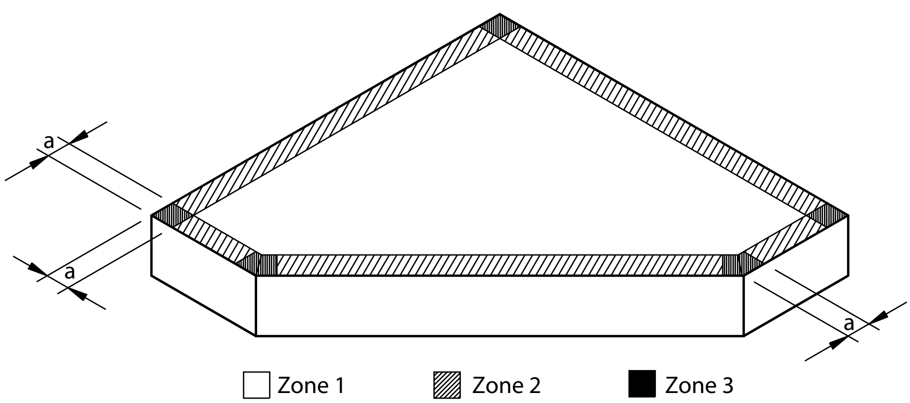
	Positive	*Negative*		
	Zones 1, 2, 3	*Zone 1*	*Zone 2*	*Zone 3*
A *(ft²)*	GC_p	GC_p	$-GC_p$	$-GC_p$
10	+0.30	–1.00	–1.80	–2.80
50	+0.23	–0.93	–1.31	–1.61
100	+0.20	–0.90	–1.10	–1.10

Table G12-14 Roof Design Pressures by Zone (psf)

	Zones 1, 2, 3	*Zone 1*	*Zone 2*	*Zone 3*
Area	*Positive*	*Negative*	*Negative*	*Negative*
10	+8.8*	–21.7	–36.4	–54.8
50	+7.5*	–20.4	–27.4	–32.9
100	+7.0*	–19.9	–23.6	–23.6

*Section 30.2.2 of the Standard requires that C&C pressures be not less than ±10 psf.

Fig. G12-7
Roof pressure zones for components and cladding



Comment

The pressures determined are limit state design pressures for strength design. Section 2.3 of the Standard indicates the load factor for wind load to be $1.0W$ for loads determined in this example. If allowable stress design is to be used, the load factor for wind load is $0.6W$ as shown in Section 2.4 of the Standard.

Chapter 13

Billboard Sign on Flexible Poles

In this example, design wind forces are determined for a tall billboard solid sign 60 ft above the ground. The example illustrates, first, determination of G_f for a flexible structure and, second, use of the provisions of Section 29.4.1 of the Standard for solid signs and provisions of Section 29.5 of the Standard for other structures. The dimensions of the billboard sign are shown in **Fig. G13-1**. The billboard sign data are as listed in **Table G13-1**.

13.1 Analytical Procedure

Wind load provisions for MWFRS of other structures and building appurtenances are given in chapter 29 of the Standard. Section 29.4.1 provides requirements for freestanding signs, and Section 29.5 is used to determine wind loads on the pipe columns.

Chapter 30 of the Standard does not have requirements for C&C for sign structures.

Building Classification

Failure of the sign represents low hazard to human life since it is located away from the highway and is not in a populated area. The structure can be classified as Category I (see Table 1.5-1 of the Standard). The wind speed map associated with Category I structures is Fig. 26.5-1C of the Standard.

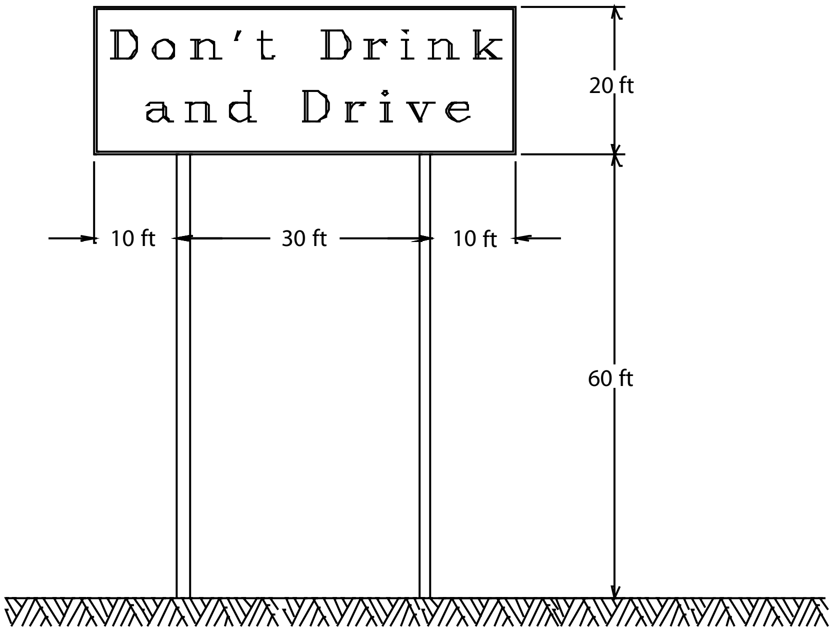
Basic Wind Speed

The wind speed map (Fig. 26.5-1C of the Standard) has only one value of wind speed in the middle of the country. Exact location of the sign in Iowa is not important. The basic wind speed $V = 105$ mph.

Exposure

The sign is located in an open area. It does not fit Exposures B or D; therefore, Exposure C is used (see Section 26.7 of the Standard).

Fig. G13-1
Characteristics for billboard sign on flexible poles



Velocity Pressures

The velocity pressures are computed using the following equation:

$$q_z = 0.00256\ K_z K_{zt} K_d V^2 \text{ psf} \qquad \text{(Eq. 29.3-1)}$$

where

V = 105 mph
K_{zt} = 1.0 because of flat terrain
K_d = 0.85 for solid sign (see Table 26.6-1 of the Standard; note that for pipe columns K_d could be 0.95 assuming similar value as chimney. However, since wind loads on pipe columns are likely to be small as compared to sign, value of 0.85 is used)
K_z = values from Table 29.3-1 of the Standard for z of 30, 60, and 80 ft. More divisions of z are not justified, because loads on pipe supports are small compared to the ones on the sign.

Table G13-1 Data for Billboard Sign on Flexible Poles

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

Values for K_z and the resulting velocity pressures are given in **Table G13-2**.

Design Force for the Structure

The design force for the Solid Sign (Section 29.4.1 of the Standard) is given by

$$F = q_h G C_f A_s \qquad \text{(Eq. 29.4-1)}$$

The design force for the support poles (Section 29.5 of the Standard) is given by

$$F = q_z G C_f A_f \qquad \text{(Eq. 29.5-1)}$$

where

q_z = value as determined previously
q_h = value determined at the top of the sign (see Fig. 29.4-1 of the Standard)
G = gust effect factor to be calculated by Eq. 26.9-10 because $f < 1$ Hz.
A_s = gross area of solid sign; $50 \times 20 = 1{,}000$ ft^2
A_f = area projected normal to wind; 1.33 sq ft/ft of support height
C_f = force coefficient values from Figs. 29.4-1 and 29.5-1 of the Standard

Gust Effect Factor

Section 26.9.2 of the Standard requires determination of fundamental natural frequency to assess whether a structure should be considered flexible or rigid. Determination of approximate frequency given in Section 26.9.3 is for buildings. Frequency for the sign can be determined using structural properties and deformational characteristics of the sign. For simplicity, a value of 0.7 Hz for this sign structure is assumed.

Since fundamental frequency is less than 1 Hz, the sign structure is considered flexible (Section 26.2 of the Standard).

For a flexible structure, the gust effect factor, G_f, is determined from Eq. 26.9-10 of the Standard:

$$G = 0.925\left[\frac{1+1.7I_{\bar{z}}\sqrt{g_Q^2Q^2+g_R^2R^2}}{1+1.7g_V I_{\bar{z}}}\right] \qquad \text{(Eq. 26.9-10)}$$

where

$I_{\bar{z}}$ = value from Eq. 26.9-7 of the Standard
g_Q, g_v = value taken as 3.4 (see Section 26.9.3 of the Standard)

Table G13-2 Velocity Pressures

Height (ft)	K_z	q_z *(psf)*
30	0.98	23.5
60	1.13	27.1
80	1.21	29.0

g_R = value from Eq. 26.9-11 of the Standard
Q = value determined from Eq. 26.9-8 of the Standard
R = value determined from Eq. 26.9-12 of the Standard
$\bar{z}$ = equivalent height of the structure, it is used to determine nominal value of $I_{\bar{z}}$; for buildings, the recommended value is $0.6h$, but for the sign, middle of the billboard area or 70 ft is logical
$c, l, \bar{\epsilon}$ = value given in Table 26.9-1 of the Standard

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.2\left(\frac{33}{70}\right)^{1/6} = 0.176 \qquad \text{(Eq. 26.9-7)}$$

$$L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500\left(\frac{70}{33}\right)^{1/5} = 581 \text{ ft} \qquad \text{(Eq. 26.9-9)}$$

$$Q^2 = \frac{1}{1+0.63\left[\dfrac{B+h}{L_{\bar{z}}}\right]^{0.63}} \qquad \text{(Eq. 26.9-8)}$$

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

Note: In Eq. 26.9-8, B and h are the dimensions of the sign.

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 0.65\left(\frac{70}{33}\right)^{1/6.5}(105)\left(\frac{88}{60}\right) = 112.4 \qquad \text{(Eq. 26.9-16)}$$

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

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(0.7)(581)}{112.4} = 3.62 \qquad \text{(Eq. 26.9-14)}$$

Note: n_1 is the fundamental frequency of the structure.

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = 0.0621 \qquad \text{(Eq. 26.9-13)}$$

For R_h,

$$\eta = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{(4.6)(0.7)(80)}{112.4} = 2.292$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) = 0.3421 \qquad \text{(Eq. 26.9-15a)}$$

Note: h is taken as 80 ft because resonance response depends on full height. For R_B (assuming $B = 50$ ft),

$$\eta = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}} = \frac{(4.6)(0.7)(50)}{112.4} = 1.4324 \qquad \text{(Eq. 26.9-15a)}$$

$$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta}) = 0.4683$$

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

$$\eta = \frac{15.4\, n_1 L}{\bar{V}_{\bar{z}}} = \frac{(15.4)(0.7)(2)}{112.4} = 0.1918 \qquad \text{(Eq. 26.9-15a)}$$

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) = 0.8835$$

$$\begin{aligned} g_R &= \sqrt{2\ln(3{,}600 n_1)} + \frac{0.577}{\sqrt{2\ln(3{,}600 n_1)}} \qquad \text{(Eq. 26.9-11)} \\ &= 4.10 \end{aligned}$$

$$\begin{aligned} R^2 &= \frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L) \\ &= \frac{1}{0.01}(0.0621)(0.3421)(0.4683)[0.53 + (0.47)(0.8835)] \qquad \text{(Eq. 26.9-12)} \\ &= 0.94 \end{aligned}$$

$$\begin{aligned} G_f &= 0.925\left[\frac{1 + 1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right] \\ &= 0.925\left[\frac{1 + 1.7(0.176)\sqrt{(3.4)^2(0.858) + (4.1)^2(0.94)}}{1 + 1.7(3.4)(0.176)}\right] \qquad \text{(Eq. 26.9-10)} \\ &= 1.15 \end{aligned}$$

Note: If the structure was considered nonflexible or rigid, value of $G = 0.85$ would be acceptable by the standard (Section 26.9). Using the strength design wind speed from the map Fig. 26.5-1C of the Standard increases the G_f from 1.09 (Mehta and Coulbourne, 2010) to 1.15. The designer is cautioned that the wind speed change in ASCE 7-10 from 25-year to 300-year recurrence interval speeds (in this case of a Category I structure) affects the gust effect factor.

Force Coefficient for Supports

The supports are round. From Fig. 29.5-1 of the Standard:

$$D\sqrt{q_z} = 1.33\sqrt{27.1} = 6.9 > 2.5 \text{ and}$$

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

For moderately smooth surface and $h/D = 45$,

$$C_f = 0.7 \text{ (Fig. 29.5-1 of the Standard)}$$

Design Force on Supports

Force, $F = q_z G_f C_f A_f$:

For one pole

0 to 30 ft	$F = 23.5(1.15)(0.7)(1.33) = 25.2$ plf
30 to 60 ft	$F = 27.1(1.15)(0.7)(1.33) = 29.0$ plf

Total for two poles

0 to 30 ft	$F = 50.4$ plf
30 to 60 ft	$F = 58.0$ plf

Force Coefficients for Solid Sign

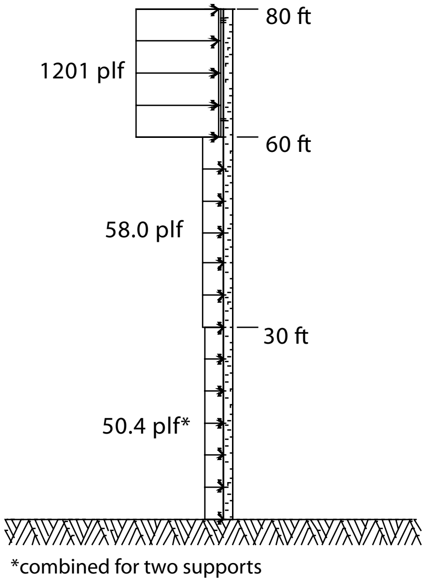
$B/s = 50/20 = 2.5$ (aspect ratio in Fig. 29.4-1 in the Standard)
$s/h = 20/80 = 0.25$ (clearance ratio in Fig. 29.4-1 in the Standard)
$C_f = 1.8$ for Case A and Case B (interpolation in Fig. 29.4-1 in the Standard)
$C_f = 2.4$ (0 to 20 ft); 1.6 (20 to 40 ft); 0.6 (40 to 50 ft) for Case C (linear interpolation)

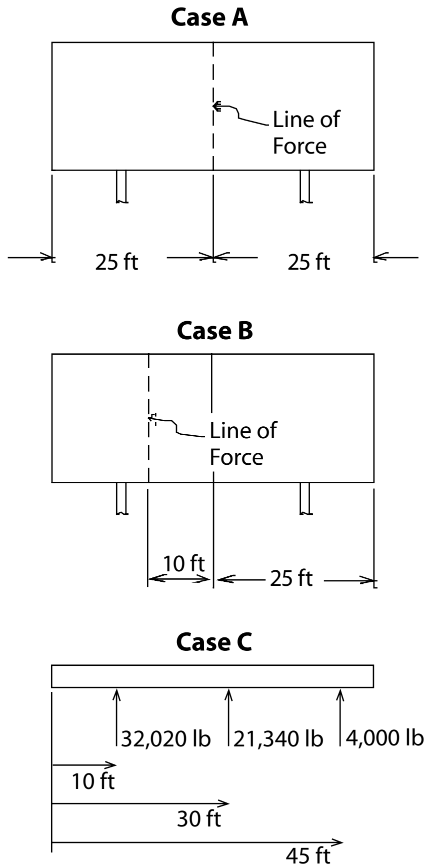
Design Force on Sign

Design force for the billboard structure follows three cases shown in **Fig. G13-2**. See Fig. 29.4-1 of the Standard for a description of the cases.

Case A: $F = 29.0\ (1.15)\ (1.8)\ (1{,}000) = 60{,}030$ lb (or 1,201 plf along the width of sign)
Case B: eccentricity $e = 0.2\ (50) = 10$ ft
force of 60,030 lb acts at 10 ft from the center of the sign
Case C: $F = 29.0\ (1.15)\ (2.4)\ (400) = 32{,}020$ lb acts at 10 ft from an edge of sign
$F = 29.0\ (1.15)\ (1.6)\ (400) = 21{,}340$ lb acts at 30 ft from the edge of sign

Fig. G13-2
Design forces for the billboard sign





$F = 29.0\ (1.15)\ (0.6)\ (200) = 4{,}000$ lb acts at 45 ft from the edge of sign

Note: Total force for Case A and C are comparable; 60,030 vs 57,360 lb. Eccentricity for Case B is higher than Case C.

Comment

The pressures determined are limit state design pressures for strength design. Section 2.3 of the Standard indicates the load factor for wind load to be $1.0W$ for loads determined in this example. If allowable stress design is to be used, the load factor for wind load is $0.6W$ as shown in Section 2.4 of the Standard.

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Chapter 14

Frequently Asked Questions

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

14.1 Wind Speed

1. Is it possible to obtain larger scale maps of basic wind speeds (see Figs. 26.5-1A, 26.5-1B, 26.5-1C) so that the locations of the wind speed contours can be determined with greater accuracy?

No. The wind speed contours in the hurricane prone region of the United States are based on hurricane wind speeds from Monte Carlo simulations and on estimates of the rate at which hurricane wind speeds attenuate to 115 mph in Fig. 26.5-1A following landfall. Because the wind speed contours of these figures represent a consensus of the ASCE 7 Wind Load Task Committee, increasing the map scale would do nothing to improve their accuracy. Some jurisdictions establish a specific basic wind speed for a jurisdictional area; it is advisable to check with authorities of that area. There are websites and other electronic tools developing that can help more accurately determine a site-specific wind speed. One such website is www.atcouncil.org/windspeed.

2. Wind speeds in the maps of ASCE 7-10 are much higher than the ones in maps of ASCE 7-05; does this mean the design for wind will be much stronger and cost more?

No; the wind speeds in the maps of ASCE 7-10 are related to ultimate loads and strength design. The load factor for wind in Section 2.3.2 of the Standard is 1.0*W* reflecting limit state design. The load factor for wind for allowable stress design in Section 2.3.3 is 0.6*W*. The final wind load design using the wind speeds of ASCE 7-10 will be approximately the same as before, provided everything else remains the same.

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

Basic wind speed in IBC wind speed map or ASCE 7-10 is defined as 3-s gust wind speed at 33 ft above ground for Exposure Category C, which is the standard measurement. The velocity pressure exposure coefficient, K_z, adjusts the wind speed for exposure and height above ground. However, for simplicity the coefficient is applied in the pressure equation, thus adjusting pressure rather than wind speed. Use of K_z adjusts the pressures from Exposure C to Exposure B.

4. If the design wind loads are to be determined for a building that is located in a special wind region (shaded areas) in Figs. 26.5-1A, 26.5-1B, and 26.5-1C, what basic wind speed should be used?

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

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

The Saffir–Simpson Hurricane Scale classifies hurricanes based on intensity and damage potential using five categories (1 through 5, with 5 being the most intense). Table C26.5-2 in the Commentary of the Standard gives an approximate relationship between basic wind speed in ASCE 7 and the Saffir–Simpson hurricane scale. The Saffir–Simpson scale has a range of wind speeds. A decision will have to be made on which specific basic wind speed over land to use.

14.2 Load Factor

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

When using the loads or load combinations specified in ASCE 7-10, no increase in allowable stress is permitted except when the increase is justified by the rate of duration of load (such as duration factors used in wood design). Instead, load combination No. 6 in Section 2.4.1 of ASCE 7-10 adjusts for the case when wind load and another transient load are combined. This load combination applies a 0.75 factor to the transient loads *only* (not to the dead load). The 0.75 factor applied to the transient loads accounts for the fact that it is extremely unlikely that two maximum events will happen at the same time. Please note that the load factor for wind in Allowable Stress Design (ASD) is $0.6W$ since the Strength Design load factor is $1.0W$.

7. Why can the wind directionality factor, K_d, only be used with the load combinations specified in Sections 2.3 and 2.4 of ASCE 7-10?

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

14.3 Terrain Exposure

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

Section 26.7 of ASCE 7-10 provides general definitions of Exposures B, C, and D; however, the designer must refer to the Commentary for a detailed explanation for each exposure. The exposure depends on the size of the parking lot, its size relative to the building, and the number and type of obstructions in the area. Section C26.7 of the Commentary includes a formula (Eq. C26.7-1) that will help the designer determine if the terrain roughness is sufficient to be categorized as Exposure B. Note that for Exposure B, the fetch distance is 2,600 ft or 20 times the structure's height, whichever is greater. Also note that the Commentary provides suggestions for determining the "upwind fetch surface area."

For clearings such as parking lots, wide roads, road intersections, underdeveloped lots, and tree clearings, the Commentary provides a rational procedure and an example to interpolate between Exposure B and C; the designer is encouraged to use this procedure. The procedure is illustrated in Figs. C26.7-3 and C26.7-4. The determination of exposure category is based on the extent and the distance from the project location of "open patches" where the exposure could be considered to be less rough than Exposure B.

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

Section 26.8 of the Standard requires the calculation of the topographic factor, K_{zt}, for buildings and other structures sited on the upper half of isolated hills or escarpments located in Exposures B, C, or D where the upwind terrain is free of such topographic features for a distance of at least 100 H or 2 mi, whichever is smaller, as measured from the crest of the topographic feature. K_{zt} need not be calculated when the height, H, is less than 15 ft in Exposures D and C, or less than 60 ft in Exposure B. In addition, K_{zt} need not be calculated when H/L_h is less than 0.2. H and L_h are defined in Fig. 26.8-1. The value of K_{zt} is never less than 1.0.

14.4 MWFRS and C&C

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

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

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

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

In the case where the tributary area on any member exceeds 700 ft^2, Section 30.2.4 permits the member to be considered an MWFRS for determination of wind load. Even when considered an MWFRS under this provision, the top chord members of a gable truss would have to follow rules of C&C if they receive load directly from the roof sheathing.

14.5 Gust Effect Factor

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

No. Definition of a flexible building given in Section 26.2 states that fundamental (first mode) natural frequency less than 1 Hz would make it flexible for the gust effect factor. The energy in the turbulence spectrum is small for frequencies above 1 Hz. Hence, a tower with fundamental frequency of 2 Hz is not likely to be dynamically excited by wind. The Commentary of the Standard has a good discussion on response of buildings and structures in turbulent wind in Section C26.9.

14.6 Pressure Coefficient

13. In the design of main wind force-resisting systems (MWFRS), the provisions of Fig. 27.4-1 apply to enclosed or partially enclosed buildings of all heights. The provisions of Fig. 28.4-1 apply to enclosed or partially enclosed buildings with mean roof height less than or equal to 60 ft. Does this mean that either figure may be used for the design of a low-rise MWFRS?

Figure 27.4-1 may be used for buildings of any height, whereas Fig. 28.4-1 applies only to low-rise buildings. Section 26.2 defines low-rise buildings to comply with mean roof height $h \leq 60$ ft and h not to exceed the least horizontal dimension. Pressure coefficients for low-rise buildings given in Fig. 28.4-1 represent "pseudo" loading conditions enveloping internal structural reactions of total uplift, total horizontal shear, and bending moment, among others (see Section C28.3.2). Thus, they are not real wind-induced loads. These loads work adequately for buildings of the shapes shown in Fig. 28.4-1; they become uncertain when extrapolated to other shapes such as a U-shaped building in **chapter 9** or an odd-shaped building of **chapter 12**.

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

Sections 27.4.4 and 30.10 specify pressure coefficients to be used for roof overhangs to determine loads for MWFRS and C&C, respectively. No specific guidance is given for balconies, but use of the loading criteria for roof overhangs should be adequate.

Other loading conditions at the overhang, such as the pressures on the upper roof surface must be considered to obtain the total loads for connections between the roof and the wall.

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

Section 30.6 permits use of pressure coefficients of Figs. 30.4-1 through 30.4-6 provided the mean roof height $h < 90$ ft, the height-to-width ratio is 1 or less, and Eq. 30.6-1 is used.

Note 6 of Fig. 30.6-1 permits use of coefficients of Fig. 30.4-2A, B, and C when the roof angle $0 > 10°$.

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

Roof trusses are classified as C&C since they receive wind load directly from the cladding (roof sheathing). In this case, the effective wind area is the span length multiplied by an effective width that need not be less than one-third the span length or (30)(30/3) = 300 ft^2 (see definition in Section 26.2). This is the area on which the selection of GC_p should be based. Note, however, that the resulting wind pressure acts on the tributary area of each truss, which is (30)(4) = 120 ft^2.

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

According to the definition of Effective Wind Area in Section 26.2 the effective wind area is (70)(70/3) = 1,633 ft^2. The tributary area of the truss is (70)(8) = 560 ft^2, which is less than the 700-ft^2 area required by Section 30.2.4 to qualify for design of the truss using the rules for MWFRS. The truss is to be designed using the rules for C&C, and the wind pressure corresponding to an effective wind area of 1,633 ft^2 is to be applied to the tributary area of 560 ft^2.

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

No; although the length of a decking panel is 20 ft, the basic span is 5 ft. According to the definition of effective wind area, this area is the span length multiplied by an effective width that need not be less than one-third the span length. This gives a minimum effective wind area of (5)(5/3) = 8.3 ft^2. However, the actual width of a panel is 2 ft, making the effective wind area equal to the tributary area of a single panel, or (5)(2) = 10 ft^2. Therefore, GC_p would be determined on the basis of 10 ft^2 of effective wind area, and the corresponding wind load would be applied to a tributary area of 10 ft^2. Note that GC_p is constant for effective wind areas less than 10 ft^2.

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

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

20. If the pressure or force coefficients for various roof shapes (e.g., a curved canopy) are not given in ASCE 7-10, how can the appropriate wind forces be determined for these shapes?

With the exception of pressure or force coefficients for certain shapes, parameters such as V, K_z, K_{zt}, and G are given in ASCE 7-10. It is possible to use pressure or force coefficients from the published literature (see **Section 1.4** of this guide) provided these coefficients are used with care. Mean pressure or force coefficients from other sources can be used to determine wind loads for MWFRS. However, it should be recognized that these coefficients might have been obtained in wind tunnels that have smooth, uniform flows as opposed to more proper turbulent boundary-layer flows. Pressure coefficients for components and cladding obtained from the literature should be adjusted to the 3-s gust speed, which is the basic wind speed adopted by ASCE 7-10.

21. Can the pressure/force coefficients given in ASCE 7-10 be used with the provisions of ASCE 7-88, 7-93, 7-95, 7-98, or 7-02?

Yes, in a limited way. ASCE 7-88 (and 7-93) used the fastest-mile wind speed as the basic wind speed. With the adoption of the 3-s gust speed starting with ASCE 7-95, the values of certain parameters used in the determination of wind forces have been changed accordingly. The provisions of ASCE 7-88 and 7-10 should not be interchanged. Coefficients in ASCE 7-95, 7-98, 7-02, 7-05, and 7-10 are consistent; they are related to 3-s gust speed.

22. What is the difference between a partially enclosed and enclosed building, and how are they determined?

The definition of these two enclosure categories is in Section 26.2. There is a specific definition for both partially enclosed and open buildings; an enclosed building is one that does not comply with the requirements for either open or partially enclosed; "enclosed" is a default designation The determination of a partially enclosed condition is a function of the area of a windward wall dominant opening compared to the area of openings in the remainder of the building envelope.

It is possible to have a building be classified as an enclosed building even when there are large openings in two or more walls and when it does not fit the definition of a partially enclosed building.

The significance in the determination of wind pressure is that partially enclosed buildings have an internal pressure coefficient $GCp_i = \pm 0.55$ and enclosed buildings have $GCp_i = \pm 0.18$.

14.7 Force Coefficient

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

An open building is a structure in which each wall is at least 80 percent open (see Section 26.2). Yes, this three-story frame would be classified as an open building, or as an "other" structure. In calculating the wind force, F, appropriate values of C_f and A_f would have to be assigned to the frame and to the equipment inside.

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

solid area? What area should be used to obtain the wind force per foot of tower height acting (1) normal to a tower face, and (2) along a tower diagonal?

The gross area of one panel face is (10)(10) = 100 ft^2, and the solidity ratio is € 22/100 = 0.22. For a tower of square cross section, the force coefficient from Fig. 29.5-3 is as follows:

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

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

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

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

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

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

25. In calculating the wind forces acting on a trussed tower of square cross section (see Fig. 29.5-3), should the force coefficient, C_f, be applied to both the front and the back (windward and leeward) faces of the tower?

No. The calculated wind forces are the total forces acting on the tower. The force coefficients given in Fig. 29.5-3 include the contributions of both front and back faces of the tower, as well as the shielding effect of the front face on the back face.

14.8 Miscellaneous

26. Equation 27.3-1 for velocity pressure uses the subscript *z* while other equations use subscripts *z* and *h*. When is *z* used and when is *h* used?

Equations 27.3-1 (as well as 28.3-1, 29.3-1, and 30.3-1) are the general formulas for the velocity pressure, q_z, at any height, z, above ground. There are many situations in the Standard where a specific value of z is called for, namely the height (or mean roof height) of a building or other structure. Whenever the subscript h is called for, it is understood that z becomes h (mean roof height) in the appropriate equations.

27. Section 26.2 of the Standard provides definitions of glazing, impact protective system; and wind-borne debris regions. To be impact resistant, the Standard specifies that the glazing of the building envelope must be shown by an approved test method to

withstand the impact of wind-borne missiles likely to be generated during design winds. Where does one find information on appropriate test methods?

Section 26.10 of the Standard refers to two standards, ASTM E1886 and ASTM E1996. Additional information on these standards is provided in Section C26.10 of the 7-10 Standard. The two standards specify test methods and performance standards.

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

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

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

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

30. What wind loads do I use during construction?

ASCE 7 does not address wind loads during construction. Construction loads are specifically addressed in the standard SEI/ASCE 37-02, *Design Loads on Structures during Construction*.

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

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

32. How do I determine what wind pressures to use for the design of a complex building with multiple uses when there are different wind speeds for different building classifications?

Wind pressures based on wind speeds should be determined by the Main Wind Force Resisting Systems. If the buildings are built such that the MWRFS is the same for all buildings, the design wind speed should be based on the most restrictive or critical use. If the MWFRS can be separated such that there is a different system for each use, then the appropriate design wind speed associated with the appropriate use should be used. The C&C pressures will follow as well with the design wind speed associated with the appropriate use.

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