BUNGALE S. TARANATH Ph.D., S.E.

WIND and EARTHQUAKE RESISTANT BUILDINGS

STRUCTURAL ANALYSIS AND DESIGN

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STRUCTURAL ANALYSIS AND DESIGN

BUNGALE S. TARANATH Ph.D., S.E.
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This book is dedicated to my wife
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without whose patience and devotion, this book would not be.

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Preface

The primary objective of this book is to disseminate information on the latest concepts, techniques, and design data to structural engineers engaged in the design of wind- and seismic-resistant buildings. Integral to the book are recent advances in seismic design, particularly those related to buildings in zones of low and moderate seismicity. These stipulations, reflected in the latest provisions of American Society of Civil Engineers (ASCE) 7-02, International Building Code (IBC)-03, and National Fire Protection Association (NFPA) 5000, are likely to be adopted as a design standard by local code agencies. There now exists the unprecedented possibility of a single standard becoming a basis for earthquake-resistant design virtually in the entire United States, as well as in other nations that base their codes on U.S. practices. By incorporating these and the latest provisions of American Concrete Institute (ACI) 318-02, American Institute of Steel Construction (AISC) 341-02, and Federal Emergency Management Agency (FEMA) 356 and 350 series, this book equips designers with up-to-date information to execute safe designs, in accordance with the latest regulations.

Chapter 1 presents methods of determining design wind loads using the provisions of ASCE 7-02, National Building Code of Canada (NBCC) 1995, and 1997 Uniform Building Code (UBC). Wind-tunnel procedures are discussed, including analytical methods for determining along-wind and across-wind response.

Chapter 2 discusses the seismic design of buildings, emphasizing their behavior under large inelastic cyclic deformations. Design provisions of ASCE 7-02 (IBC-03, NFPA 5000) and UBC-97 that call for detailing requirements to assure seismic performance beyond the elastic range are discussed using static, dynamic, and time-history procedures. The foregone design approach—in which the magnitude of seismic force and level of detailing were strictly a function of the structure's location—is compared with the most recent provisions, in which these are not only a function of the structure's location, but also of its use and occupancy, and the type of soil it rests upon. This comparison will be particularly useful for engineers practicing in many seismically low- and moderate-risk areas of the United States, who previously did not have to deal with seismic design and detailing, but are now obligated to do so. Also explored are the seismic design of structural elements, nonstructural components, and equipment. The chapter concludes with a review of structural dynamic theory.

The design of steel buildings for lateral loads is the subject of Chapter 3. Traditional as well as modern bracing systems are discussed, including outrigger and belt truss systems that have become the workhorse of lateral bracing systems for super-tall buildings. The lateral design of concentric and eccentric braced frames, moment frames with reduced beam section, and welded flange plate connections are discussed, using provisions of ASCE 341-02 and FEMA-350 as source documents.

Chapter 4 addresses concrete structural systems such as flat slab frames, coupled shear walls, frame tubes, and exterior diagonal and bundled tubes. Basic concepts of structural behavior that emphasize the importance of joint design are discussed. Using design provisions of ACI 318-02, the chapter also details building systems such as ordinary, intermediate, and special reinforced concrete moment frames, and structural walls.

The design of buildings using a blend of structural steel and reinforced concrete, often referred to as composite construction, is the subject of Chapter 5. The design of composite beams, columns, and shear walls is discussed, along with building systems such as composite shear walls and megaframes.

Chapter 6 is devoted to the structural rehabilitation of seismically vulnerable buildings. Design differences between a code-sponsored approach and the concept of ductility trade-off for strength are discussed, including seismic deficiencies and common upgrade methods.

Chapter 7 is dedicated to the gravity design of vertical and horizontal elements of steel, concrete, and composite buildings. In addition to common framing types, novel systems such as haunch and stub girder systems are also discussed. Considerable coverage is given to the design of prestressed concrete members based on the concept of load balancing.

The final chapter is devoted to a wide range of topics. Chapter 8 begins with a discussion of the evolution of different structural forms particularly applicable to the design of tall buildings. Case studies of buildings with structural systems that range from run-of-the-mill bracing techniques to unique composite systems—including megaframes and external superbraced frames—are examined. Next, reduction of building occupants' motion perceptions using damping devices is considered, including tuned mass dampers, slashing water dampers, tuned liquid column dampers, and simple and nested pendulum dampers. Panel zone effects, differential shortening of columns, floor-leveling problems, and floor vibrations are studied, followed by a description of seismic base isolation and energy dissipation techniques. The chapter concludes with an explanation of buckling-restrained bracing systems that permit plastic yielding of compression braces.

The book speaks to a multifold audience. It is directed toward consulting engineers and engineers employed by federal, state, and local governments. Within the academy, the book will be helpful to educators and students alike, particularly as a teaching tool in courses for students who have completed an introductory course in structural engineering and seek a deeper understanding of structural design principles and practice. To assist readers in visualizing the response of structural systems, numerous illustrations and practical design problems are provided throughout the text.

Wind- and Earthquake-Resistant Buildings integrates the design aspects of steel, concrete, and composite buildings within a single text. It is my hope that it will serve as a comprehensive design reference for practicing engineers and educators.

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Wind Loads

1.1. DESIGN CONSIDERATIONS

Windstorms pose a variety of problems in buildings—particularly in tall buildings—causing concerns for building owners, insurers, and engineers alike. Hurricane winds are the largest single cause of economic and insured losses due to natural disasters, well ahead of earthquakes and floods. For example, in the United States between 1986 and 1993, hurricanes and tornadoes caused about $41 billion in insured catastrophic losses, compared with $6.18 billion for all other natural hazards combined, hurricanes being the largest contributor to the losses. In Europe in 1900 alone, four winter storms caused $10 billion in insured losses, and an estimated $15 billion in economic losses. According to one 1999 insurance industry estimate, the natural catastrophe resulting in the largest amount of insured losses up to that date was hurricane Andrew in 1992 ($16.5 billion). The runner-up, the 1994 Northridge earthquake, resulted in $12.5 billion in reported losses.

In designing for wind, a building cannot be considered independent of its surroundings. The influence of nearby buildings and land configuration on the sway response of the building can be substantial. The sway at the top of a tall building caused by wind may not be seen by a passerby, but may be of concern to those occupying its top floors. There is scant evidence that winds, except those due to a tornado or hurricane, have caused major structural damage to new buildings. However, a modern skyscraper, with lightweight curtain walls, dry partitions, and high-strength materials, is more prone to wind motion problems than the early skyscrapers, which had the weight advantage of masonry partitions, heavy stone facades, and massive structural members.

To be sure, all buildings sway during windstorms, but the motion in earlier tall buildings with heavy full-height partitions has usually been imperceptible and certainly has not been a cause for concern. Structural innovations and lightweight construction technology have reduced the stiffness, mass, and damping characteristics of modern buildings. In buildings experiencing wind motion problems, objects may vibrate, doors and chandeliers may swing, pictures may lean, and books may fall off shelves. If the building has a twisting action, its occupants may get an illusory sense that the world outside is moving, creating symptoms of vertigo and disorientation. In more violent storms, windows may break, creating safety problems for pedestrians below. Sometimes, strange and frightening noises are heard by the occupants as the wind shakes elevators, strains floors and walls, and whistles around the sides.

Following are some of the criteria that are important in designing for wind:

1. Strength and stability.
2. Fatigue in structural members and connections caused by fluctuating wind loads.
3. Excessive lateral deflection that may cause cracking of internal partitions and external cladding, misalignment of mechanical systems, and possible permanent deformations of nonstructural elements.

4. Frequency and amplitude of sway that can cause discomfort to occupants of tall, flexible buildings.
5. Possible buffeting that may increase the magnitude of wind velocities on neighboring buildings.
6. Wind-induced discomfort in pedestrian areas caused by intense surface winds.
7. Annoying acoustical disturbances.
8. Resonance of building oscillations with vibrations of elevator hoist ropes.

1.2. NATURE OF WIND

Wind is the term used for air in motion and is usually applied to the natural horizontal motion of the atmosphere. Motion in a vertical or nearly vertical direction is called a *current*. Movement of air near the surface of the earth is three-dimensional, with horizontal motion much greater than the vertical motion. Vertical air motion is of importance in meteorology but is of less importance near the ground surface. On the other hand, the horizontal motion of air, particularly the gradual retardation of wind speed and the high turbulence that occurs near the ground surface, are of importance in building engineering. In urban areas, this zone of turbulence extends to a height of approximately one-quarter of a mile aboveground, and is called the surface boundary layer. Above this layer, the horizontal airflow is no longer influenced by the ground effect. The wind speed at this height is called the *gradient wind speed*, and it is precisely in this boundary layer where most human activity is conducted. Therefore, how wind effects are felt within this zone is of great concern.

Although one cannot see the wind, it is a common observation that its flow is quite complex and turbulent in nature. Imagine taking a walk outside on a reasonably windy day. You no doubt experience the constant flow of wind, but intermittently you will experience sudden gusts of rushing air. This sudden variation in wind speed, called gustiness or turbulence, plays an important part in determining building oscillations.

1.2.1. Types of wind

Winds that are of interest in the design of buildings can be classified into three major types: prevailing winds, seasonal winds, and local winds.

1. *Prevailing winds*. Surface air moving toward the low-pressure equatorial belt is called prevailing winds or trade winds. In the northern hemisphere, the northerly wind blowing toward the equator is deflected by the rotation of the earth to become northeasterly and is known as the northeast trade wind. The corresponding wind in the southern hemisphere is called the southeast trade wind.
2. *Seasonal winds*. The air over the land is warmer in summer and colder in winter than the air adjacent to oceans during the same seasons. During summer, the continents become seats of low pressure, with wind blowing in from the colder oceans. In winter, the continents experience high pressure with winds directed toward the warmer oceans. These movements of air caused by variations in pressure difference are called seasonal winds. The monsoons of the China Sea and the Indian Ocean are an examples.
3. *Local winds*. Local winds are those associated with the regional phenomena and include whirlwinds and thunderstorms. These are caused by daily changes in temperature and pressure, generating local effects in winds. The daily variations in temperature and pressure may occur over irregular terrain, causing valley and mountain breezes.

All three types of wind are of equal importance in design. However, for the purpose of evaluating wind loads, the characteristics of the prevailing and seasonal winds are analytically studied together, whereas those of local winds are studied separately. This grouping is to distinguish between the widely differing scale of fluctuations of the winds; prevailing and seasonal wind speeds fluctuate over a period of several months, whereas the local winds vary almost every minute, The variations in the speed of prevailing and seasonal winds are referred to as *fluctuations* in mean velocity. The variations in the local winds, are referred to as *gusts*.

The flow of wind, unlike that of other fluids, is not steady and fluctuates in a random fashion. Because of this, wind loads imposed on buildings are studied statistically.

1.3. CHARACTERISTICS OF WIND

The flow of wind is complex because many flow situations arise from the interaction of wind with structures. However, in wind engineering, simplifications are made to arrive at design wind loads by distinguishing the following characteristics:

- Variation of wind velocity with height.
- Wind turbulence.
- Statistical probability.
- Vortex shedding phenomenon.
- Dynamic nature of wind–structure interaction.

1.3.1. Variation of Wind Velocity with Height

The viscosity of air reduces its velocity adjacent to the earth's surface to almost zero, as shown in Fig. 1.1. A retarding effect occurs in the wind layers near the ground, and these inner layers in turn successively slow the outer layers. The slowing down is reduced at each layer as the height increases, and eventually becomes negligibly small. The height at which velocity ceases to increase is called the gradient height, and the corresponding velocity, the gradient velocity. This characteristic of variation of wind velocity with height is a well-understood phenomenon, as evidenced by higher design pressures specified at higher elevations in most building codes.

At heights of approximately 1200 ft (366 m) aboveground, the wind speed is virtually unaffected by surface friction, and its movement is solely dependent on prevailing seasonal and local wind effects. The height through which the wind speed is affected by topography is called the *atmospheric boundary layer.* The wind speed profile within this layer is given by

$$V_z = V_g(Z/Z_g)^{1/\alpha} \tag{1.1}$$

where

V_z = mean wind speed at height Z aboveground
V_g = gradient wind speed assumed constant above the boundary layer
Z = height aboveground
Z_g = nominal height of boundary layer, which depends on the exposure (Values for Z_g are given in Fig. 1.1.)
α = power law coefficient

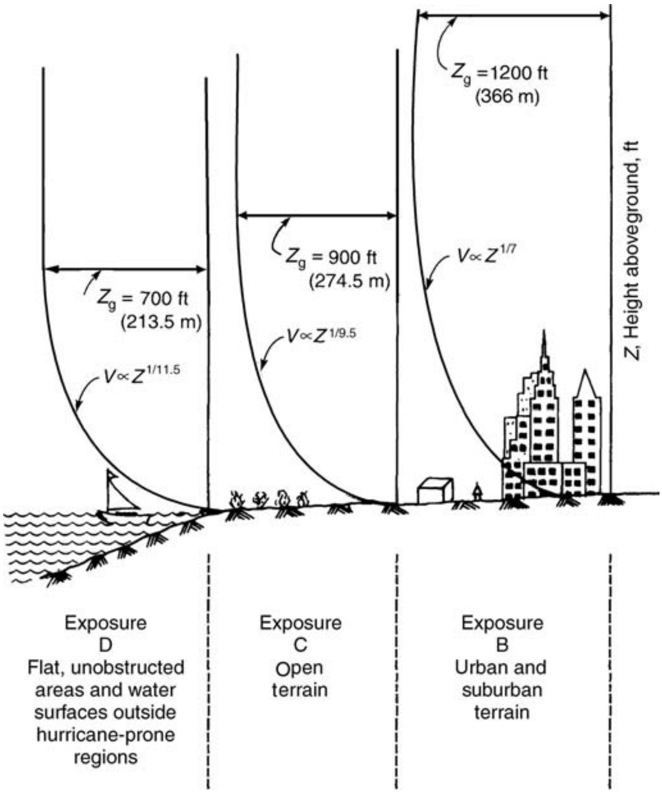


Figure 1.1. Influence of exposure terrain on variation of wind velocity with height.

With known values of mean wind speed at gradient height and exponent α, wind speeds at height Z are calculated by using Eq. (1.1). The exponent $1/\alpha$ and the depth of boundary layer Z_g vary with terrain roughness and the averaging time used in calculating wind speed. α ranges from a low of 0.087 for open country of 0.20 for built-up urban areas, signifying that wind speed reaches its maximum value over a greater height in an urban terrain than in the open country.

1.3.2. Wind Turbulence

Motion of wind is turbulent. A concise mathematical definition of turbulence is difficult to give, except to state that it occurs in wind flow because air has a very low viscosity—about one-sixteenth that of water. Any movement of air at speeds greater than 2 to 3 mph (0.9 to 1.3 m/s) is turbulent, causing particles of air to move randomly in all directions. This is in contrast to the laminar flow of particles of heavy fluids, which move predominantly parallel to the direction of flow.

For structural engineering purposes, velocity of wind can be considered as having two components: a mean velocity component that increases with height, and a turbulent velocity that remains the same over height (Fig. 1.1a). Similarly, the wind pressures, which are proportional to the square of the velocities, also fluctuate as shown in Fig. 1.2. The total pressure P_t at any instant t is given by the relation

$$P_t = \overline{P} + P' \tag{1.2}$$

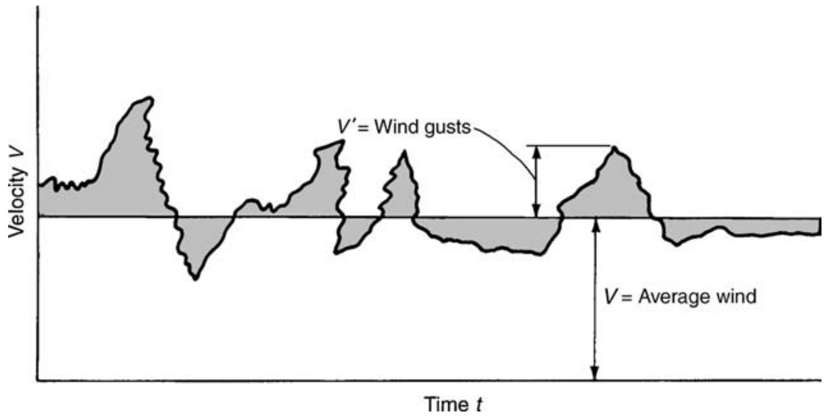


Figure 1.1a. Variation of wind velocity with time; at any instant t, velocity $V_t = V' + V$.

where

P_t = pressure at instant t
$\overline{P}$ = average or mean pressure
P' = instantaneous pressure fluctuation

1.3.3. Probabilistic Approach

In many engineering sciences the intensity of certain events is considered to be a function of the duration recurrence interval (return period). For example, in hydrology the intensity of rainfall expected in a region is considered in terms of a return period because the rainfall expected once in 10 years is less than the one expected once every 50 years. Similarly, in wind engineering the speed of wind is considered to vary with return periods. For example, the fastest-mile wind 33 ft (10 m) above ground in Dallas, TX, corresponding

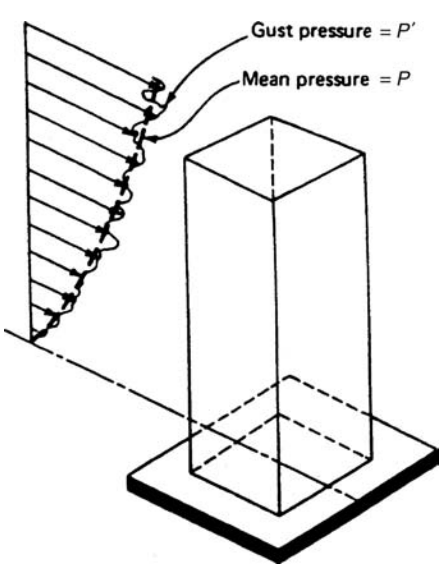


Figure 1.2. Schematic representation of mean and gust pressure. At any instant t, the pressure $P_t = P' + P$.

to a 50-year return period, is 67 mph (30 m/s), compared to the value of 71 mph (31.7 m/s) for a 100-year recurrence interval.

A 50-year return-period wind of 67 mph (30 m/s) means that on the average, Dallas will experience a wind faster than 67 mph within a period of 50 years. A return period of 50 years corresponds to a probability of occurrence of 1/50 = 0.02 = 2%. Thus the chance that a wind exceeding 67 mph (30 m/s) will occur in Dallas within a given year is 2%. Suppose a building is designed for a 100-year lifetime using a design wind speed of 67 mph. What is the probability that this wind will exceed the design speed within the lifetime of the structure? The probability that this wind speed will not be exceeded in any year is 49/50. The probability that this speed will not be exceeded 100 years in a row is $(49/50)^{100}$. Therefore, the probability that this wind speed will be exceeded at least once in 100 years is

$$1 - \left(\frac{49}{50}\right)^{100} = 0.87 = 87\%$$

This signifies that although a wind with low annual probability of occurrence (such as a 50-year wind) is used to design structures, there still exists a high probability of the wind being exceeded within the lifetime of the structure. However, in structural engineering practice it is believed that the actual probability of overstressing a structure is much less because of the factors of safety and the generally conservative values used in design.

It is important to understand the notion of probability of occurrence of design wind speeds during the service life of buildings. The general expression for probability P that a design wind speed will be exceeded at least once during the exposed period of n years is given by

$$P = 1 - (1 - P_a)^n \tag{1.3}$$

where

P_a = annual probability of being exceeded (reciprocal of the mean recurrence interval)
n = exposure period in years

Consider a building in Dallas designed for a 50-year service life instead of 100 years. The probability of exceeding the design wind speed at least once during the 50-year lifetime of the building is

$$P = 1 - (1 - 0.02)^{50} = 1 - 0.36 = 0.64 = 64\%$$

The probability that wind speeds of a given magnitude will be exceeded increases with a longer exposure period of the building and the mean recurrence interval used in the design. Values of P for a given mean recurrence interval and a given exposure period are shown in Table 1.1.

Wind velocities (measured with anemometers usually installed at airports across the country) are necessarily averages of the fluctuating velocities measured during a finite interval of time. The value usually reported in the United States, until the publication of the American Society of Civil Engineers' ASCE 7-95 standard, was the average of the velocities recorded during the time it takes a horizontal column of air 1 mile long to pass a fixed point. For example, if a 1-mile column of air is moving at an average velocity of 60 mph, it passes an anemometer in 60 seconds, the reported velocity being the average of the velocities recorded these 60 seconds. The fastest mile is the highest velocity in one day. The annual extreme mile is the largest of the daily maximums. Furthermore, since the annual extreme mile varies from year to year, wind pressures used in design are based on

TABLE 1.1 Probability of Exceeding Design Wind Speed During Design Life of Building

Annual probability P_a	Mean recurrence interval ($1/P_a$) years	Exposure period (design life), n (years)					
		1	5	10	25	50	100
0.1	10	0.1	0.41	0.15	0.93	0.994	0.999
0.04	25	0.04	0.18	0.34	0.64	0.87	0.98
0.034	30	0.034	0.15	0.29	0.58	0.82	0.97
0.02	50	0.02	0.10	0.18	0.40	0.64	0.87
0.013	75	0.013	0.06	0.12	0.28	0.49	0.73
0.01	100	0.01	0.05	0.10	0.22	0.40	0.64
0.0067	150	0.0067	0.03	0.06	0.15	0.28	0.49
0.005	200	0.005	0.02	0.05	0.10	0.22	0.39

a wind velocity having a specific mean recurrence interval. Mean recurrence intervals of 20 and 50 years are generally used in building design, the former interval for determining the comfort of occupants in tall buildings subject to wind storms, and the latter for designing lateral resisting elements.

1.3.4. Vortex Shedding

In general, wind buffeting against a bluff body gets diverted in three mutually perpendicular directions, giving rise to forces and moments about the three directions. Although all six components, as shown in Fig.1.3, are significant in aeronautical engineering, in civil and structural work, the force and moment corresponding to the vertical axis (lift and yawing moment) are of little significance. Therefore, aside from the uplift forces on large roof areas, the flow of wind is simplified and considered two-dimensional, as shown in Fig.1.4, consisting of *along wind* and *transverse wind.*

Along wind—or simply wind—is the term used to refer to drag forces, and transverse wind is the term used to describe crosswind. The crosswind response causing motion in a plane perpendicular to the direction of wind typically dominates over the along-wind response for tall buildings. Consider a prismatic building subjected to a smooth wind flow.

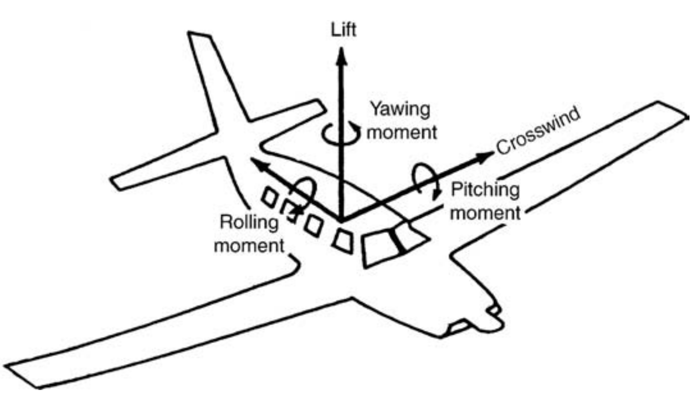


Figure 1.3. Six components of wind.

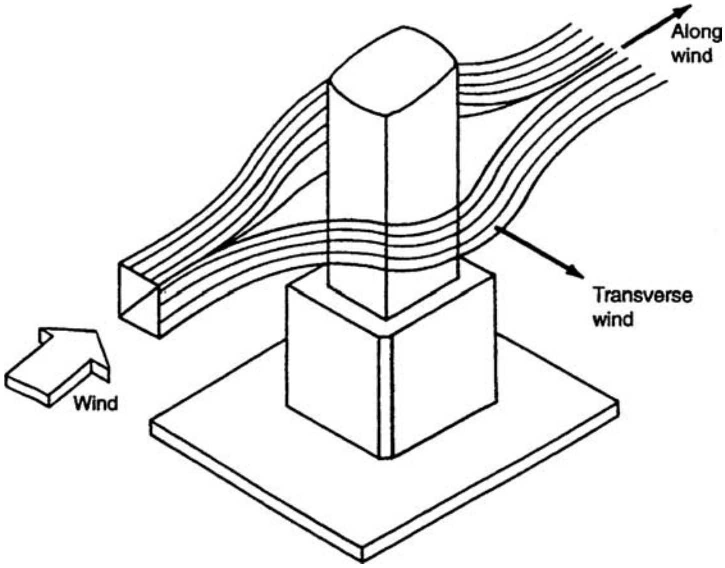


Figure 1.4. Simplified two-dimensional flow of wind.

The originally parallel upwind streamlines are displaced on either side of the building, Fig.1.5. This results in spiral vortices being shed periodically from the sides into the downstream flow of wind, called the *wake*. At relatively low wind speeds of, say, 50 to 60 mph (22.3 to 26.8 m/s), the vortices are shed symmetrically in pairs, one from each side. When the vortices are shed, i.e., break away from the surface of the building, an impulse is applied in the transverse direction.

At low wind speeds, since the shedding occurs at the same instant on either side of the building, there is no tendency for the building to vibrate in the transverse direction. It is therefore subject to along-wind oscillations parallel to the wind direction. At higher speeds, the vortices are shed alternately, first from one and then from the other side. When this occurs, there is an impulse in the along-wind direction as before, but in addition, there is an impulse in the transverse direction. The transverse impulses are, however, applied alternately to the left and then to the right. The frequency of transverse impulse is precisely half that of the along-wind impulse. This type of shedding, which gives rise to structural vibrations in the flow direction as well as in the transverse direction, is called *vortex shedding* or the *Karman vortex street*, a phenomenon well known in the field of fluid mechanics.

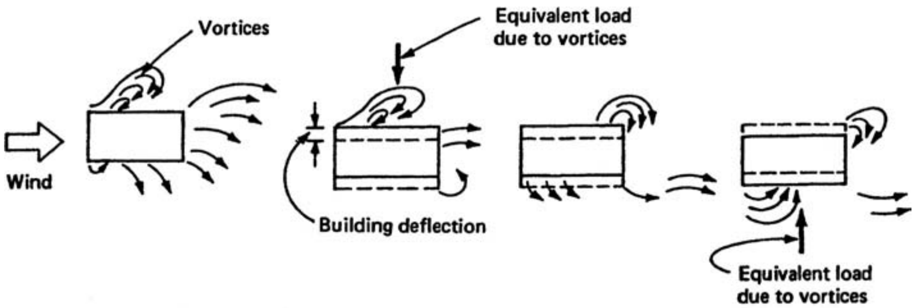


Figure 1.5. Vortex-shedding phenomenon.

There is a simple formula to calculate the frequency of the transverse pulsating forces caused by vortex shedding:

$$f = \frac{V \times S}{D} \tag{1.4}$$

where

f = frequency of vortex shedding in hertz
V = mean wind speed at the top of the building
S = a dimensionless parameter called the Strouhal number for the shape
D = diameter of the building

In Eq. (1.4), the parameters V and D are expressed in consistent units such as ft/s and ft, respectively.

The Strouhal number is not a constant but varies irregularly with wind velocity. At low air velocities, S is low and increases with the velocity up to a limit of 0.21 for a smooth cylinder. This limit is reached for a velocity of about 50 mph (22.4 m/s) and remains almost a constant at 0.20 for wind velocities between 50 and 115 mph (22.4 and 51 m/s).

Consider for illustration purposes, a circular prismatic-shaped high-rise building having a diameter equal to 110 ft (33.5 m) and a height-to-width ratio of 6 with a natural frequency of vibration equal to 0.16 Hz. Assuming a wind velocity of 60 mph (27 m/s), the vortex-shedding frequency is given by

$$f = \frac{V \times 0.2}{110} = 0.16 \text{ Hz}$$

where V is in ft/s.

If the wind velocity increases from 0 to 60 mph (27.0 m/s), the frequency of vortex excitation will rise from 0 to a maximum of 0.16 Hz. Since this frequency happens to be very close to the natural frequency of the building, and assuming very little damping, the structure would vibrate as if its stiffness were zero at a wind speed somewhere around 60 mph (27 m/s). Note the similarity of this phenomenon to the ringing of church bells or the shaking of a tall lamppost whereby a small impulse added to the moving mass at each end of the cycle greatly increases the kinetic energy of the system. Similarly, during vortex shedding an increase in deflection occurs at the end of each swing. If the damping characteristics are small, the vortex shedding can cause building displacements far beyond those predicted on the basis of static analysis.

When the wind speed is such that the shedding frequency becomes approximately the same as the natural frequency of the building, a resonance condition is created. After the structure has begun to resonate, further increases in wind speed by a few percent will not change the shedding frequency, because the shedding is now controlled by the natural frequency of the structure. The vortex-shedding frequency has, so to speak, locked in with the natural frequency. When the wind speed increases significantly above that causing the lock-in phenomenon, the frequency of shedding is again controlled by the speed of the wind. The structure vibrates with the resonant frequency only in the lock-in range. For wind speeds either below or above this range, the vortex shedding will not be critical.

Vortex shedding occurs for many building shapes. The value of S for different shapes is determined in wind tunnel tests by measuring the frequency of shedding for a range of wind velocities. One does not have to know the value of S very precisely because the lock-in phenomenon occurs within a range of about 10% of the exact frequency of the structure.

1.3.5. Dynamic Nature of Wind

Unlike the mean flow of wind, which can be considered as static, wind loads associated with gustiness or turbulence change rapidly and even abruptly, creating effects much larger than if the same loads were applied gradually. Wind loads, therefore, need to be studied as if they were dynamic in nature. The intensity of a wind load depends on how fast it varies and also on the response of the structure. Therefore, whether the pressures on a building created by a wind gust, which may first increase and then decrease, are considered as dynamic or static depends to a large extent on the dynamic response of the structure to which it is applied.

Consider the lateral movement of an 800-ft tall building designed for a drift index of *H*/400, subjected to a wind gust. Under wind loads, the building bends slightly as its top moves. It first moves in the direction of wind, with a magnitude of, say, 2 ft (0.61 m), and then starts oscillating back and forth. After moving in the direction of wind, the top goes through its neutral position, then moves approximately 2 ft (0.61 m) in the opposite direction, and continues oscillating back and forth until it eventually stops. The time it takes a building to cycle through a complete oscillation is known as a *period*. The period of oscillation for a tall steel building in the height range of 700 to 1400 ft (214 to 427 m) normally is in the range of 5 to 10 seconds, whereas for a 10-story concrete or masonry building it may be in the range of 0.5 to 1 seconds. The action of a wind gust depends not only on how long it takes the gust to reach its maximum intensity and decrease again, but on the period of the building itself. If the wind gust reaches its maximum value and vanishes in a time much shorter than the period of the building, its effects are dynamic. On the other hand, the gusts can be considered as static loads if the wind load increases and vanishes in a time much longer than the period for the building. For example, a wind gust that develops to its strongest intensity and decreases to zero in 2 seconds is a dynamic load for a tall building with a period of, say, 5 to 10 seconds, but the same 2-second gust is a static load for a low-rise building with a period of less than 2 seconds.

1.3.6. Cladding Pressures

The design of cladding for lateral loads is of major concern to architects and engineers. Although the failure of exterior cladding resulting in broken glass may be of less consequence than the collapse of a structure, the expense of replacement and hazards posed to pedestrians require careful consideration. Cladding breakage in a windstorm is an erratic occurrence, as witnessed in hurricane Alicia, which hit Galveston and downtown Houston on August 18, 1983, causing breakage of glass in several tall buildings. Wind forces play a major role in glass breakage, which is also influenced by other factors, such as solar radiation, mullion and sealant details, tempering of the glass, double- or single-glazing of glass, and fatigue. It is known with certainty that glass failure starts at nicks and scratches that may be made during manufacture, and by handling operations.

There appears to be no analytical approach available for a rational design of curtain walls of all shapes and sizes. Although most codes have tried to identify regions of high wind loads around building corners, the modern trend in architecture of using nonprismatic and curvilinear shapes combined with the unique topography of each site, has made experimental determination of wind loads even more necessary.

Thus it has become routine to obtain design information concerning the distribution of wind pressures over a building's surface by conducting wind tunnel studies. In the past two decades, curtain wall has developed into an ornamental item and has emerged as a significant architectural element. Sizes of window panes have increased considerably,

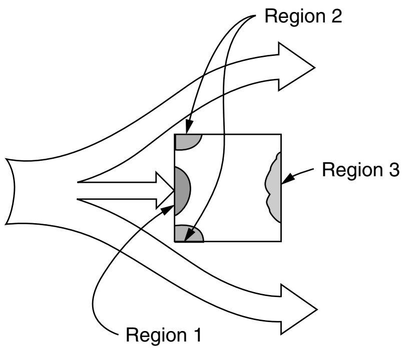


Figure 1.6. Distribution of pressures and suctions.

requiring that the glass panes be designed for various combinations of forces due to wind, shadow effects, and temperature movement. Glass in curtain walls must not only resist large forces, particularly in tall buildings, but must also be designed to accommodate the various distortions of the total building structure. Breaking of large panes of glass can cause serious damage to neighboring properties and can injure pedestrians.

1.3.6.1. Distribution of Pressures and Suctions

When air flows around edges of a structure, the resulting pressures at the corners are much in excess of the pressures on the center of elevation. This has been evidenced by damage caused to corner windows, eave and ridge tiles, etc., in windstorms. Wind tunnel studies conducted on scale models of buildings indicate that three distinct pressure areas develop around a building. These are shown schematically in Fig.1.6.

1. Positive-pressure zone on the upstream face (Region 1).
2. Negative pressure zones at the upstream corners (Regions 2).
3. Negative pressure zone on the downstream face (Region 3).

The highest negative pressures are created in the upstream corners designated as Regions 2 in Fig. 1.6. Wind pressures on a building's surface are not constant, but fluctuate continuously. The positive pressure on the upstream or the windward face fluctuates more than the negative pressure on the downstream or the leeward face. The negative-pressure region remains relatively steady as compared to the positive-pressure zone. The fluctuation of pressure is random and varies from point to point on the building surface. Therefore, the design of the cladding is strongly influenced by local pressures. As mentioned earlier, the design pressure can be thought of as a combination of the mean and the fluctuating velocity. As in the design of buildings, whether or not the pressure component arising from the fluctuating velocity of wind is treated as a dynamic or as a pseudostatic load is a function of the period of the cladding. The period of cladding on a building is usually on the order of 0.2 to 0.02 sec, which is much shorter than the period it takes for wind to fluctuate from a gust velocity to a mean velocity. Therefore, it is sufficiently accurate to consider both the static and the gust components of winds as equivalent static loads in the design of cladding.

The strength of glass, and indeed of any other cladding material, is not known in the same manner that the strengths of steel and concrete are known. For example, it is not possible to buy glass based on yield strength criteria as with steel. Therefore, the selection, testing, and acceptance criteria for glass are based on statistical probabilities rather than on absolute strength. The glass industry has addressed this problem, and commonly uses 8 failures per 1000 lights (panes) of glass as an acceptable probability of failure.

1.3.6.2. Local Cladding Loads and Overall Design Loads

The overall wind load for lateral analysis consists of combined positive and negative pressures around the building. The local wind loads that act on specific areas of the building are required for the design of exterior cladding elements and their connections to the building. The two types of loads differ significantly, and it is important that these differences be understood. These are

1. Local winds are more influenced by the configuration of the building than the overall loading.
2. The local load is the maximum load that may occur at any location at any time on any wall surface, whereas the overall load is the summation of positive and negative pressures occurring simultaneously over the entire building surface.
3. The intensity and character of local loading for any given wind direction and velocity differ substantially on various parts of the building surface, whereas the overall load is considered to have a specific intensity and direction.
4. The local loading is sensitive to the momentary nature of wind, but in determining the critical overall loading, only gusts of about 2 sec or more are significant.
5. Generally, maximum local negative pressures, also referred to as suctions, are of greater intensity than the overall load.
6. Internal pressures caused by leakage of air through cladding systems have a significant effect on local cladding loads but are of no consequence in determining the overall load.

The relative importance of designing for these two types of wind loading is quite obvious. Although proper assessment of overall wind load is important, very few, if any buildings have been toppled by winds. There are no classic examples of building failures comparable to the Tacoma bridge disaster. On the other hand, local failures of roofs, windows, and wall cladding are not uncommon.

The analytical determination of wind pressure or suction at a specific surface of a building under varying wind direction and velocity is a complex problem. Contributing to the complexity are the vagaries of wind action as influenced both by adjacent surroundings and the configuration of the wall surface itself. Much research is needed on the microeffects of common architectural features such as projecting mullions, column covers, and deep window reveals, etc. In the meantime, model testing of building wind tunnels is perhaps the only answer.

Probably the most important fact established by tests is that the negative or outward-acting wind loads on wall surfaces are greater and more critical than had formerly been assumed. These loads may be as much as twice the magnitude of positive loading. In most instances of local cladding failure, glass panels have been blown off of the building, not into it, and the majority of such failures have occurred in areas near building corners. Therefore it is important to give careful attention to the design of both anchorage and glazing details to resist outward-acting forces, especially near the corners.

Another feature that has come to light from model testing is that wind loads, both positive and negative, do not vary in proportion to height aboveground. Typically, the positive-pressure contours follow a concentric pattern as illustrated in Fig.1.7, with the highest pressure near the lower center of the facade, and pressures at the very top somewhat

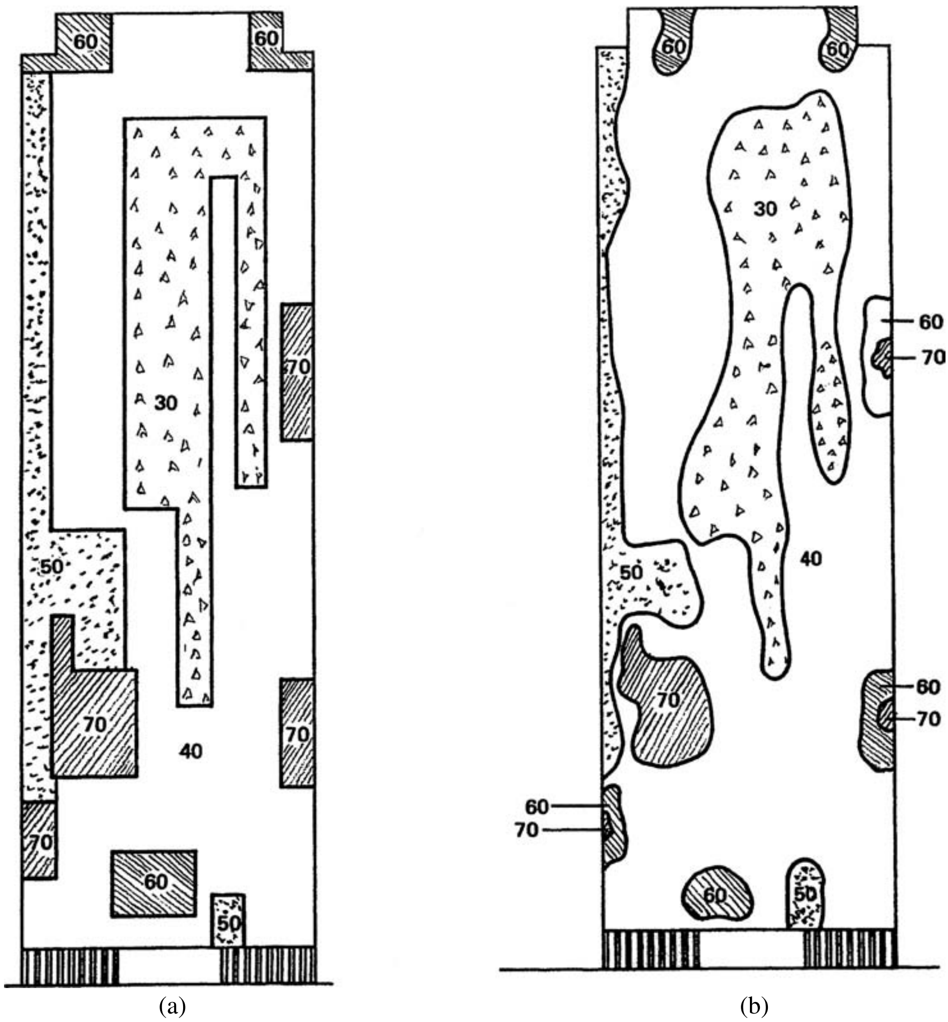


Figure 1.7. (a) Block pressure diagram, in psf; (b) Pressure countours in psf.

less than those a few stories below the roof. Figure 1.7a shows a pressure diagram for the design of cladding derived from pressure contours measured in wind tunnel tests shown in Fig.1.7b. The block pressure diagram shown in Fig.1.7a gives zones of design pressures based on the building grid system, to assist in the cladding design.

1.4. CODE PROVISIONS FOR WIND LOADS

In recent years, wind loads specified in codes and standards have been refined significantly. This is because our knowledge of how wind affects buildings and structures has expanded due to new technology and advanced research that have ensued in greater accuracy in predicting wind loads. We now have an opportunity to design buildings that will satisfy anticipated loads without excessive conservatism. The resulting complexity in the determination of wind loads may be appreciated by comparing the 1973 Standard Building Code (SBC), which contained only a page and one-half of wind load requirements, to the 2002 edition of the ASCE 7, which contains 97 pages of text, commentary, figures, and tables to predict wind loads for a particular structure. As compared to a single method given in the 1973 SBC, ASCE 7 contains three methods for determining winds: the simplified procedure, the analytical procedure, and the wind-tunnel procedure. The controlling equations for determining wind loads require calculating velocity pressure as before, but are now modified to account for several variables such as gusts, internal pressure, and aerodynamic properties of the element under consideration, as well as topographic effects. Using the low-rise buildings' analytical procedure in ASCE-7 and applying it to the simplest building requires the use of up to 11 variables. An important criterion that influences the calculation of wind loads is the enclosure classification of the building. Three classifications are used: 1) enclosed; 2) partially enclosed; or 3) open. A building classified as partially enclosed assumes that a large opening is on one side of a building and no (or minimal) openings are on the other walls. As openings on one wall reach a certain size with respect to openings on the other walls, the building is classified as partially enclosed. Depending upon the wind's direction, this type of situation allows two conditions to develop: internal pressure or internal suction. Internal pressure occurs when air enters a building opening on the windward wall and becomes trapped, exerting an additional force on the interior elements of the building. Typically the internal pressures act in the same directions as the external pressures on all walls except the windward wall. Internal suction is a condition that exists when there is an opening on the leeward wall allowing air to be pulled out of the building. This results in the internal forces acting in the same direction as the external forces on the windward wall. The additional forces produced by this type of pressurization are characterized by requiring an internal pressure coefficient that is more than three times greater than that required for an enclosed building.

Another criterion that significantly affects the magnitude of the wind pressures is the site's exposure category, which provides a way to define the relative roughness of the boundary layers at the site.

The ASCE 7-02 and IBC-03 define three exposure categories: B, C, and D. Exposure B is the roughest and D is the smoothest. Consequently, when all other conditions are equal, calculated wind loads are reduced as the exposure category moves from D to B. Exposure B is the most common category, consisting primarily of terrain associated with a suburban or urban site. Accordingly, B is the default exposure category in both ASCE 7 and IBC. Exposure C consists primarily of open terrain with scattered obstructions but also includes shoreline in hurricane-prone regions. Exposure D applies to shore lines (excluding those in hurricane-prone regions) with wind flowing over open water for a distance of at least one mile.

Buildings must also be classified based on their importance. The wind importance factor I_w specified in the codes is used to adjust the return period for a structure based on its relative level of importance. For example, the importance factor for structures housing critical national defense functions is 1.15, while the importance factor for an agricultural building not as critical as a defense facility, is 0.87.

The applicable wind speeds for the United States and some tropical islands specified in the wind speed maps are three-second gusts at 33 feet above ground for Exposure Category C. In the model codes that preceded the IBC (the National Building Code. Standard Building Code, and Uniform Building Code) and versions of ASCE 7 prior to 1995, wind speeds were shown as "fastest-mile winds," which is defined as the average speed of a one-mile column of air passing a reference point.

While the designated 3-sec gust wind speed for a particular site is higher than values on the fastest-mile map, the averaging times are also different. The averaging time for a fastest-mile wind speed is different for each wind speed, while the averaging time for the 3-sec gust speeds varies from 3 to 8 sec, depending upon the sensitivity of the instruments.

Wind load provisions given in three nationally and internationally recognized standards are discussed in this section. These are the

1. Uniform Building Code (UBC) 1997.
2. ASCE Minimum Design Loads for Buildings and Other Structures (ASCE 7-02).
3. National Building Code of Canada (NBCC) 1995.

1.4.1. Uniform Building Code, 1997: Wind Load Provisions

Wind load provisions of UBC 1997 are based on the ASCE 7-88 standard with certain simplifying assumptions to make calculations easier. The design wind speed is based on the fastest-mile wind speed as compared to the 3-sec gust speeds of the later codes. The prevailing wind direction at the site is not considered in calculating wind forces on the structures: The direction that has the most critical exposure controls the design. Consideration of shielding by adjacent buildings is not permitted because studies have shown that in certain configurations, the nearby buildings can actually increase the wind speed through funneling effects or increased turbulence. Additionally, it is possible that adjacent existing buildings may be removed during the life of the building being designed.

To shorten the calculation procedure, certain simplifying assumptions are made. These assumptions do not allow determination of wind loads for flexible buildings that may be sensitive to dynamic effects and wind-excited oscillations such as vortex shedding. Such buildings typically are those with a height-to-width ratio greater than 5, and over 400 ft (121.9 m) in height. The general section of the UBC directs the user to an approved standard for the design of these types of structures. The ASCE 7-02, adopted by IBC 2003 (discussed later in this chapter), is one such standard for determining the dynamic gust response factor required for the design of these types of buildings.

UBC provisions are not applicable to buildings taller than 400 ft (122 m) for normal force method, Method 1, and 200 ft (61 m) for projected area method, Method 2. Any building, including those not covered by the UBC, may be designed using wind-tunnel test results.

1.4.1.1. Wind Speed Map

The minimum basic wind speed at any site in the United States is shown in Fig. 1.8. The wind speed represents the fastest-mile wind speed in an exposure C terrain at 33 ft (l0 m) above grade, for a 50-year mean recurrence interval. The probability of experiencing a wind speed faster than the value indicted in the map, in any given year is 1 in 50, or 2%.

1.4.1.2. Special Wind Regions

Although basic wind speeds are constant over hundreds of miles, some areas have local weather or topographic characteristics that affect design wind speeds. These special wind regions are defined in the UBC map. Because some jurisdictions prescribe basic wind speeds higher than the map, it is prudent to contact local building officials before commencing with the wind design.

1.4.1.3. Hurricanes and Tornadoes

The wind speeds shown in the UBC map come from data collected by meterological stations throughout the continental United States, Alaska, Hawaii, Puerto, Rico, and Virgin Islands. However, coastal regions did not have enough statistical measurements to predict hurricane wind speeds. Therefore data generated by computer simulations have been used to formulate basic hurricane wind speeds.

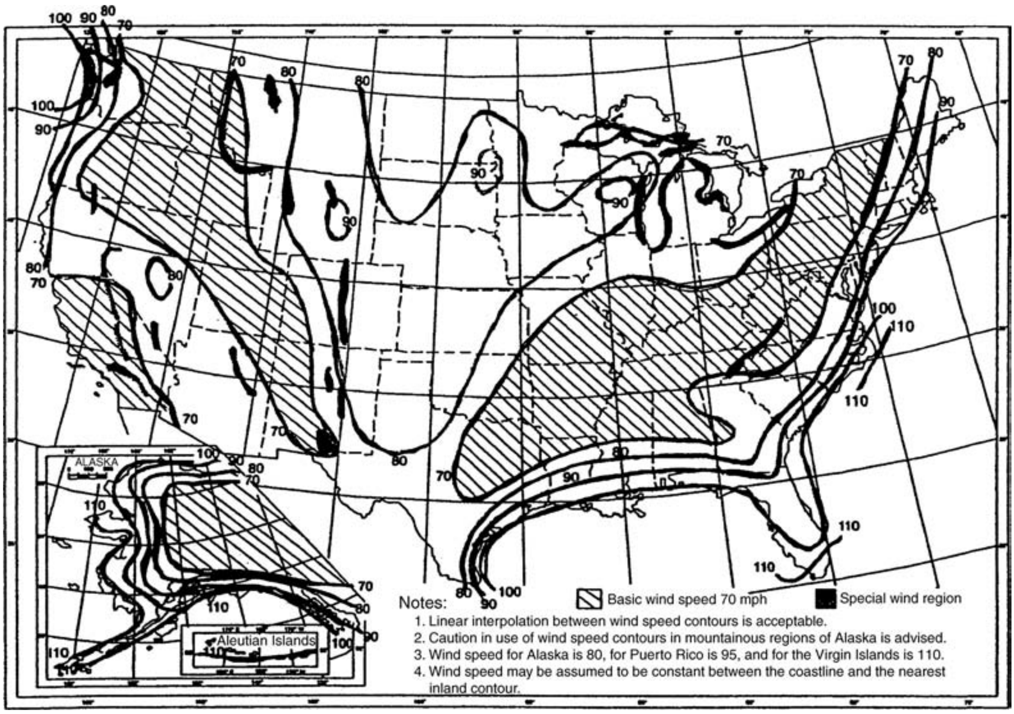


Figure 1.8. Minimum basic wind speeds in miles per hour (× 1.61 for km/h). (From UBC 1997.)

Tornado level winds are not included in the map because the mean recurrence intervals of tornadoes are in the range of 400–500 years, as compared to the 50 years interval typically used in wind design.

1.4.1.4. Exposure Effects

Every building site has its own unique characteristics in terms of surface roughness and length of upwind terrain associated with the roughness. Simplified code methods cannot account for the uniqueness of the site. Therefore the code approach is to assign broad exposure categories for design purposes.

Similar to the ASCE method, the UBC distinguishes between three exposure categories; B, C, and D. Exposure B is the least severe, representing urban, suburban, wooded, and other terrain with numerous closely spaced surface irregularities; Exposure C is for flat and generally open terrain with scattered obstructions; and the most severe, Exposure *D*, is four unobstructed coastal areas directly exposed to large bodies of water. Discussion of the exposure categories follows.

It should be noted that Exposure A (centers of large cities where over half the buildings have a height in excess of 70 feet), included in some standards, is not recognized in the UBC. The UBC considers this type of terrain as Exposure B, allowing no further decrease in wind pressure.

Exposure B has terrain with buildings, forest, or surface irregularities, covering at least 20% of the ground level area extending 1 mile (1.61 km) or more from the site.

Exposure C has terrain that is flat and generally open, extending one-half mile (0.81 km) or more from the site in any full quadrant.

Exposure D represents the most severe exposure in areas of basic wind speeds of 80 mph (129 km/h) or greater, and has terrain that is flat and unobstructed facing large bodies of water over one mile (1.61 km) or more in width relative to any quadrant of the building site. Exposure D extends inland from shoreline one-fourth mile (0.4 km) or 10 times the building height, whichever is greater.

1.4.1.5. Site Exposure

Even though a building site may have different exposure categories in different directions, the most severe exposure is used for all wind-load calculations regardless of building orientation or direction of wind.

Exposure D is perhaps the easiest to determine because it is explicitly for unobstructed coastal areas directly exposed to large bodies of water. It is not as easy to determine whether a site falls into Exposure B or C because the description of these categories is somewhat ambiguous. Morevoer, the terrain surrounding a site is usually not uniform and can be composed of zones that would be classified as Exposure B while others would be classified as Exposure C. When such a mix is encountered, the more severe exposure governs. The UBC classifies a site as Exposure C when open terrain exists for one full 90° quadrant extending outward from the building for at least one-half mile. If the quadrant is less than 90° or less than one-half mile, then the site is classified as Exposure B. It is essential to select the appropriate category because force levels could differ by as much as 65% between Exposure B and C. It is advisable to contact the local building official before embarking on a building design with a questionable site exposure category. If the site has a view of a cliff or hill, it may be prudent to assign Exposure C to D to account for higher wind velocity effects.

1.4.1.6. Design Wind Pressures

The design wind pressure p is given as a product of the combined height, exposure, and gust factor coefficient C_e; the pressure coefficient C_q; the wind stagnation pressure q_s; and building Importance Factor I_w.

$$p = C_e C_q q_s I_w \tag{1.5}$$

The pressure q_s manifesting on the surface of a building due to a mass of air with density ρ, moving at a velocity v is given by Bernoulli's equation:

$$q_s = \tfrac{1}{2}\rho V^2 \tag{1.6}$$

The density of air ρ is 0.0765 pcf, for conditions of standard atmosphere, temperature (59 °F), and barometric pressure (29.92 in. of mercury).

Since velocity given in the wind map is in mph, Eq. (1.6) reduces to

$$q_s = \frac{1}{2}\left[\frac{0.0765\text{ pcf}}{32.2\text{ ft/s}^2}\right]\left[\frac{5280\text{ft}}{\text{mile}} \times \frac{1\text{ hr}}{3600\text{s}}\right]V^2 \tag{1.7}$$
$$q_s = 0.00256V^2$$

For instance, if the wind speed is 80 mph, $q_s = 0.00256 \times 80^2 = 16.38$ psf, which the UBC rounds off to 16.4 psf (Table 2.10). Note UBC does not consider the effect of reduced air density at sites located at higher altitudes.

1.4.1.7. The C_e Factor

The effects of height, exposure, and gust factor are all lumped into one factor C_e in the interest of keeping the UBC method simple. Values of C_e shown in Table 1.2 (UBC, Table 16-G) are essentially equal to the product of two parameters—K_z, the velocity pressure exposure

TABLE 1.2 Combined Height, Exposure, and Gust Factor Coefficient (C_e)[a]

Height above average level of adjoining ground (feet) × 304.8 for mm	Exposure D	Exposure C	Exposure B
0–15	1.39	1.06	0.62
20	1.45	1.13	0.67
25	1.50	1.19	0.72
30	1.54	1.23	0.76
40	1.62	1.31	0.84
60	1.73	1.43	0.95
80	1.81	1.53	1.04
100	1.88	1.61	1.13
120	1.93	1.67	1.20
160	2.02	1.79	1.31
200	2.10	1.87	1.42
300	2.23	2.05	1.63
400	2.34	2.19	1.80

[a] Values for intermediate heights above 15 feet (4572 mm) may be interpolated.
(From UBC 1997, Table 16-G.)

coefficient, and G_h, the gust response factor. Both these parameters are defined separately in ASCE 7-02, and hence are more appropriate for non-ordinary buildings.

The height and exposure factors account for the terrain effects on gradient heights and typically cause lower wind speeds in built-up terrain than in an open terrain. The gust factor accounts for air turbulence and dynamic building behavior.

For low-rise buildings with natural period of less than 1 sec, the wind response is essentially static with the lateral deflection proportional to the wind force. For tall buildings, on the other hand, the response is dynamic resulting in deflections greater than those estimated by simple procedures. Therefore for slender buildings a procedure such as the one given in the ASCE 7-02, which takes into account the dynamic characteristic of the building, would likely to be more appropriate.

1.4.1.8. Pressure Coefficient C_q

The C_q given in Table 1.3 is a function of building shape and location, and whether the wind load induces inward or outward pressures.

It is given in two parts. The first part, Primary Frames Systems, is for the design of the entire building. The second part, Elements and Components of Structure, is for the design of cladding.

Wind gusting around a building does not cause peak pressures and sections simultaneously over the entire surface of the building. Therefore, wind loads for design of primary frames and systems are calculated using average wind pressures and suctions. On the other hand the design of building components such as curtain walls and cladding is controlled by the instantaneous peak pressures and suction acting over relatively small localized areas. This is the reason why the pressures and suctions for building components are larger than those for the entire building.

Wind pressures and suctions for primary systems are mainly a function of the building height. Although these are influenced by the building's shape, the roughness of its exterior, and its plan aspect ratio, these are ignored. For example, even though wind load on a circular building is theoretically about 80% of that for a rectangular building, no reduction of forces is permitted in the UBC.

TABLE 1.3 Pressure Coefficients C_g for Primary Frames and Systems

Description	C_g
Method 1 (Normal force method) Maximum height 400 ft	
Walls	
Windward wall	0.8 inward
Leeward wall	0.5 outward
Roof	
Wind perpendicular to ridge	
Leeward roof or flat roof	0.7 outward
Windward roof	
Slope less than 2:12 (16.7%)	0.7 outward
Slope 2:12 (16.7%) to less than 9:12 (75%)	0.9 outward or 0.3 inward
Slope 9:12 (75%) to 12:12 (100%)	0.4 inward
Slope > 12:12 (100%)	0.7 inward
Wind parallel to ridge and flat roofs	0.7 outward
Method 2 (Projected area method) Maximum height 200 ft	
On vertical projected area	
Structures 40 feet (12.19 m) or less in height	1.3 horizontal any direction
Structures over 40 feet (12.19 m) in height	1.4 horizontal any direction
On horizontal projected area	0.7 upward

(From UBC 1997.)

Two methods, are given in the UBC for determining wind loads for primary frames (Table 1.3). Method 1, the normal force method, is applicable to all structures, and is the only method permitted for the design of gable-roofed buildings. It assumes wind loads act perpendicular to the surfaces of the roof, and the walls. Method 2, the projected area method, is easier to use than Method 1. The wind pressures and suctions are integrated into a single value and are assumed to act on the entire projected area of the building, instead of on individual surfaces of roof and walls.

Another important difference between the two methods is that method 1 uses a constant value of C_e based on mean roof height to calculate wind suctions on leeward walls. Method 2 uses a C_e value that varies with height. Hence, method 2 underestimates the wind loads on taller structures. For this reason, use of method 2 is limited to structures less than 200 ft (61 m), in order to minimize the underestimated leeward forces.

1.4.1.9. Importance Factor I_w

Importance factor I_w is applied to increase the wind loads for certain occupancy categories. The 1997 UBC gives five separate occupancy categories: essential facilities, hazardous facilities, special occupancy structures, standard occupancy structures, and miscellaneous structures. Essential or hazardous facilities are assigned an importance factor $I_w = 1.15$, which has the effect of increasing the mean reference interval from a 50-year to a 10-year return period. Special structures, standard occupancy structures, and miscellaneous structures are assigned an importance factor I_w of 1.00. Office and residential buildings are typically assigned a standard occupancy factor of 1.00.

1.4.1.10. Design Examples, UBC 1997

Eleven-Story Building: UBC 1997.

Given.

- Eleven-story communication building deemed necessary for post-disaster emergency communications, $I_w = 1.15$

- Building height 120 ft (36.6 m) consisting of 2 bottom floors at 15 ft (4.6 m) and 9 typical floors at 10 ft (3.05 m)
- Exposure category = C
- Basic wind speed V = 100 mph
- Building width = 60 ft

Required. Design wind pressures on primary wind-resisting system.
Solution. The design pressure is given by the chain equation

$$p = C_e C_q q_s I_w$$

The values of C_e—the combined height, exposure, and gust factor coefficient tabulated in Table 1.4—are taken directly from Table 1.2. Note that for suction on the leeward face, C_e is at the roof hight, and is constant for the full height of the building. The wind pressure q_s corresponding to basic wind speed of 100 mph is given by

$$q_s = 0.00256V^2$$
$$q_s = 0.00256 \times 100^2 = 25.6\ psf$$

The values of pressure coefficient C_q (Table 1.3), obtained using the normal force method (Method 1), are 0.8 for the inward pressure on the windward face, and 0.5 for the suction on the leeward face. Because the building is less than 200 ft (61 m), the combined value of 0.8 + 0.5 = 1.3 may be used throughout the height to calculate the wind load on the primary wind-resisting system. Observe that Method 2 (projected area method) yields the same value of C_q = 1.3.

Design pressures and floor-by-floor wind loads are shown in Table 1.4. Notice that the wind pressure and suction on the lower half of the first story (between the ground and 7.5 ft aboveground) is commonly considered to be transmitted directly into the ground. The wind load at each level is obtained by multiplying the tributary area for the level by the average of design pressures above and below that level. For example,

$$\text{wind force at level } 10 = \frac{60 \times 10(62.6 + 61.6)}{1000 \times 2} = 37.52 \text{ kips}$$

Thirty-Story Building: UBC 1997.
Given.

Basic wind speed	90 mph
Plan dimensions of building	98.5 × 164 ft
Height of building	394 ft
Importance Factor I_w	1.0
Exposure Category D	Flat unobstructed terrain facing a large body of water

Required. Design wind pressures for lateral load analysis of the building.
Solution. The design wind pressure is given by

$$P = C_e C_c Q_s I_w$$

The values of C_e given in Table 1.2. (UBC Table 1.2) are shown for the example problem in Table 1.5. Observe that the coefficient C_e for the leeward wall is the value at the roof level, and remains constant for the entire building height. The pressure of corresponding to V = 90 mph is given by

$$q_s = 0.00256 \times 90^2$$
$$= 20.8 \text{ psf}$$

TABLE 1.4 Design Example: Design Loads for Primary Wind-Resisting System; UBC 1997 Procedure

Level (1)	Height above ground, ft (2)	C_e (3)		C_q (4)		Windward pressure, psf $p = C_eC_qq_sI_w$[b,c] (5)	Leeward suction, psf $p = C_eC_gq_sI_w$ (6)	Design pressure, psf (5) + (6)	Floor-by-floor load, kips
		Windward	Leeward	Windward	Leeward				
Roof	120	1.67	1.67	+0.8	−0.5	39.40	24.6	64.0	19.2
11	110	1.64	1.67	+0.8	−0.5	38.70	24.6	63.30	38.18
10	100	1.61	1.67	+0.8	−0.5	38.00	24.6	62.6	37.77
9	90	1.57	1.67	+0.8	−0.5	37.00	24.6	61.6	37.26
8	80	1.53	1.67	+0.8	−0.5	36.00	24.6	60.6	36.66
7	70	1.48	1.67	+0.8	−0.5	35.00	24.6	59.6	36.06
6	60	1.43	1.67	+0.8	−0.5	33.80	24.6	58.40	35.40
5	50	1.37	1.67	+0.8	−0.5	32.40	24.6	57.0	34.02
4	40	1.31	1.67	+0.8	−0.5	31.0	24.6	55.6	33.78
3	30	1.23	1.67	+0.8	−0.5	29.0	24.6	53.60	32.76
2	15	1.06	1.67	+0.8	−0.5	25.0	24.6	49.62	38.40

[a] Building width perpendicular to wind = 60 ft.
[b] $q_s = 25.60$ psf.
[c] $I_w = 1.15$.

TABLE 1.5 Design Example: 30-Story Building, Design Wind Loads; UBC 1997 Procedure

Level (1)	Height above ground, ft (2)	C_e Windward (3)	C_e Leeward (4)	C_q Windward (5)	C_g Leeward (6)	Windward pressure $p = c_e c_q q_s I_{10}$, psf (7)	Leeward suction $p = c_e c_q q_s I_w$, psf (8)	Design pressure, psf (9) = (7) + (8)	Floor-by-floor lateral loads, kips (10)
ROOF	394	2.34	2.34	+0.8	−0.5	38.94	24.34	63.28	40.51
30	381	2.32	2.34	+0.8	−0.5	38.60	24.34	62.94	80.81
29	368	2.31	2.34	+0.8	−0.5	38.44	24.34	62.78	80.49
28	355	2.29	2.34	+0.8	−0.5	38.10	24.34	62.44	80.18
27	342	2.28	2.34	+0.8	−0.5	37.94	24.34	62.28	79.85
26	329	2.26	2.34	+0.8	−0.5	37.61	24.34	61.95	79.54
25	316	2.25	2.34	+0.8	−0.5	37.44	24.34	61.78	79.21
24	303	2.23	2.34	+0.8	−0.5	37.11	24.34	61.45	78.90
23	290	2.22	2.34	+0.8	−0.5	36.94	24.34	61.28	78.57
22	277	2.21	2.34	+0.8	−0.5	36.77	24.34	61.11	78.35
21	264	1.19	2.34	+0.8	−0.5	36.44	24.34	60.78	78.04
20	251	2.17	2.34	+0.8	−0.5	36.11	24.34	60.45	77.63
19	238	2.15	2.34	+0.8	−0.5	35.78	24.34	60.12	77.19
18	225	2.13	2.34	+0.8	−0.5	35.44	24.34	59.78	76.76
17	212	2.12	2.34	+0.8	−0.5	35.28	24.34	59.62	76.45
16	199	2.10	2.34	+0.8	−0.5	34.95	24.34	59.29	76.13
15	186	2.07	2.34	+0.8	−0.5	34.44	24.34	58.78	75.60
14	173	2.05	2.34	+0.8	−0.5	34.11	24.34	58.45	75.05
13	160	2.02	2.34	+0.8	−0.5	33.61	24.34	57.95	74.52
12	147	2.00	2.34	+0.8	−0.5	33.28	24.34	57.62	74.00
11	134	1.96	2.34	+0.8	−0.5	32.61	24.34	56.95	73.35
10	121	1.93	2.34	+0.8	−0.5	32.11	24.34	56.45	72.60
9	108	1.91	2.34	+0.8	−0.5	31.79	24.34	56.13	72.08
8	95	1.88	2.34	+0.8	−0.5	31.28	24.34	55.62	71.54
7	82	1.81	2.34	+0.8	−0.5	30.12	24.34	54.46	70.48
6	69	1.78	2.34	+0.8	−0.5	29.62	24.34	53.96	69.42
5	56	1.72	2.34	+0.8	−0.5	28.62	24.34	52.96	68.45
4	43	1.63	2.34	+0.8	−0.5	27.12	24.34	51.46	66.86
3	30	1.54	2.34	+0.8	−0.5	25.63	24.34	50.00	64.95
2	17	1.41	2.34	+0.8	−0.5	23.46	24.34	47.80	72.25

Notes: $V = 90$ mph, exposure category D.
Importance factor $I_w = 1.0$.
Wind pressure q_s at 33 ft for 90 mph basic wind speed = 20.8 psf.

Because the building height is more than 200 ft according to UBC 1997, use of method 2 is not permitted. Therefore method 1, with different values of C_q for the windward and leeward walls, is used.

$C_q = 0.8$ inward pressure for windward wall (Table 1.3)

$C_q = 0.5$ outward suction for leeward wall (Table 1.3)

Windward pressures are calculated using the tabulated values of C_e for various heights. Leeward suction is calculated only at the roof level. Therefore the suction on the leeward wall remains constant for the entire building height (Table 1.5, column 6). The combined design pressures and floor-by-floor wind loads for lateral design are tabulated in Fig. 1.8a.

It should be noted that the height of 394 ft chosen for the example problem is just under the 400-ft limit, the maximum permitted by the simple procedure of the UBC. If

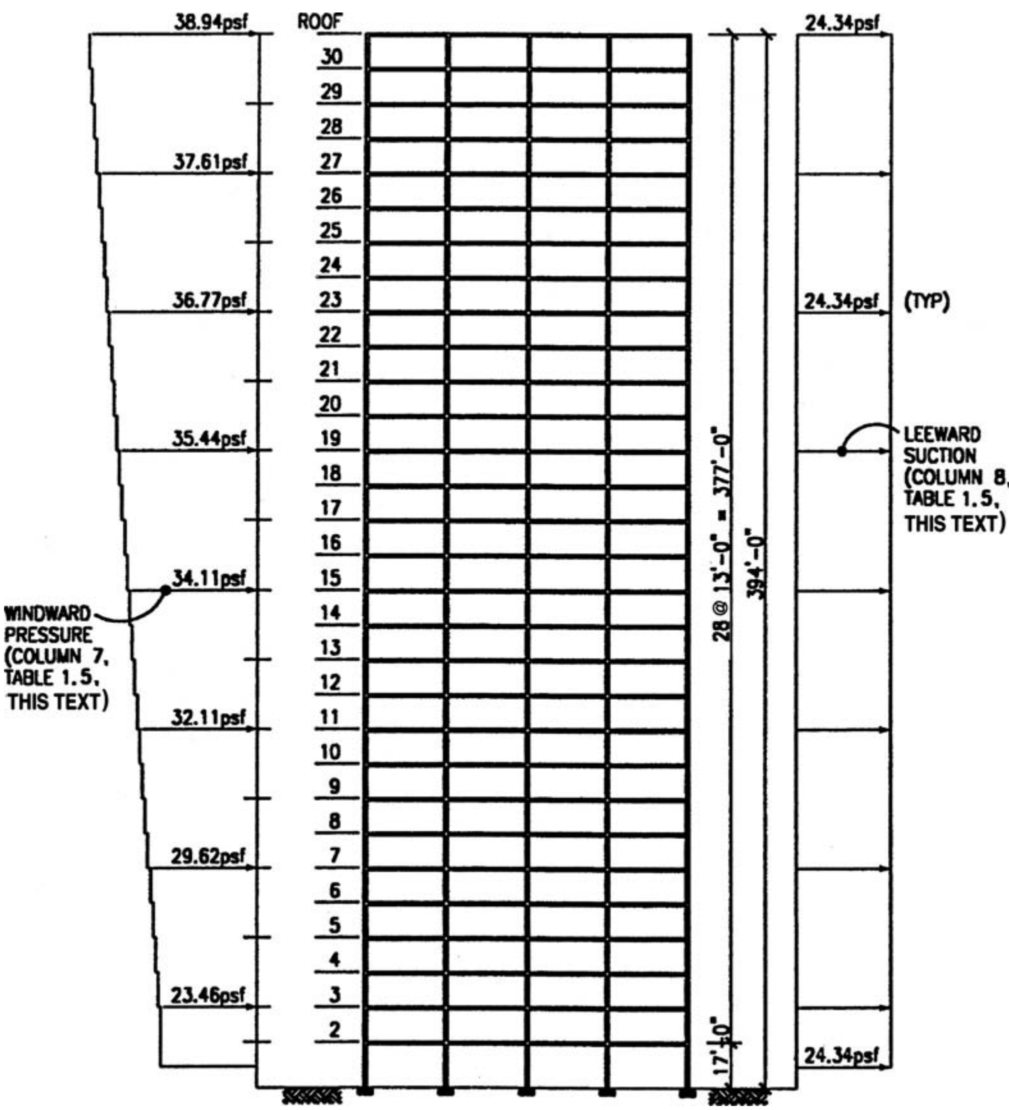


Figure 1.8a. Thirty-story building example, UBC 1997 method.

the building were taller than 400 ft, we would be required by the UBC to use other notionally accepted standards for determining the wind loads. ASCE 7-02 is one such standard discussed later in this chapter

1.4.2. ASCE 7-02: Wind Load Provisions

The full title of this ASCE standard is American Society of Civil Engineers *Minimum Design Loads for Buildings and Other Structures.* In one of its 10 sections, ASCE 7-02 provides three procedures for calculating wind loads for buildings and other structures, including the main wind-force-resisting systems and all components thereof. The designer can use Method 1, the simplified procedure, to select wind pressures directly without calculation when the building is less than 60 ft in height and meets all requirements given in Section 6.4 of the standard. Method 2 can be used for buildings and structures of any height that are regular in shape, provided the buildings are not sensitive to across-wind loading, vortex shedding, or instability due to galloping or flutter; or do not have a site for which channeling effects warrant special consideration. Method 3 is a wind-tunnel test procedure that can be used in lieu of methods 1 and 2 for any building or structure. Method 3 is recommended for buildings that possess any of the following characteristics:

- Have nonuniform shapes.
- Are flexible with natural frequencies less than 1 Hz.
- Are subject to significant buffeting by the wake of upwind buildings or other structures.
- Are subject to accelerated flow of wind by channeling or local topographic features.

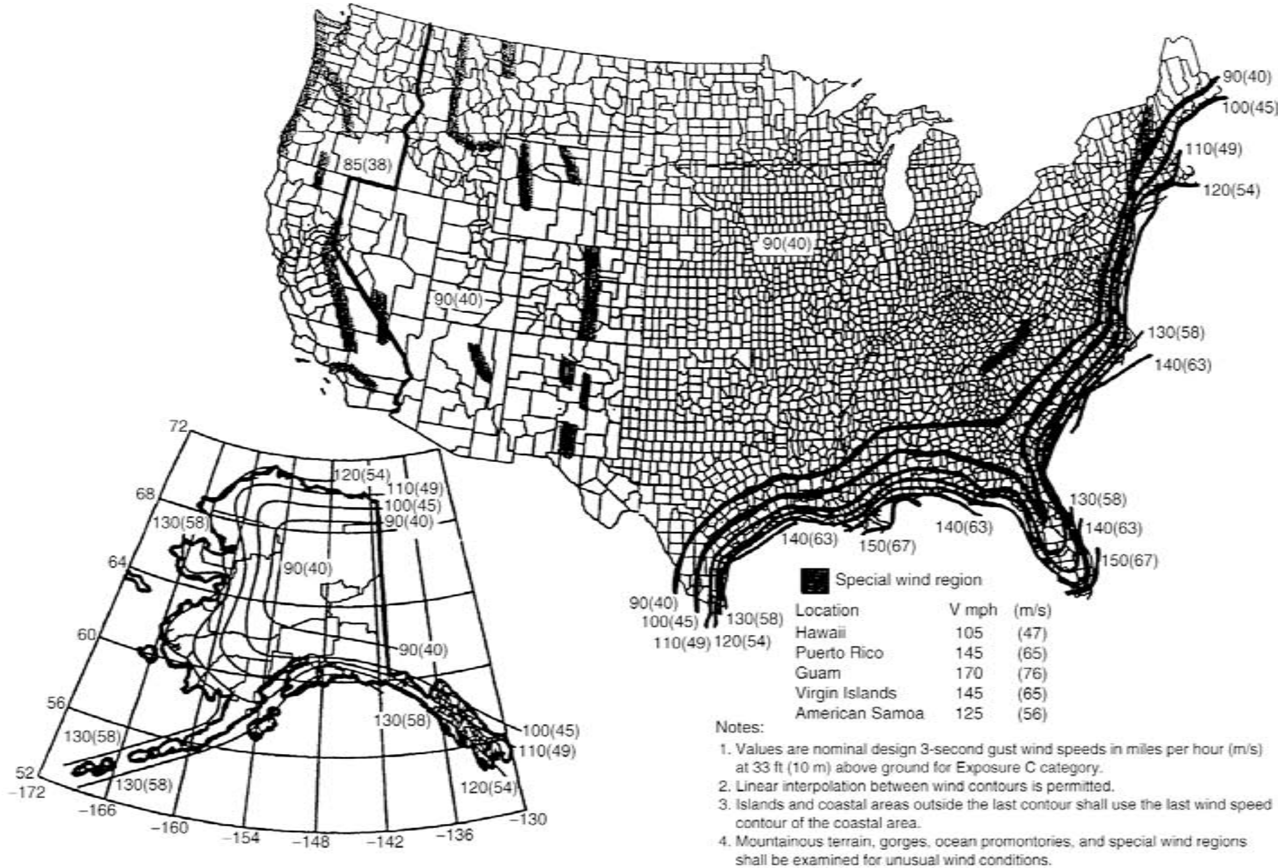
Basic wind speeds for any location in the continental United States and Alaska are shown on a map having isotachs representing a 3-sec gust speed at 33 ft (10 m) above the ground (see Fig. 1.9). For certain locations, such as Hawaii and Puerto Rico, basic wind speeds are given in a table as 105 and 145 mph (47 and 65 m/s), respectively. The map is standardized to represent a 50-year recurrence interval for exposure C topography (flat, open, country and grasslands with open terrain and scattered obstructions generally less than 30 ft (9 m) in height). The minimum basic wind speed provided in the standard is 85 mph (38 m/s). Increasing the minimum wind speed for special topographies such as mountain terrain, gorges, and ocean fronts is recommended.

The wind speed map for the United States and adjoining landmasses is based on data collected over a long period of time at weather stations located throughout the country. The maximum wind velocity expected at any location can be found simply by referring to the map.

The abandonment of the fastest-mile speed in favor of a 3-sec-gust speed first took place in the ASCE 7-1995 edition. The reasons are: 1) modern weather stations no longer measure wind speeds using the fastest-mile method; 2) a 3-sec-gust speed is closer to the sensational wind speeds often quoted by news media; and 3) it matches closely the wind speeds experienced by small buildings and by components of all buildings.

Method 1, the simplified procedure, and Method 3, the wind tunnel procedure, are not discussed here. The emphasis is on Method 2.

Method 2, the analytical procedure covered in this section, applies to a majority of buildings. It accounts for the following factors that influence the design wind forces:



(a)

Figure 1.9. Basic wind speed map, 3-sec gust wind speed: (a) map of the United States; (b) western Gulf of Mexico hurricane coastline (enlarged); (c) eastern Gulf of Mexico and southeastern U.S. hurricane coastline (enlarged); (d) mid- and north-Atlantic hurricane coastline (enlarged). (Adapted from ASCE 7-02.)

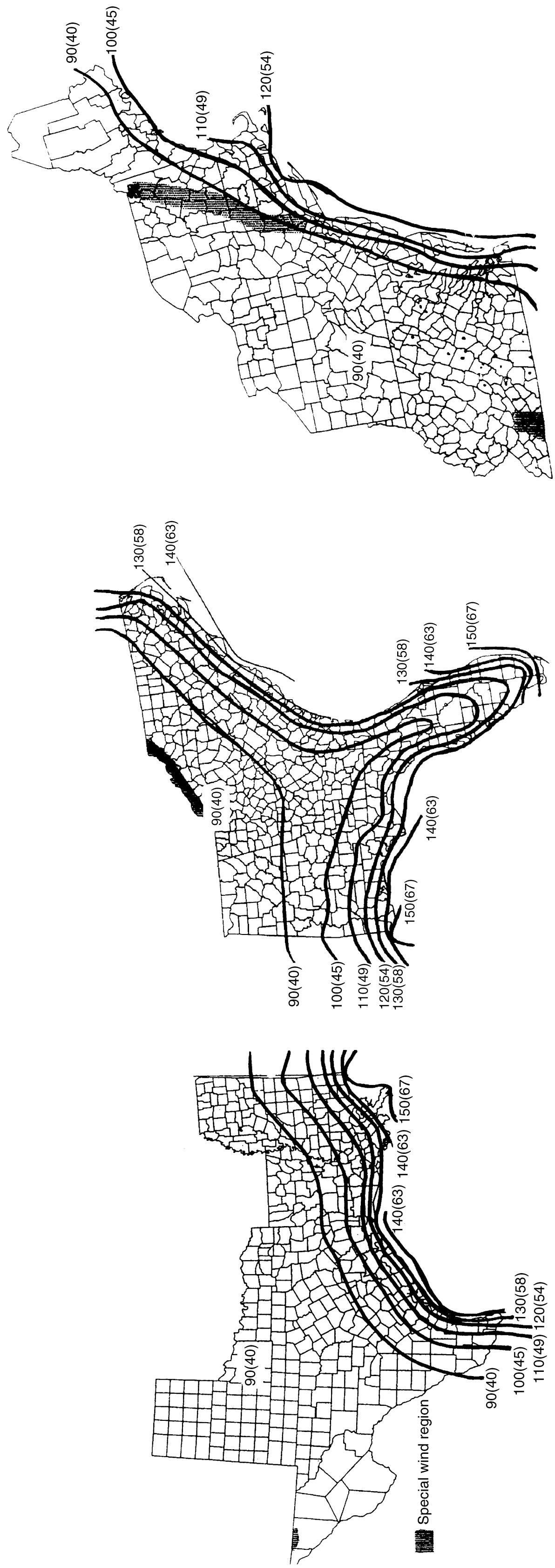


Figure 1.9. *(Continued)*

1. The basic wind speed.
2. The mean recurrence interval of the wind speed considered appropriate for the design.
3. The characteristics of the terrain surrounding the building.
4. The height at which the wind load is being determined.
5. Directional properties of the wind climate.
6. The size, geometry, and aerodynamics of the building.
7. The positions of the area acted on by the wind flow.
8. The magnitude of the area of interest.
9. The porosity of the building envelope.
10. The structural properties that may make the building susceptible to dynamic effects.
11. The speed-up effect of certain topographic features such as hills and escarpments.

1.4.2.1. Wind Loads on Main Wind-Force-Resisting System: Overview of Analytical Procedure

The analytical procedure has two steps. The first step considers the properties of the wind flow and the second accounts for the properties of the structure and its dynamic response to the longitudinal (along-wind) wind turbulence. The effects of across-wind response are not explicitly considered in the ASCE 7-02, Methods 1 and 2.

The velocity pressure at elevation z is given by the equation

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I \qquad (q_z \text{ in psf, } V \text{ in mph}) \tag{1.8}$$

The basic wind speed V corresponds to a 50-year mean recurrence interval. It represents the speed from any direction at an elevation 33 ft (10 m) aboveground in flat open country (exposure C).

The velocity pressure exposure coefficient K_z depends on the velocity, terrain roughness (i.e., exposure category), and the height aboveground.

Three exposure categories—B, C, and D—are defined. Exposure A, at one time intended for heavily built-up city centers, was deleted in the 2002 edition of ASCE 7. The exposure for each wind direction is now defined as the worst case of the two 45° sectors on either side of the wind direction being considered.

In summary, exposure B corresponds to surface roughness B typical of urban and suburban areas, exposure C to surface roughness C in flat open country, and exposure D to surface roughness D representative of flat unobstructed area and water surfaces outside hurricane-prone regions. Exposure C applies to all cases where exposures B and D do not apply. Interpolation between exposure categories is now permitted for the first time in the ASCE 7-02. Formal definitions of exposure categories are given later in Section 1.4.2.9.

The importance factor I is a factor that accounts for the degree of hazard to human life and damage to property. For category II buildings (See Table 1.7), or other structures representative of typical occupancy, $I = 1.0$. For category I buildings or other structures representing low hazard in the event of failure (e.g., agriculture facilities), $I = 0.87$ or 0.77, depending upon whether the building site is located in hurricane-prone regions. For buildings and other structures in category III posing a substantial hazard to human life in the event of failure (e.g., buildings where more than 300 people congregate in one area, and essential facilities such as fire stations), $I = 1.15$. For category IV buildings or other structures deemed as essential facilities, $I = 1.15$, the same as for category III.

The topographic factor K_{zt} is given by

$$K_{zt} = (1 + K_1 K_2 K_3)^2 \tag{1.9}$$

It reflects the speed-up effect over hills and escarpments. The multipliers K_1, K_2, and K_3 are given in Fig. 6.4 of the Standard (Figs. 1.11 and 1.11a of this text).

Wind directionality is explicitly accounted for by introducing a new factor K_d. It is no longer a component of the wind load factor. K_d varies depending upon the type of structure. Prior to introduction of exposure factor K_d, the load factor for wind was 1.3. Now it is 1.6, obtained by dividing 1.3 by the K_d factor equal to 0.85 for most buildings. Thus the new load factor = 1.3/0.85 = 1.53 rounded to 1.6.

Internal pressures and suctions on side walls and the roof of buildings do not affect the value of wind load for the main wind-force-resisting system (MWFRS). Therefore, pressures and suctions, both denoted by P_z, are calculated for the MWFRS using the following equations:

$$P_z = q_z G_f C_p \qquad \text{(for positive pressures)} \tag{1.10}$$

$$P_z = q_h C_f C_p \qquad \text{(for negative pressures)} \tag{1.11}$$

instead of the more general equation:

$$P = q(GC_p) - q_i\,(GC_{pi}) \qquad \text{[ASCE 7-02 Eq. (6.23)] (1.11a)}$$

The overall wind load is the summation of positive pressures on the windward wall, and negative pressure or suction, on the leeward wall. In the above equations G_f is a gust factor equal to 0.85 for rigid buildings, and C_p is an external coefficient, typically equal to 0.8 and 0.5 for the windward and leeward walls, respectively.

Thus, for a typical rigid buildings, the total design wind pressure at height Z above ground level is given by

$$P_z = 0.85(0.8q_z + 0.5q_h) \tag{1.12}$$

1.4.2.2. Analytical Procedure: Step-by-Step Process

Design wind pressure or suction on a building surface is given by the equation:

$$P_z = q_z \times G_f \times C_p \tag{1.13}$$

where

P_z = design wind pressure or suction, in psf, at height z, above ground level
q_z = velocity pressure, in psf, determined at height z above ground
G_f = gust effect factor, dimensionless
C_p = external pressure coefficient, which varies with building height acting as pressure (positive load) on windward face, and as suction (negative load) on nonwindward faces and roof. The values of C_p, unchanged from the previous edition of the Standard, are shown in Figs. 1.10 and 1.10a for various ratios of building width to depth.

The velocity pressure and suction q_z and q_h are given by

$$q_z = 0.00256K_zK_{zt}K_dV^2I \tag{1.14}$$

$$q_h = 0.00256K_hK_{zt}K_dV^2I \tag{1.15}$$

where

K_h and K_z = combined velocity pressure exposure coefficients (dimensionless), which take into account changes in wind speed aboveground and the nature of the terrain (exposure category B, C, or D). (See Fig. 1.11b and Table 1.6.)

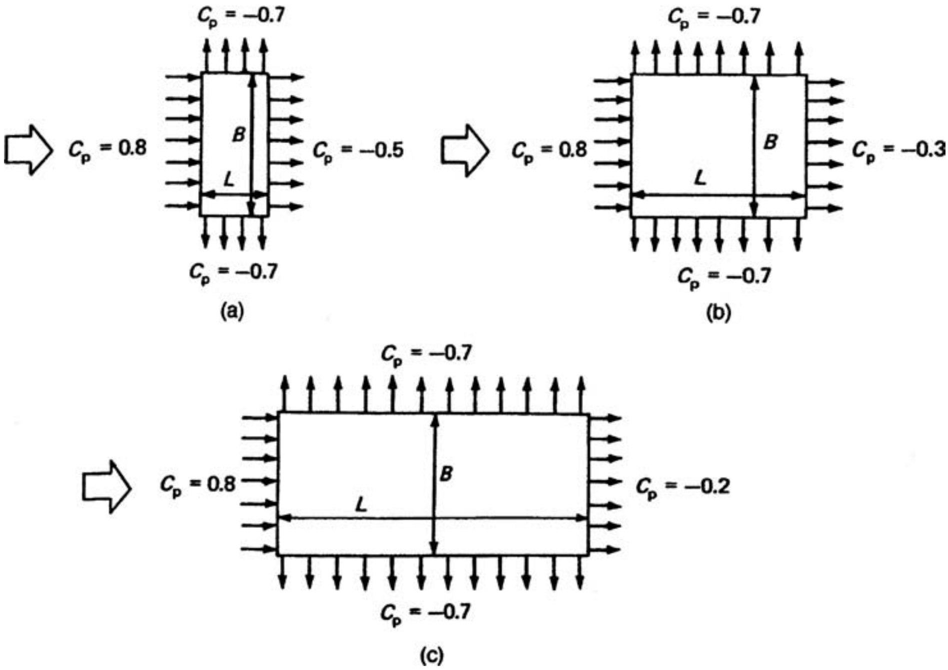


Figure 1.10. Horizontal variation of external wind pressure coefficient C_p with respect to plan aspect ratio L/B: (a) $0 \leq L/B \leq 1$; (b) $L/B = 2$; (c) $L/B > 4$. (Adapted from ASCE 7-02)

K_{zt} = topographic factor, introduced in ASCE 7-95 for the first time

I = importance factor, a dimensionless parameter that accounts for the degree of hazard to human life and damage to property (Tables 1.7 and 1.7a)

V = basic wind speed, Fig. 1.10 in miles per hour that corresponds to a 3-sec gust speed at 33-ft (10 m) aboveground, exposure category C, for a 50-year mean recurrence interval

K_d = wind directionality factor that varies from 0.85 to 0.95 depending on the structure type (Table 1.8, ASCE 7-02 Table 6.4)

The wind directionality factor identified as K_d in ASCE 7-02 accounts for two effects:

- The reduced probability of maximum winds flowing from any given direction
- The reduced probability of the maximum pressure coefficient occurring for any given direction

This factor, which was hidden in the load factors of the previous editions of the Standard, is now explicity included in the equation for velocity pressure:

$$q_z = 0.00256 K_z K_{2t} K_d V^2 I$$

The value of K_d is equal to 0.85 for most types of structures, including buildings. Therefore, q_z calculated from the previous equation is equal to 85% of the value designers were used to, prior to publication of ASCE 7-02. However, the load factors specified in ASCE 7-02 have been

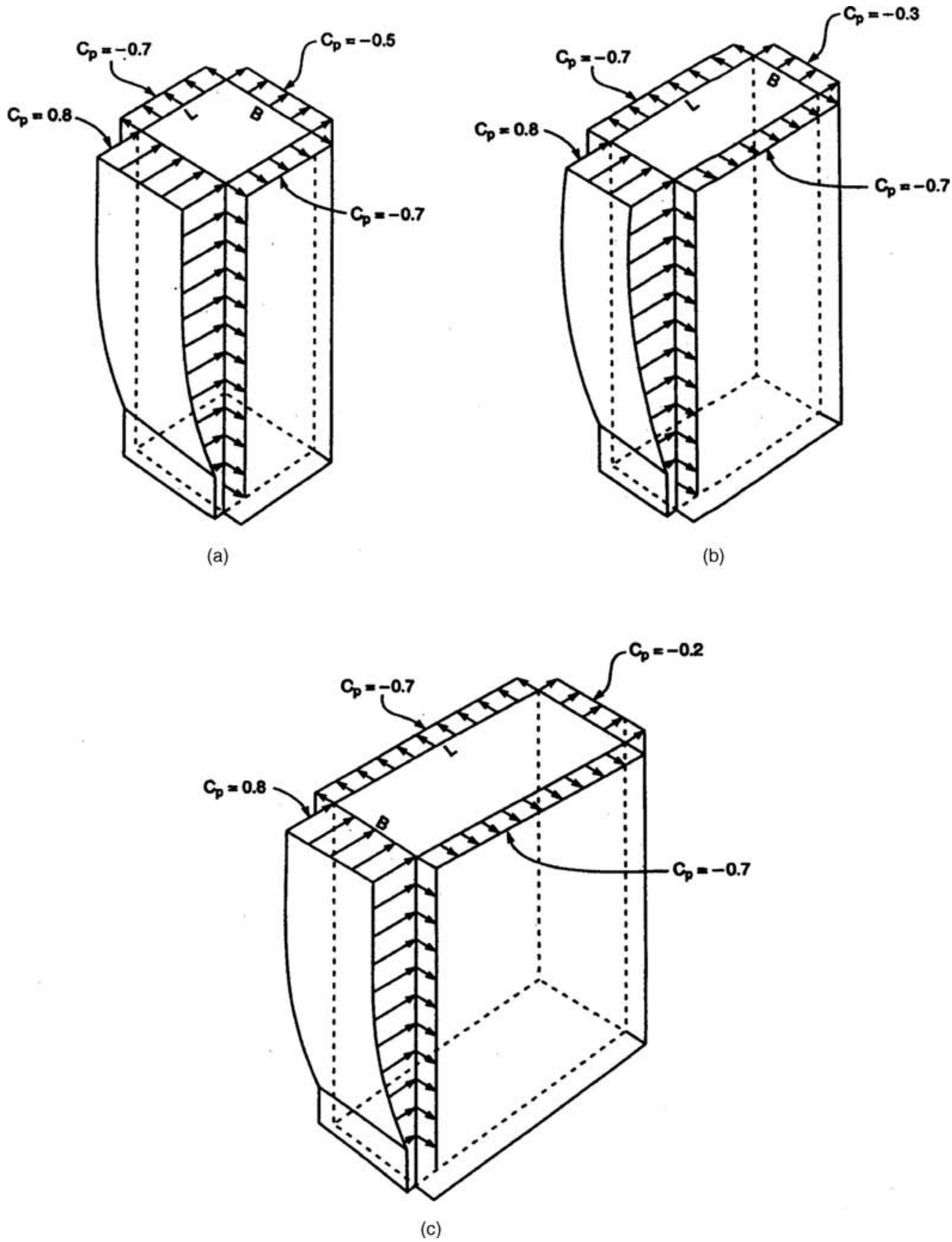
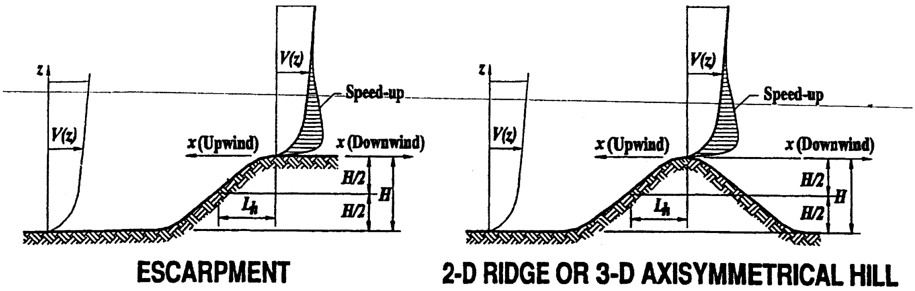


Figure 1.10a. Vertical variation of external wind pressure coefficient C_p with respect to plan aspect ratio L/B. (a) $0 \le L/B \le 1$; (b) $L/B = 2$; (c) $L/B > 4$.

adjusted upward, so the wind loads are about the same as before. Thus, for LRFD or strength design, the new load factor is 1.6, which previously was 1.3. The factor 1.6, when multiplied by the directionality factor K_d = 0.85, gives an effective load factor equal to $1.6 \times 0.85 = 1.36$ approximately equal to the previous factor of 1.3.



Topographic Multipliers for Exposure C										
	K_1 Multiplier				K_2 Multiplier			K_3 Multiplier		
H/L_h	2-D Ridge	2-D Escarp.	3-D Axisym. Hill	x/L_h	2-D Escarp.	All Other Cases	z/L_h	2-D Ridge	2-D Escarp.	3-D Axisym. Hill
0.20	0.29	0.17	0.21	0.00	1.00	1.00	0.00	1.00	1.00	1.00
0.25	0.36	0.21	0.26	0.50	0.88	0.67	0.10	0.74	0.78	0.67
0.30	0.43	0.26	0.32	1.00	0.75	0.33	0.20	0.55	0.61	0.45
0.35	0.51	0.30	0.37	1.50	0.63	0.00	0.30	0.41	0.47	0.30
0.40	0.58	0.34	0.42	2.00	0.50	0.00	0.40	0.30	0.37	0.20
0.45	0.65	0.38	0.47	2.50	0.38	0.00	0.50	0.22	0.29	0.14
0.50	0.72	0.43	0.53	3.00	0.25	0.00	0.60	0.17	0.22	0.09
				3.50	0.13	0.00	0.70	0.12	0.17	0.06
				4.00	0.00	0.00	0.80	0.09	0.14	0.04
							0.90	0.07	0.11	0.03
							1.00	0.05	0.08	0.02
							1.50	0.01	0.02	0.00
							2.00	0.00	0.00	0.00

Figure 1.11. Topographic factor K_{zt}.

Notes:

1. For values of H/L_h, x/L_h, and z/L_h other than those shown, linear interpolation is permitted.
2. For $H/L_h > 0.5$, assume $H/L_h = 0.5$ for evaluating K_1 and substitute 2H for L_h for evaluating K_2 and K_3.
3. Multipliers are based on the assumption that wind approaches the hill or escarpment along the direction of maximum slope.
4. Notation:
   - H: Height of hill or escarpment relative to the upwind terrain, in feet (meters).
   - L_h: Distance upwind of crest to where the difference in ground elevation is half the height of hill or escarpment, in feet (meters).
   - K_1: Factor to account for shape of topographic feature and maximum speed-up effect.
   - K_2: Factor to account for reduction in speed-up with distance upwind or downwind of crest.
   - K_3: Factor to account for reduction in speed-up with height above local terrain.
   - x: Distance (upwind or downwind) from the crest to the building site, in feet (meters).
   - z: Height above local ground level, in feet (meters).
   - μ: Horizontal attenuation factor.
   - γ: Height attenuation factor.

(From ASCE 7-02, Fig. 6.4.)

For allowable stress design, the ASCE 7-02 load factor is still equal to 1.0. However, since one-third increase in allowable stress is not permitted, the overall effect is the same as before.

The basic wind speed is converted to the design speed at any height z for a given exposure category by using the velocity exposure coefficient K_z, evaluated at height z.

Parameters for Speed-Up Over Hills and Escarpments						
Hill Shape	$K_1/(H/L_h)$ Exposure			γ	μ	
	B	C	D		Upwind of Crest	Downwind of Crest
2-Dimensional ridges (or valleys with negative H in $K_1/(H/L_h)$)	1.30	1.45	1.55	3	1.5	1.5
2-Dimensional escarpments	0.75	0.85	0.95	2.5	1.5	4
3-Dimensional axisym. hill	0.95	1.05	1.15	4	1.5	1.5

Figure 1.11a. Topographic factor K_{zt} based on equations

$$K_{zt} = (1 + K_1K_2K_3)^2$$

K_1 determined from table

$$K_2 = \left(1 - \frac{|x|}{\mu L_h}\right)$$

$$K_3 = e^{-\gamma z/L_h}$$

(From Fig. 6.4 in ASCE 7-02.)

K_z is given by

$$K_z = 2.01\left(\frac{z}{z_g}\right)^{2/\alpha} \qquad \text{(for 15 ft} < z < z_g) \tag{1.16}$$

$$K_z = 2.01\left(\frac{15}{z_g}\right)^{2/\alpha} \qquad \text{(for } z < \text{15 ft)} \tag{1.17}$$

where z_g is the gradient height above which the frictional effect of terrain becomes negligible. It varies with the characteristics of the ground surface irregularities at the building site that arise as a result of natural topographic variations as well as human-made features. In the ASCE 7-02 standard, the K_z expressions are unchanged from ASCE 7-98. However, interpolation of the K_z values between standard exposures is permitted for the first time in the 2002 edition of ASCE 7.

The power coefficient α (Table 1.9) is the exponent for velocity increase in height, and has values of 7.0, 9.5, and 11.5, respectively, for exposure B, C, and D. The values of K_z for various exposures up to a height of 500 ft (152.6 m) are given in ASCE 7-02. An extended version up to a height of 1500 ft (457 m) is given in Table 1.6 and in Fig. 1.11b. The values of the gradient height z_z, given in ASCE 7.02, are of course identical to those given in the previous ASCE-7 editions. This should be obvious because the gradient height z_z for a given exposure does not vary with the reference wind speed used in design. As with the previous ASCE-7 editions, the values of K_z are assumed to be constant for heights less than 15 ft (4.6 m), and for heights greater than the gradient height z. The variation of velocity pressure q_z for exposure categories B, C, and D is given in Fig. 1.12.

1.4.2.3. Wind Speed-Up Over Hills and Escarpments: K_{zt} Factor

The topographic factor K_{zt} accounts for the effect of isolated hills or escarpments located in exposures B, C, and D. Buildings sited on the upper half of an isolated hill

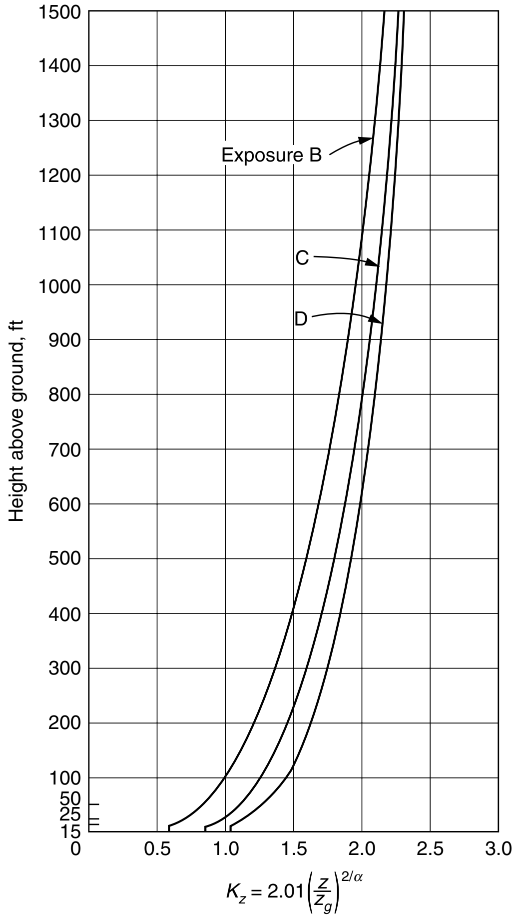


Figure 1.11b. Velocity pressure exposure coefficients K_h and K_z. The graphical representation of K_z values is given in Table 1.6. (From ASCE 7-02.)

or escarpment may experience significantly higher wind speeds than buildings situated on level ground. To account for these higher wind speeds, the velocity pressure exposure coefficients are multiplied by a topographic factor K_{zt}, determined from the three multipliers K_1, K_2, and K_3 (Fig. 1.11), K_1 is related to the shape of the topographic feature and the maximum speed-up with distance upwind or downward of the crest, K_2 accounts for the reduction in speed-up with distance upwind or downwind of the crest, and K_3 accounts for the reduction in speed-up with height above the local ground surface.

1.4.2.4. Design Wind Load Cases

This requirement, first introduced in ASCE 7-95 under the heading "Full and Partial Loading," was for including the torsional response of buildings. Now the design requirements have become more stringent under a new heading, "Design Wind Load Cases."

Recent wind tunnel research has shown that torsional load requirements previously given in ASCE 7-98 often grossly underestimated the actual torsion on a building under wind, even those that are symmetric in geometric form and stiffness. This torsion is a result of nonuniform pressures on the different faces of the building as wind flows around

TABLE 1.6 Velocity Pressure Exposure Coefficients, K_z[a,b]

Height above ground level, z		Exposure category		
ft	(m)	B	C	D
0–15	(0–4.6)	0.57	0.85	1.03
20	(6.1)	0.62	0.90	1.08
25	(7.6)	0.66	0.94	1.12
30	(9.1)	0.70	0.98	1.16
40	(12.2)	0.76	1.04	1.22
50	(15.2)	0.81	1.09	1.27
60	(18)	0.85	1.13	1.31
70	(21.3)	0.89	1.17	1.34
80	(24.4)	0.93	1.21	1.38
90	(27.4)	0.96	1.24	1.40
100	(30.5)	0.99	1.26	1.43
120	(36.6)	1.04	1.31	1.48
140	(42.7)	1.09	1.36	1.52
160	(48.8)	1.13	1.39	1.55
180	(54.9)	1.17	1.43	1.58
200	(61.0)	1.20	1.46	1.61
250	(76.2)	1.28	1.53	1.68
300	(91.4)	1.35	1.59	1.73
350	(106.7)	1.41	1.64	1.78
400	(121.9)	1.47	1.69	1.82
450	(137.2)	1.52	1.73	1.86
500	(152.4)	1.56	1.77	1.89
550	(167.6)	1.61	1.81	1.93
600	(182.9)	1.65	1.85	1.96
650	(191.1)	1.69	1.88	1.98
700	(213.3)	1.72	1.91	2.01
750	(228.6)	1.76	1.93	2.03
800	(243.8)	1.79	1.96	2.06
850	(259.1)	1.82	1.99	2.08
900	(274.3)	1.85	2.01	2.10
950	(289.5)	1.88	2.03	2.12
1000	(304.8)	1.91	2.06	2.14
1050	(320)	1.93	2.08	2.16
1100	(335.3)	1.96	2.10	2.17
1150	(350.5)	1.99	2.12	2.19
1200	(365.7)	2.01	2.14	2.21
1250	(381)	2.03	2.15	2.22
1300	(396.2)	2.06	2.17	2.24
1350	(411.5)	2.08	2.19	2.26
1400	(426.7)	2.10	2.21	2.27
1450	(441.9)	2.12	2.22	2.28
1500	(457.2)	2.14	2.24	2.29

[a] The velocity pressure exposure coefficient K_z may be determined from the following formula:
For $15 \text{ ft} \le z \le z_g$, $K_z = 2.01\ (z/z_g)^{2/\alpha}$.
For $z < 15$ ft, $K_z = 2.01\ (15/z_g)^{2/\alpha}$.
[b] All main wind force resisting systems in buildings and in other structures except those in low-rise buildings.
(Adapted from Table 6.3 of ASCE 7-02.)

TABLE 1.7 Classification of Buildings for Flood, Wind, Snow, Earthquake, and Ice Loads

Nature of occupancy	Category
Buildings that represent a low hazard to human life in the event of failure including, but not limited to: Agricultural facilities Certain temporary facilities Minor storage facilities	I
All buildings except those listed in Categories I, III, and IV	II
Buildings that represent a substantial hazard to human life in the event of failure including, but not limited to Buildings where more than 300 people congregate in one area. Buildings with day care facilities with capacity greater than 150. Buildings with elementary school or secondary school facilities with capacity greater than 250. Buildings with a capacity greater than 500 for colleges or adult education facilities. Health care facilities with a capacity of 50 or more resident patients but not having surgery or emergency treatment facilities. Jails and detention facilities. Power-generating stations and other public utility facilities not included in Category IV. Buildings not included in Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, chemicals, and waste, or explosives) containing sufficient quantities of hazardous materials to be dangerous to the public if released. Buildings containing hazardous materials shall be eligible for classification as Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the hazardous material does not pose a threat to the public.	III
Buildings designated as essential facilities including, but not limited to Hospitals and other health care facilities having surgery or emergency treatment facilities. Fire, rescue, ambulance, and police stations and emergency vehicle garages. Designated earthquake, hurricane, or other emergency shelters. Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response. Power-generating stations and other public utility facilities required in an emergency. Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, cooling towers, electrical substation structures, fire water storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for operation of Category IV structures during an emergency. Aviation control towers, air traffic control centers, and emergency aircraft hangars. Water storage facilities and pump structures required to maintain water pressure for fire suppression. Buildings and other structures having critical national defense functions.	IV

(*Continued*)

TABLE 1.7 (Continued)

Nature of occupancy	Category
Buildings (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, chemicals, and waste, or explosives) containing extremely hazardous materials where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction.	IV
Buildings containing extremely hazardous materials shall be eligible for classification as Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction that a release of the extremely hazardous material does not pose a threat to the public. This reduced classification shall not be permitted if the buildings also function as essential facilities.	

(From ASCE 7-02 Table 1.1.)

TABLE 1.7a Importance Factor, I (Wind Loads)

Category[a]	Non-hurricane-prone regions and hurricane-prone regions with $V = 85–100$ mph and Alaska	Hurricane-prone regions with $V > 100$ mph
I	0.87	0.77
II	1.00	1.00
III	1.15	1.15
IV	1.15	1.15

[a] The building and structure classification categories are listed in Table 1.7, ASCE 7-02, Table 1.1.
(From ASCE 7-02 Table 6.1.)

TABLE 1.8 Wind Directionality Factor K_d

Structure type	Directionality factor K_d^{a}
Buildings	
Main wind-force-resisting system	0.85
Components and cladding	0.85
Arched roofs	0.85
Chimneys, tanks, and similar structures	
Square	0.90
Hexagonal	0.95
Round	0.95
Solid signs	0.85
Open signs and lattice framework	0.85
Trussed towers	
Triangular, square, rectangular	0.85
All other cross sections	0.95

[a] Directionality factor K_d shall only be applied when used in conjunction with load combinations specified in ASCE 7-02 Sections 2.3 and 2.4.
(From ASCE 7-02 Table 6.4.)

TABLE 1.9 Terrain Exposure Constants

Exposure	α	z_g (ft)	$\hat{a}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	ℓ(ft)	$\bar{\varepsilon}$	z_{min} (ft)[a]
B	7.0	1200	1/7	0.84	1/4.0	0.45	0.30	320	1/3.0	30
C	9.5	900	1/9.5	1.00	1/6.5	0.65	0.20	500	1/5.0	15
D	11.5	700	1/11.5	1.07	1/9.0	0.80	0.15	650	1/8.0	7

[a] z_{min} = minimum height used to ensure that the equivalent height $\bar{z}$ is the greater of $0.6h$ or z_{min}. For buildings with $h \leq z_{min}$, $\bar{z}$ shall be taken as z_{min}.
(From ASCE 7-02, Table 6.2.)

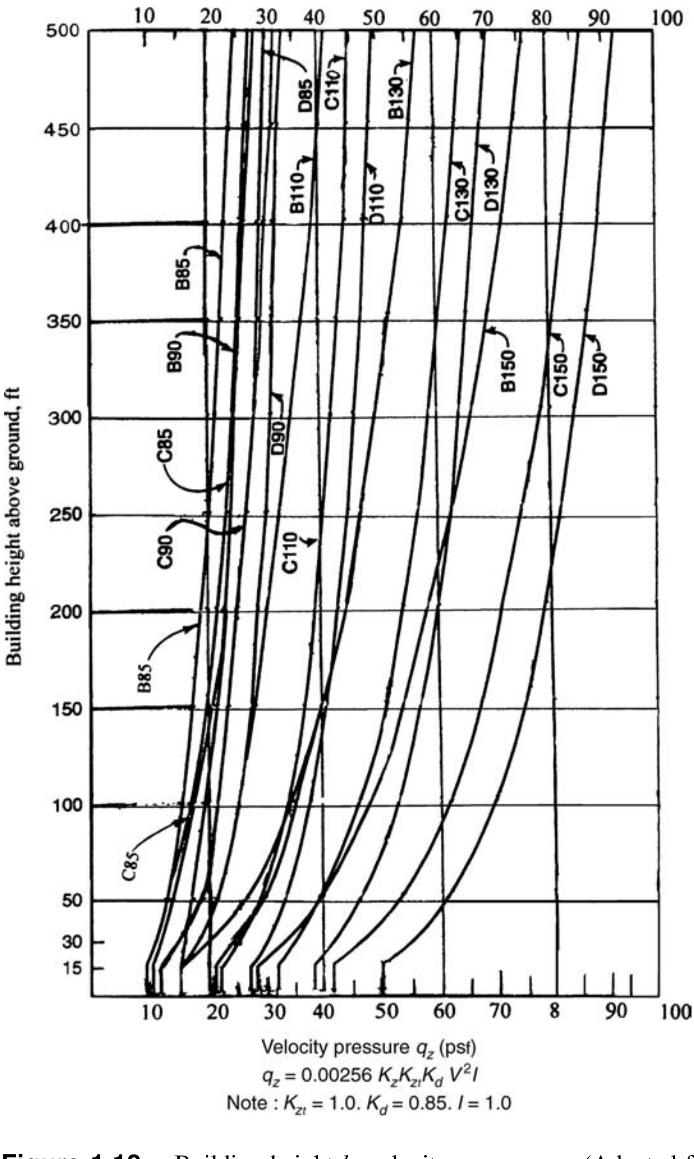


Figure 1.12. Building height h, velocity pressure q_z. (Adapted from ASCE 7-02.)

the building. These irregular pressures are the results of interference effects of nearby buildings and terrain, and dynamic effects on more flexible buildings. The ASCE 7-02 requirements represent of square and rectangular buildings with aspect ratios up to about 2.5. They may not cover all cases, even for symmetric and common building shapes where larger torsions have been observed. Therefore, the designer may wish to apply this level of eccentricity at full, rather than reduced, wind loading for certain more critical buildings, even though it is not required by the Standard.

In buildings with unusual structural systems, such as the one used for the City Corp. Tower in New York, more severe loading can occur when the resultant wind load acts diagonally to the building. To account for this effect and the fact that many buildings exhibit maximum response in the across-wind direction, a structure should be capable of resisting 75% of the design wind load applied simultaneously along each principal axis, as required by case 3 in Fig. 6.9 of ASCE 7-02.

For flexible buildings, dynamic effects can increase torsional loading. Additional torsional loading can occur because of eccentricity between the elastic shear center and the center of mass at each level of the structure. The new Eq. (1.18) given below accounts for this effect.

$$e = \frac{e_Q + 1.7I_{\bar{z}}\sqrt{(g_Q Q e_Q)^2 + (g_R R e_R)^2}}{1.7I_{\bar{z}}\sqrt{(g_Q Q)^2 + (g_R R)^2}} \tag{1.18}$$

where

e_Q = eccentricity e as determined for rigid structures in Fig. 6.9 of ASCE 7-02

e_R = distance between the elastic shear center and center of mass of each floor

$I_{\bar{z}}$ g_Q, Q, g_R shall be as defined in 6.5.8 of ASCE 7-02

The sign of the eccentricity e shall be plus or minus, whichever causes the more severe load effect.

The eccentricity e for flexible structures shall be considered for each principal axis (e_X, e_Y).

The eccentricity is used for calculating torsional moment M_T per unit height acting about a vertical axis of the building. The designer is referred to ASCE 7-02, Fig. 6.9 for additional information.

1.4.2.5. Gust Effect Factor

The gust effect factor accounts for additional dynamic amplification of loading in the along-wind direction due to wind turbulence and structure interaction. It does not include allowances for across-wind loading effects, vortex shedding, instability due to galloping or flutter, or dynamic torsional effects. Buildings susceptible to these effects should be designed using wind tunnel results.

Three methods are permitted for calculating G. The first two are for rigid structures and the third is for flexible or dynamically sensitive structures.

Gust Effect Factor *G* for Rigid Structure: Simplified Method. For rigid structures (defined as those having a natural frequency of vibration greater than 1 Hz), the engineer can use a single value of $G = 0.85$, irrespective of exposure category.

Gust Effect Factor *G* for Rigid Structure: Improved Method. As an option to using $G = 0.85$ the designer may calculate a more accurate value by including specific features of the wind environment at the building site. The procedure is as follows:

The gust effect factor G is given by

$$G = 0.925\left(\frac{(1 + 1.7g_Q I_{\bar{z}} Q)}{1 + 1.7g_v I_{\bar{z}}}\right) \tag{1.19}$$

$$I_{\bar{z}} = c(33/\bar{z})^{1/6} \tag{1.20}$$

where $I_{\bar{z}}$ = the intensity of turbulence at height $\bar{z}$ and where $\bar{z}$ = the equivalent height of the structure defined as 0.6 h but not less than $z_{\min}$ for all building heights h. $z_{\min}$ and c are listed for each exposure in Table 1.9; g_Q and g_v shall be taken as 3.4. The background response Q is given by

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B + h}{L_{\bar{z}}}\right)^{0.63}}} \tag{1.21}$$

where B, h are defined in Section 6.3; and $L_{\bar{z}}$ = the integral length scale of turbulence at the equivalent height given by

$$L_{\bar{z}} = l(\bar{z}/33)^{\bar{\varepsilon}} \tag{1.22}$$

in which l and $\bar{\varepsilon}$ are constants listed in Table 6.2 of ASCE 7-02 (Table 1.9 of this chapter).

Gust Effect Factor G_f for Flexible or Dynamically Sensitive Structures. Flexible buildings are those that have a frequency less than 1 Hz (i.e., buildings with a fundamental period greater than 1 sec.). Included are buildings with heights in excess of four times their least plan dimension.

The formula for calculating G_f is as follows:

$$G_f = 0.925\left(\frac{1 + 1.7I_{\bar{x}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7g_v I_{\bar{z}}}\right) \tag{1.23}$$

where g_Q and g_v shall be taken as 3.4 and g_R is given by

$$g_R = \sqrt{2\ln(3{,}600 n_1)} + \frac{0.577}{\sqrt{2\ln(3{,}600 n_1)}} \tag{1.24}$$

and where R, the resonant response factor, is given by

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} \tag{1.25}$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} \tag{1.26}$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\overline{V}_{\bar{z}}} \tag{1.27}$$

$$R_\ell = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta}) \quad \text{for } \eta > 0 \tag{1.28}$$

$$R_\ell = 1 \text{ for } \eta = 0 \tag{1.29}$$

where the subscript ℓ in Eq. 1.28 shall be taken as h, B, and L respectively, and where

n_1 = building natural frequency

$R_\ell = R_h$ setting $\eta = 4.6 n_1 h/\overline{V}_{\bar{z}}$

$R_\ell = R_B$ setting $\eta = 4.6 n_1 B/\overline{V}_{\bar{z}}$
$R_\ell = R_L$ setting $\eta = 15.4 n_1 L/\overline{V}_{\bar{z}}$
β = damping ratio, percent of critical h, B, L are defined in Section 6.3
$\overline{V}_{\bar{z}}$ = mean hourly wind speed (ft/s) at height $\bar{z}$ determined from Eq. 1.30,

$$\overline{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) \tag{1.30}$$

where $\bar{b}$ and $\bar{\alpha}$ are constants listed in Table 1.9 and V is the basic wind speed in mph

1.4.2.6. Along-Wind Response

A typical modern building that is light and flexible is more prone to dynamic motions than its earlier counterpart with heavy masonry cladding and partition walls. Dynamic motions are those cause by time-dependent forces such as seismic accelerations and short-period wind loads, or gusts. Building dynamic excitations during earthquakes, insofar as perception of motion by the occupants is concerned, is irrelevant because occupants are thankful to have survived the trauma and are less prone to complain about motion perception. However, the sentiment when estimating peak dynamic response of buildings to fluctuating wind forces is quite different, because windstorms occur more frequently and are not as traumatic as earthquakes. Consequently, it is necessary to determine whether the building is prone to wind-induced problems related to the comfort of the occupants.

When considering the response of a tall building to wind gusts, both along-wind and across-wind responses must be considered. These arise from different effects of wind, the former being primarily due to buffeting effects caused by turbulence; the latter being primarily due to alternate-side vortex shedding. The cross-wind response may be of particular importance with regard to the comfort of the occupants because it is likely to exceed along-wind accelerations if the building is slender about both axes, such that the geometric ratio $\sqrt{WD/H}$ is less than one-third, where W and D are the across- and along-wind plan dimensions, and H is the building height.

The most important criterion for verifying the comfort of the building's occupants is the peak acceleration they experience. It is thus important to be able to estimate the probable maximum accelerations in both the along-wind and across-wind directions. ASCE 7-02 gives a method for predicting along-wind responses, including peak acceleration, but does not provide a procedure for estimating across-wind response. However, the National Building Code of Canada (NBCC), addressed presently in Section 1.4.3, provides such a procedure.

Maximum Along-Wind Displacement. The maximum along-wind displacement $X_{max}(z)$ as a function of height above the ground surface is given by

$$X_{max}(z) = \frac{\phi(z)\rho B h C_{fx} \hat{V}_{\bar{z}}^2}{2m_1(2\pi n_1)^2} KG \tag{1.31}$$

where

$\phi(z)$ = the fundamental model shape = $(z/h)^{\xi}$
ξ = the mode exponent
ρ = air density
C_{fx} = mean alongwind force coefficient

$$m_1 = \text{modal mass} = \int_o^h \mu(z)\phi^2(z)dz \tag{1.32}$$

where $\mu(z)$ = mass per unit height.

$$K = (1.65)^{\hat{\alpha}}/(\hat{\alpha} + \xi + 1) \tag{1.33}$$

$\overline{V}_{\bar{z}}$ is the 3-sec gust speed at height $\bar{z}$. This can be evaluated as $\hat{V}_{\bar{z}} = \hat{b}(z/33)^{\hat{\alpha}} V$, where V is the 3-sec gust speed in exposure C at the reference height (obtained from Fig. 1.10), $\hat{b}$ and $\hat{\alpha}$ are given in Table 1.9.

RMS Along-Wind Acceleration. The rms along-wind acceleration $\sigma_{\ddot{x}}(z)$ as a function of height above the ground surface is given by

$$\sigma_{\hat{x}}(z) = \frac{0.85\phi(z)\rho BhC_{fx}\overline{V}_{\bar{z}}^2}{m_1} I_{\bar{z}}KR \tag{1.34}$$

where $\overline{V}_{\bar{z}}$ is the mean hourly wind speed at height $\bar{z}$, ft/s

$$\overline{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V \tag{1.35}$$

where $\bar{b}$ and $\bar{\alpha}$ are defined in Table 1.7a.

Maximum Along-Wind Acceleration. The maximum along-wind acceleration as a function of height above the ground surface is given by

$$\ddot{X}_{\max}(z) = g_{\ddot{x}}\sigma_{\ddot{x}}(z) \tag{1.36}$$

$$g_{\ddot{x}} = \sqrt{2\ln(n_1 T)} + \frac{0.5772}{\sqrt{2\ln(n_1 T)}} \tag{1.37}$$

where T = the length of time over which the acceleration is computed, usually taken to be 3600 s to represent 1 h.

1.4.2.7. Worksheet for Calculation of Gust Effect Factor G_f and Along-Wind Displacement and Acceleration

The formulas given in the ASCE Standard are in a concise format. They may be harder to use without rewriting many of the formulas in an expanded manner. Therefore, to make the calculation of G_f somewhat less forbidding, the ASCE formulas have been expanded and given in a worksheet format in the following section. The worksheets also include formulas for calculating along-wind response, given in the commentary section of the Standard. Also included are comments that may be helpful in going through various calculations.

Worksheet for Calculating Gust Effect Factor, Along-Wind Displacement, and Accelerations

ASCE 7-02 formulas	Commentary
V = wind speed in ft/s $= V_{\text{mph}} \times 1.467$	V from wind map, converted from mph to ft/s
$\overline{Z} = 0.6$ h, but not less than $z_{\min}$	$Z_{\min}$ from Table 6.4 h = building height, ft
$I_{\bar{z}} = C\left(\frac{33}{\bar{z}}\right)^{1/6}$	C from Table 6.4
$L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\varepsilon}$	l and ε from Table 6.4

ASCE 7-02 formulas	Commentary
$Q = \sqrt{\dfrac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$	B = building width perpendicular to wind h = building height Q = background response, a term used in random vibration theory.
$\bar{V}_{\bar{z}} = \bar{b}\left(\dfrac{\bar{z}}{33}\right)^{\bar{\alpha}} V$	$\bar{b}$ and $\bar{\alpha}$ from Table 6.4
$\hat{V}_{\bar{z}} = \hat{b}\left(\dfrac{\bar{z}}{33}\right)^{\hat{\alpha}}$	$\hat{b}$ and $\hat{\alpha}$ from Table 6.4
$N_1 = \dfrac{n_1 L_z}{\bar{V}_{\bar{z}}}$	n_1 = natural frequency of the building
$R_n = \dfrac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}}$	R_n is a parameter required for calculating R^2
$\eta_b = \dfrac{4.6 n_1 \mathrm{B}}{\bar{V}_{\bar{z}}}$ $R_B = \dfrac{1}{\eta} - \dfrac{1}{2\eta_b^2}(1 - e^{-2\eta_b})$ for $\eta_b > 0$ $R_B = 1,$ for $\eta_b = 0$	R_b is a parameter required for calculating R^2
$\eta_h = \dfrac{4.6 n_1 h}{\bar{V}_{\bar{z}}}$ $R_h = \dfrac{1}{\eta_h} - \dfrac{1}{2\eta_h^2}(1 - e^{-2\eta_h})$ for $\eta_h > 0$ $R_h = 1,$ for $\eta_h = 0$	R_h is a parameter required for calculating R^2
$\eta_L = \dfrac{15.4 n_1 L}{\bar{V}_{\bar{z}}}$ $R_L = \dfrac{1}{\eta_L} - \dfrac{1}{2\eta_L^2}(1 - e^{-2\eta_L})$ for $\eta_L > 0$ $R_L = 1$ for $\eta_L = 0$	R_L is a parameter required for calculating R^2
$R^2 = \dfrac{1}{\beta} R_n R_h R_B (0.53 + 0.47\, R_L)$ $G = 0.925\left(\dfrac{1 + 1.7\, I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v\, I_{\bar{z}}}\right)$	β = damping ratio, percent of critical g_Q = peak factor for background response g_R = peak factor for resonance response G = gust factor g_V = peak factor for wind response $g_Q = g_V$ always taken = 3.4
$k = \dfrac{1.65^{\bar{\alpha}}}{(\bar{\alpha} + \xi + 1)}$	ξ = mode exponent
$g_{\ddot{x}} = 2 \ln(n_1 T) + \dfrac{0.5772}{\sqrt{2 \ln(n_1 T)}}$	ln means logarithm to base e = 2.71.
m_1 = modal mass = $\displaystyle\int_0^h \mu_z Q_z^2 dz$ $= \mu_z \times \dfrac{h}{3}$ as shown below.	μ_z = mass per unit height, slugs/ft h = building height

For a linear first-mode shape, $Q = x/h$, where x is the displacement of the building at top. If we assume that μ_z is constant for the full height of the building (meaning that the building is uniform with a constant density = μ slugs/ft^3), the modal mass is given by

$$m = \int_o^h \mu_z Q^2 dz$$

$$= \mu_z \int_o^h \frac{x^2}{h^2} dz$$

$$= \mu_z \left[\frac{x^3}{3h^2} \right]_o^h$$

$$= \mu_z \times \frac{h}{3}$$

Maximum Along-Wind Acceleration

$$X_{\max(z)} = \frac{\phi_{(2)}\rho B h C_{fx} \hat{V}_{\bar{z}}^2}{2m_1(2\pi n_1)^2} KG$$

At $z = h$, $X_{\max(h)}$ gives the maximum lateral load deflection at top

ρ = air density = 0.0024 slugs/ft^3

C_{fx} = mean along-wind force coefficient, typically equal to 1.3.

$$g_{\ddot{x}} = \sqrt{2 \ln (n_1 T)} + \frac{0.5772}{\sqrt{2 \ln (n_1 T)}}$$

T = time in seconds over which acceleration is computed, usually taken to be 1 hour = 3600 seconds.

$$\sigma_{\ddot{x}(z)} = \frac{0.85\,\phi_z \rho B h C_{fx} \bar{V}_{\bar{z}}^2}{m_1} I_{\bar{z}} K R$$

$\sigma_{\ddot{x}(z)}$ = the root-mean-square along-wind acceleration above the ground surface

$$\ddot{X}_{\max(z)} = g_{\ddot{x}} \sigma_{\ddot{x}(z)}$$

$\ddot{X}_{\max(z)}$ = the maximum along-wind acceleration as a function of height above the ground surface.

$$\ddot{X}_{\max(h)} = g_{\ddot{x}} \sigma_{\ddot{x}(h)}$$

$\ddot{X}_{\max(h)}$ = the maximum acceleration at the building top—the item of interest. If greater than 20 milli-g, further investigation is recommended.

1.4.2.8. Design Examples: ASCE 7-02

Several examples are given in this section. The first demonstrates calculations for gust effect factor G for a rigid structure. The second is for a flexible structure. The third shows calculations for design wind pressures for a 450-ft tall building using the graphs given in Fig. 1.12. Also given in this example are calculations for gust effect factor and along-wind response, using the worksheets given in Section 1.4.1.7.

In the fourth and fifth examples, design wind pressure calculations for a 10- and 30-story building are given. The final example gives a comparison of gust effect factor and along-wind displacements and acceleration for four randomly chosen buildings.

Calculations for Gust Effect Factor *G*: Rigid Structure.
Given. A 10-story concrete building with the following characteristics:
Height h = 112 ft
Width perpendicular to wind, B = 90 ft

Exposure category = C
Basic wind speed $V = 100$ mph
Topographic factor $K_{zt} = 1.0$
Building depth parallel to wind, $L = 95$ ft
Building natural frequency $n_1 = 1.1$ Hz

Required. Gust effect factor G, using the improved method.
Solution.

ASCE 7-02 formulas	Commentary
$\bar{z} = 0.6 \times h = 0.6 \times 112 = 67.2$ ft	h = building height = 112 ft, given
$I_{\bar{z}} = C\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.2\left(\frac{33}{67.2}\right)^{1/6} = 0.1776$	c from Table 6.4
$L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\varepsilon} = 500\left(\frac{67.2}{33}\right)^{1/5} = 576$	l and ε from Table 6.4
$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{90+112}{576}\right)^{0.63}}} = 0.87$	Q = background response B = building width perpendicular to wind = 112 ft, given
$G = 0.925\left(\frac{(1+1.7g_Q I_{\bar{z}} Q)}{1+1.7g_v I_{\bar{z}}}\right) = 0.925\left(\frac{1+1.7\times 3.4\times 0.1776\times 0.86}{1+1.7\times 3.4\times 0.1776}\right) = 0.86$	G = gust effect factor

Observe this is not much different from $G = 0.85$ permitted for rigid structures.

Calculations for Gust Effect Factor G_f: Flexible Structure
Given.
Building height $h = 600$ ft
Building width perpendicular to wind, $B = 100$ ft
Building depth parallel to wind, $L = 100$ ft
Building natural frequency $n_1 = 0.2$ Hz
Damping ratio = 0.015
Exposure category = C
Basic windspeed $V = 140$ mph

Required. Gust effect factor G_f

ASCE 7-02 formulas	Commentary
V = wind speed in ft/s $= V_{\text{mph}} \times 1.467$ $= 140 \times 1.467 = 205$ ft/s	$V = 140$ mph, given
$\bar{z} = 0.6$ h, but not less than $z_{\min}$ $= 0.6 \times 600 = 360 \text{ ft} > \bar{z}_{\min} = 15$ ft OK	h = building height $= 600$ ft, given $\bar{z}_{\text{mir}} = 15$ ft, from Table 6.4
$I_{\bar{z}} = C\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.20\left(\frac{33}{360}\right)^{1/6} = 0.134$	$c = 0.20$, from Table 6.4
$L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\varepsilon} = 500\left(\frac{360}{33}\right)^{1/5} = 806$ ft	$l = 500$ ft, $\varepsilon = 1/5$, from Table 6.4
$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}}}$ $= \sqrt{\frac{1}{1+0.63\left(\frac{100+600}{761}\right)^{0.63}}}$ $= 0.796, \quad Q^2 = 0.634$	Q = background response B = building with perpendicular to wind $= 100$ ft, given
$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V = 0.65\left(\frac{360}{33}\right)^{1/6.5} \times 205 = 192$ ft/s	$\bar{b} = 0.65, \bar{\alpha} = 1/6.5$, from Table 6.4
$\hat{V}_z = \hat{b}\left(\frac{\bar{z}}{33}\right)^{\hat{\alpha}} V = 1.0 \times \left(\frac{360}{33}\right)^{1/9.5} \times 205 = 264$ ft/s	$\hat{b} = 1.0, \hat{\alpha} = 1.0$, from Table 6.4
$N_1 = \frac{n_1 L_z}{\bar{V}_z} = \frac{0.2 \times 806}{192} = 0.84$ $R_n = \frac{7.47\, N_1}{(1+10.3\, N_1)^{5/3}}$ $= \frac{7.47 \times 0.84}{(1+10.3 \times 0.84)^{5/3}} = 0.143$	n_1 = natural frequency $= 0.2$ Hz, given
$\eta_b = \frac{4.6 n_1 B}{\bar{V}_z} = \frac{4.6 \times 0.2 \times 100}{192} = 0.479$ $R_B = \frac{1}{\eta_b} - \frac{1}{2\eta_b^2}(1 - e^{-2\eta_b})$ $= \frac{1}{0.479} - \frac{1}{2 \times 0.479^2}(1 - 2.71^{-2\times 0.479})$ $= 0.739$ $\eta_h = \frac{4.6 n_1 h}{\bar{v}_{\bar{z}}} = \frac{4.6 \times 0.2 \times 600}{192} = 2.875$	$e = 2.71$

$$R_h = \frac{1}{\eta_h} - \frac{1}{2\eta_h^2}(1 - e^{-2\eta_h})$$

$$= \frac{1}{2.875} - \frac{1}{2 \times 2.875^2}(1 - 2.71^{-2\times 2.875})$$

$$= 0.288$$

$$\eta_L = \frac{15.4 n_1 L}{\bar{v}_{\bar{z}}} = \frac{15.4 \times 0.2 \times 100}{192} = 1.604$$

L = building breadth parallel to wind = 100 ft, given

$$R_L = \frac{1}{\eta_L} - \frac{1}{2\eta_L^2}(1 - 2.71^{-2\eta_L})$$

$$= \frac{1}{1.604} - \frac{1}{2 \times 1.604^2}(1 - 2.71^{-2\times 1.604})$$

$$= 0.618$$

$$R^2 = \frac{1}{\beta} R_n R_h R_B (0.53 + 0.47\, R_L)$$

β = damping ratio = 0.015, given

$$= \frac{1}{0.015} \times 0.143 \times 0.288 \times 0.739(0.53 + 0.47 \times 0.618)$$

$$= 1.665 \qquad R = \sqrt{1.665} = 1.29$$

$$g_R = \sqrt{2\ln(3600 \times n_1)} + \frac{0.577}{\sqrt{2\ln(3600 \times n_1)}}$$

ln means logarithm to base $e = 2.71$.

$$= \sqrt{2\ln(3600 \times 0.2)} + \frac{0.577}{\sqrt{2\ln(3600 \times 0.2)}}$$

$$= 3.789$$

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

G is the gust factor.
$g_Q = g_V = 3.4$ (defined in the equation for G).

$$= 0.925\left(\frac{1 + 1.7 \times 0.134\sqrt{3.4^2 \times 0.634 + 3.789^{-2} \times 1.665}}{1 + 1.7 \times 3.4 \times 0.134}\right)$$

$$= 1.185$$

Calculations for Design Wind Pressures: Graphical Procedure.

Given. A concrete building located in a hurricane prone region with the following characteristics:

- Building height = 450 ft (137.15 m)
- Building plan dimensions = 185 × 125 ft (56.396 × 38.10 m)
- Exposure category = C
- Basic wind speed = 110 mph (49 m/s)
- The building is sited on the upper half of a 2-D ridge and has the following topographic parameters:

$$L_h = 200 \text{ ft}, \quad H = 200 \text{ ft}, \quad x = 50 \text{ ft}$$

(See Fig. 1.11 for definitions.)

- It is anticipated that design will be performed by using basic load combinations specified in Sections 2.3.2 and 2.4.1 of ASCE 7-02. Observe that load factors associated with wind load combinations do not account for the directionality factor K_d. Therefore, the values of q_z that account for $K_d = 0.85$, as shown in Fig. 1.2, may be used directly in the ASCE 7-02 load combinations.
- The building is for typical office occupancy. However, it does have designated areas where more than 300 people congregate in one area.
- Damping ratio = 0.02 (2% of critical)

Required. Using the graph given in Fig. 1.12, determine wind pressure for the main wind-force-resisting system (MWFRS) of the building. Use case 1 given in ASCE 7-02, Section 6.5.12.3. It should be noted that ASCE 7-02 specifies four distinct wind load cases. These include: 1) torsional effects caused by nonuniform pressure; 2) wind loads acting diagonally to the building; 3) torsion due to eccentricity between the elastic shear center and the center of mass at each level of the structure; and 4) wind distribution to capture possible across-wind response. In this example, we will calculate the wind loads in the x-direction for case 1, which consists of full design pressure acting on the projected area perpendicular to each principal axis of the building, considered separately along each principle axis. The designer is directed to Fig. 6.9 of ASCE 7-02 for a full description of the load cases.

Solution.

- The building is for office occupancy with certain areas designated for the congregation of more than 300 people. From Tables 1.1 and 6.1 of ASCE 7-02 (Tables 1.7 and 1.7a of this text), the classification of the building for wind load is category III, and importance factor for wind $I_w = 1.15$.
- Exposure category is C and basic wind speed $V = 110$ mph, as given in the statement of the problem. We select the curve designated as C110 in Fig. 1.12 to read the positive and negative pressures up the building height.
- The building's height-to-least-horizontal dimension is 450/125 = 3.6, less than 4.

Therefore, the building may be considered rigid from the first definition given in ASCE 7-02, Section C6.2. The second definition refers to the fundamental period T of the building. Using the formula

$T = C_t h_n^{3/4}$, determine T

where

T = fundamental period of the building, in secs
h_n = height of the building, in feet
C_T = coefficient equal to 0.030 for concrete moment frame buildings
$T = 0.030 \times 450^{3/4} = 2.93$ sec (say, 3 sec)

The natural frequency, n, which is the reciprocal of the period, is equal to $1/T = 1/3 = 0.33$ Hz. This is less than 1 Hz, the limiting frequency that delineates a rigid structure from a flexible structure. Therefore gust effect factor G_f must be determined using the procedure given in ASCE 7-02, Section 6.5.8.2. However, to emphasize the graphical procedure, for now we will assume $G_f = 0.910$, a value that will be determined shortly. Observe that if the building is considered rigid $G_f = G$, would have been 0.85.

- Because the building is located on a 2-D ridge, it may experience higher winds than buildings situated on level ground. Therefore, consider topographic effects in the determination of design wind pressures.

For the given values of L_h, H, and x, the multipliers K_1, K_2, and K_3 are obtained from Fig. 1.11. Observe that for $H/L_h > 0.5$, Note 2 for Fig. 1.11 alerts us to assume $H/L_h = 0.5$ for evaluating K_1 and to substitute 2 H for L_h for evaluating K_2 and K_3. Therefore, for $H/L_h = 200/200 = 1.0$, which is greater than 0.5, from Fig. 1.11, for exposure C, for a 2-D ridge, $K_1 = 0.725$.

Substituting 2 H for H, $x/H = x/2H = 50/400 = 0.125$, and from Fig. 1.11, $K_2 = 0.92$. Instead of the values tabulated in Fig. 1.11, we may also use the formulas in Fig. 1.11a to calculate K_2 and K_3. Thus,

$$K_2 = \left(1 - \frac{|x|}{\mu L_h}\right)$$

$$= \left(1 - \frac{50}{1.5 \times 400}\right) = 0.92 \qquad \text{(Note that for } H/L_h > 0.5, \; L_H = 2\,H, \text{ for calculating } K_2\text{)}$$

The parameter K_3 varies as the ratio x/L_h. It may be obtained by using either the tabulated values in Fig. 1.11 or the formula given in Fig. 1.11a.

$$K_3 = e^{-\gamma \frac{z}{L_h}}$$

Again, substiuting 2 H for L_h, and $\gamma = 3$,

$$K_3 = e^{-3 \frac{z}{400}}$$

We use the preceding formula to calculate K_3 for the selected z/L_h values shown. Note that $\gamma = 3$ for 2-D ridges, which is the topography for our building.

z(ft)	450	350	250	150	100	50	30	15
z/L_h	2.25	1.75	1.25	0.75	0.50	˙0.25	0.15	0.08
K_3	0.001	0.005	0.023	0.106	0.224	0.473	0.638	0.78

Wind Parallel to X-Axis. From the building's plan dimensions, $L/B = 125/180 = 0.694 < 1.0$. Therefore, from Fig. 1.9, C_p for the windward face = 0.8, and C_p for the leeward face = –0.5. From Fig. 1.12, select the curve identified as C110. C stands for exposure C, and 110 stands for $V = 110$ mph. Use the graph to read the values of q_z at various heights. For example, at $h = 150$ ft, $q_z = 36.3$ psf.

However, since the q_z and q_h values in Fig. 1.12 are normalized for $K_{zt} = 1.0$, $K_d = 0.85$, and $I_w = 1.0$, we multiply these values by the K_{zt} and I_w values of the example problem before recording the corresponding values in columns (7) and (8) of Table 1.10. For example, $q_z = 36.3$ psf at $z = 150$ ft, obtained from the graph is multiplied by $K_{zt} = 1.145$ and $I_w = 1.15$ to get a value of $q_z = 47.79$ psf, shown in column (7).

Observe that K_{zt} varies up the height. Similarly, values of q_z for different heights are recorded in column (7) of Table 1.10 after multiplication by K_{zt} and I_w. The suction q_h in column (8) is the value from the graph at $z = h = 450$ ft multiplied by $K_{zt} = 1.002$ and $I_w = 1.15$. Observe that the suction q_h referenced at roof height remains constant for the entire height of leeward wall. Column (9) gives the total design wind pressure throughout the building height. It is the summation of $0.8q_z$, the positive pressure on the windward wall, plus $0.5q_h$, the suction on the leeward wall, multiplied by the gust effect factor $G_f = 0.91$.

TABLE 1.10 Design Wind Pressures, Graphical Procedure; ASCE 7-02 Procedure

Height (ft)	K_z (1)	K_h (2)	K_1 (3)	K_2 (4)	K_3 (5)	$K_{zt} = (1 + K_1K_2K_3)^2$ (6)	q_z (psf) (From Fig 1.12) $\times I_w \times K_{zt}$ (7)	q_h (psf) (8)	Design pressure $P = (0.8q_z + 0.5q_h) \times G_f$ (9)	Design pressure (psf) without topographic factors, is $K_{zt} = 1.0$ (10)
450	1.74	1.74	0.725	0.92	0.00	1.002	52.68	52.68	63.4	62.2
400	1.69	1.74	0.725	0.92	0.00	1.003	54.48	52.68	62.5	61.3
350	1.65	1.74	0.725	0.92	0.01	1.007	50.24	52.68	61.6	60.2
300	1.59	1.74	0.725	0.92	0.01	1.014	49.01	52.68	60.7	59.1
250	1.53	1.74	0.725	0.92	0.02	1.031	47.94	52.68	59.9	57.7
200	1.46	1.74	0.725	0.92	0.05	1.067	47.33	52.68	59.5	56.2
150	1.38	1.74	0.725	0.92	0.11	1.145	47.79	52.68	59.8	54.3
100	1.26	1.74	0.725	0.92	0.22	1.318	50 .53	52.68	61.8	51.8
50	1.09	1.74	0.725	0.92	0.47	1.726	57.18	52.68	66.6	48.0
30	0.98	1.74	0.725	0.92	0.64	2.027	60.29	52.68	68.9	45.6
15	0.85	1.74	0.725	0.92	0.80	2.343	60.22	52.68	68.9	42.6

Notes: $G_f = 0.91$, $I_w = 1.15$, $K_d = 0.85$.

For comparative purposes, column (10) of Table 1.10 gives the design pressures P for the building assuming that it is located on a flat terrain, i.e., $K_{zt} = 1.0$.

Calculations for Gust Effect Factor G_f, and Along-Wind Displacements and Accelerations. As mentioned previously, to place emphasis on the graphical solution, we assumed $G_f = 0.910$ in our calculations for design wind pressures. We will now calculate this value using the worksheet format given in Section 1.4.2.7. We will also calculate the maximum values for along-wind displacement and acceleration using the worksheet.

Given. This is a continuation of the illustrative problem stated in the section titled "Calculations for Design Wind Pressures: Graphical Procedure." Therefore we use the same building characteristics and wind environment data given therein.

Required. Gust effect factor G_f and maximum values for along-wind displacement and accelerations using the worksheet given in Section 1.4.2.7.

ASCE 7-02 Formulas	Commentary
V = wind speed in ft/s $= V_{\text{mph}} \times 1.467$ $= 110 \times 1.467 = 161$ ft/s	$V = 110$ mph, given
$\bar{z} = 0.6\text{ h}$, but not less than $z_{\min}$ $= 0.6 \times 450 = 270 \text{ ft} > \bar{z}_{\min} = 15 \text{ ft}$ OK	h = building height = 450 ft, given $\bar{z}_{\min} = 15$ ft, from Table 6.4
$I_{\bar{z}} = C\left(\dfrac{33}{\bar{z}}\right)^{1/6} = 0.20\left(\dfrac{33}{270}\right)^{1/6} = 0.141$	$c = 0.20$, from Table 6.4
$L_{\bar{z}} = l\left(\dfrac{\bar{z}}{33}\right)^{\varepsilon} = 500\left(\dfrac{270}{33}\right)^{1/5} = 761.3 \text{ ft}$	$l = 500$ ft, $\varepsilon = 1/5$, from Table 6.4
$Q = \sqrt{\dfrac{1}{1 + 0.63\left(\frac{B+h}{L_z}\right)^{0.63}}}$ $= \sqrt{\dfrac{1}{1 + 0.63\left(\frac{185+450}{761}\right)^{0.63}}}$ $= 0.80, \quad Q^2 = 0.64$	Q = the background response B = building width perpendicular to wind = 185 ft, given
$\bar{V}_{\bar{z}} = \bar{b}\left(\dfrac{\bar{z}}{33}\right)^{\bar{\alpha}} V = 0.65\left(\dfrac{270}{33}\right)^{1/6.5} \times 161 = 144.9 \text{ ft/s}$	$\bar{b} = 0.65, \bar{\alpha} = 1/6.5$, from Table 6.4
$\hat{V}_z = \hat{b}\left(\dfrac{\bar{z}}{33}\right)^{\hat{\alpha}} V = 1.0 \times \left(\dfrac{270}{33}\right)^{1/9.5} \times 161 = 201 \text{ ft/s}$	$\hat{b} = 1.0, \hat{\alpha} = 1.0$, from Table 6.4
$N_1 = \dfrac{n_1 L_z}{\bar{V}_z} = \dfrac{0.33 \times 761}{144.9} = 1.751$ $R_n = \dfrac{7.47\, N_1}{(1 + 10.3\, N_1)^{5/3}}$ $= \dfrac{7.47 \times 1.751}{(1 + 10.3 \times 1.751)^{5/3}} = 0.096$	n_1 = natural frequency = 0.33 Hz, given

$$\eta_b = \frac{4.6 n_1 B}{\bar{V}_z} = \frac{4.6 \times 0.33 \times 185}{144.9} = 1.958$$

$e = 2.71$

$$\begin{aligned} R_B &= \frac{1}{\eta_b} - \frac{1}{2\eta_b^2}(1 - e^{-2\eta_b}) \\ &= \frac{1}{1.958} - \frac{1}{2 \times 1.958^2}(1 - 2.71^{-2\times 1.958}) \\ &= 0.383 \end{aligned}$$

$$\eta_h = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{4.6 \times 0.33 \times 450}{144.9} = 4.762$$

$$\begin{aligned} R_h &= \frac{1}{\eta_h} - \frac{1}{2\eta_h^2}(1 - e^{-2\eta_h}) \\ &= \frac{1}{4.762} - \frac{1}{2 \times 4.762^2}(1 - 2.71^{-2\times 4.762}) \\ &= 0.188 \end{aligned}$$

$$\eta_L = \frac{15.4 n_1 L}{\bar{V}_{\bar{z}}} = \frac{15.4 \times 0.33 \times 125}{144.9} = 4.429$$

L = building breadth parallel to wind = 125 ft, given

$$\begin{aligned} R_L &= \frac{1}{\eta_L} - \frac{1}{2\eta_L^2}(1 - 2.71^{-2\eta_L}) \\ &= \frac{1}{4.429} - \frac{1}{2 \times 4.429^2}(1 - 2.71^{-2\times 4.429}) \\ &= 0.200 \end{aligned}$$

$$\begin{aligned} R^2 &= \frac{1}{\beta} R_n R_h R_B (0.53 + 0.47\, R_L) \\ &= \frac{1}{0.02} \times 0.096 \times 0.188 \times 0.388(0.53 + 0.47 \times 0.200) \\ &= 0.216 \qquad R = \sqrt{0.216} = 0.465 \end{aligned}$$

β = damping ratio = 0.02, given

$$\begin{aligned} g_R &= \sqrt{2\ln(3600 \times n_1)} + \frac{0.577}{\sqrt{2\ln(3600 \times n_1)}} \\ &= \sqrt{2\ln(3600 \times 0.33)} + \frac{0.577}{\sqrt{2\ln(3600 \times 0.33)}} \\ &= 3.919 \end{aligned}$$

ln means logarithm to base $e = 2.71$

$$\begin{aligned} G &= 0.925\left(\frac{1 + 1.7\, I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R R^2}}{1 + 1.7\, g_V I_{\bar{z}}}\right) \\ &= 0.925\left(\frac{1 + 1.7 \times 0.141\sqrt{3.4^2 \times 0.64 + 3.919^2 \times 0.216}}{1 + 1.7 \times 3.4 \times 0.141}\right) \\ &= 0.910 \end{aligned}$$

G is the gust factor
$g_Q = g_V = 3.4$ (defined in the equation for G)

For comparative purposes, it may be of interest to calculate the gust factor G_f for this building using the improved method for rigid structures given in Section 1.4.2.5.

$$\begin{aligned}
G &= 0.925\left(\frac{1+1.7g_q I_{\bar{z}} Q}{1+1.7g_v I_z}\right)\\
I_z &= C\left(\frac{33}{z}\right)^{1/6} = 0.141\\
L_z &= 761 \qquad Q = 0.80\\
G &= 0.925\left(\frac{1+1.7\times 3.4\times 0.141\times 0.80}{1+1.7\times 3.4\times 0.141}\right)\\
&= 0.912
\end{aligned} \tag{1.38}$$

Observe that this value of 0.912 is not much different from 0.910 calculated using the more complex procedure.

$$K = \frac{1.65^{\hat{\alpha}}}{\hat{\alpha}+\xi+1} = \frac{1.65^{1/9.5}}{\frac{1}{9.5}+1+1} = 0.50 \qquad \xi = \text{the first mode exponent taken} = 1.0$$

$$g_{\ddot{x}} = g_R = 3.91$$

Maximum Along-Wind Displacement.

$$\begin{aligned}
X_{\max(z)} &= \frac{\phi_{(z)}\rho B h C_{fx}\hat{V}^2}{2m_1(2\pi n_1)^2}\\
&= \frac{1\times 0.0024\times 185\times 450\times 1.3\times 201^2}{2\times 1,618,200\,(2\times\pi\times 0.3)^2} \qquad \rho = \text{air density} = 0.0024\text{ slugs}\\
&\quad \times 0.5\times 0.925\\
&= 0.4221\text{ ft}
\end{aligned} \tag{1.39}$$

Note: $X_{\max(z)}$ is also commonly reffered to as lateral drift, Δ. Tall buildings are usually designed for a drift index $\frac{\Delta}{h} \cong \frac{1}{500}$. In our case, $\frac{\Delta}{h} = \frac{0.4221}{450} = \frac{1}{1056}$, indicating that the example building is quite stiff.

$$\begin{aligned}
\sigma_{\ddot{x}_{(z)}} &= \frac{0.85\phi_z \rho B h C_{fx}\bar{v}^2}{m_1}\times I_z K R\\
&= \frac{0.85\times 1\times 0.0024\times 185\times 450\times 1.3\times 144.6^2}{1,618,200}\times 0.141\times 0.5\times 0.5196\\
&= 0.104\text{ ft/sec}^2
\end{aligned} \tag{1.40}$$

Maximum Along-Wind Acceleration.

$$\begin{aligned}
\ddot{X}_{\max(z)} &= g_{\ddot{x}}\sigma_{\ddot{x}_{(z)}}\\
&= 3.89\times 0.1040\\
&= 0.40\text{ ft/sec}^2\\
&= 0.40\times 31.11 = 12.58\text{ milli-g}
\end{aligned} \tag{1.41}$$

This is well below the normally accepted limit of 20 milli-g, warranting no further investigation.

Calculations for Wind Pressure (ASCE 7-02): 10-Story Building.

Given. An office building with the following structural characteristics:

Plan dimensions	60 ft × 120 ft (18.28 m × 36.57 m)
Building height	10 floors at 14 ft floor-to-floor = 10 × 14 = 140 ft (42.67 m)
Fundamental frequency	1.1 Hz
Building classification	Category II
Basic wind speed	90 mph, 3-sec gust speed for Las Vegas, from wind speed map, Fig. 1.9
Exposure category	Urban terrain
Topographic factor K_{zt}	1.0
Load combinations	ASCE 7-02 ultimate strength design. Therefore, use directionality factor $K_d = 0.85$.

Required. Wind pressures for the design of primary lateral system.

Solution.

Step 1. Building Classification. A typical office building is not generally considered an essential facility in the aftermath of windstorm, nor is its primary function for occupancy by more than 300 persons in one area. Therefore, the example building is judged to be type-II category. However, before a building is classified into a category, it is good practice to ascertain with building owners and plan-check officials that the category is consistent with their policies.

Results of Step 1: Category II, $I_w = 1.0$ (ASCE 7-02 Tables 1.1 and 6.1)
(Tables 1.7 and 1.7a of this text)

Step 2. Basic Wind Speed V. The ASCE wind speed map shown in Fig. 1.10 indicates that Las Vegas is in a wind contour of 90 mph. As indicated previously, it is good practice to confirm the design wind speed with local plan-check officials.

Results of Step 2: $V = 90$ mph

Step 3. Determination of Gust Response Factor G. The building height-to-width ratio of 140/60 = 2.3 is less than 4, and its natural frequency of 1.1 Hz is more than 1.0 Hz. Therefore, the building may be considered a nonflexible building and a value of 0.85 may be used for the gust response factor G.

Results of Step 3: $G = 0.85$

Step 4. Directionality Factor K_d. Since ASCE 7-02 load combinations are anticipated, we use $K_d = 0.85$.

Step 5. External Pressure Coefficient C_p. From the given plan dimensions, the building width-to-depth ratio of L/B = 60/120 = 0.5 for wind parallel to the 60-ft face. For wind parallel to the 120-ft face, the ratio = 120/60 = 2.0.
From Figs. 1.10 and 1.10a the following values of C_p are obtained:

$C_p = 0.8$ for the windward wall
$C_p = 0.5$ for the leeward wall, wind parallel to 60-ft face
$C_p = 0.3$ for leeward wall, wind parallel to 120-ft face

Roof and internal pressures and suctions are not relevant in determining wind loads for primary lateral system: Internal pressures and suctions acting on the windward and leeward walls cancel out without adding or subtracting to the overall wind loads. Roof suction, which results in uplift forces, is generally neglected in the design of a primary lateral system.

Results of Step 4 are shown as C_p values in Table 1.11.

TABLE 1.11 Example Problem, 10-Story Building: Design Pressures for Main Wind-Force-Resisting Frame; ASCE 7-02 Procedure

	Value of K_z and q_z				Design pressures for main wind-force-resisting system	
	Windward		Leeward		X-wind, psf	Y-wind, psf
Height, ft	K_z	q_z, psf	K_h	q_h	$p_z = q_zG \times 0.8 + q_hG \times 0.5$	$p_z = q_zG \times 0.8 + q_hG \times 0.3$
140	1.09	19.2	1.09	19.2	21.2	18.0
100	0.99	17.4	1.09	19.2	20.0	16.7
80	0.93	16.0	1.09	19.2	19.0	15.8
60	0.85	15.0	1.09	19.2	18.4	15.1
40	0.76	13.0	1.09	19.2	17.0	13.7
30	0.70	12.3	1.09	19.2	16.5	13.3
20	0.62	11.4	1.09	19.2	15.9	12.6
0–15	0.57	10.2	1.09	19.2	15.1	11.8

X-Wind: $c_p = +0.8$ Windward; $c_p = -0.5$ Leeward
Y-Wind: $c_p = +0.8$ Windward; $c_p = -0.3$ Leeward
$q_z = 0.00256\ K_z\ K_{zt}\ K_d\ V^2\ I_w$
Gust factor $G = 0.85$
Exposure category = B
$V = 90$ mph (3-sec. gust)
Topographic factor $K_{zt} = 1$
Directionality factor $K_d = 0.85$
Wind importance factor $I_w = 1.0$

Step 6. Building Exposure. Since the building is located in an urban terrain, the exposure category is judged to be B.

Results of Step 5: Exposure B

Step 7. Combined Velocity Pressure, Exposure Coefficient K_z. The gradient height Z_g and the power coefficient α for exposure B are 1200 ft and 7.5, respectively (Table 1.7a). Below 15 ft, the value of K_z is taken as a constant determined at height 15 ft.

$$K_z = 2.01\left(\frac{z}{1200}\right)^{2/7.5} \qquad \text{for} \qquad 15\text{ ft} \le z \le z_g \tag{1.42}$$

$$K_z = 2.01\left(\frac{15}{1200}\right)^{2/7.5} \qquad \text{for} \qquad z \le 15\text{ ft} \tag{1.43}$$

Instead of calculating values of Kz from the preceding equations, they can be obtained directly from Table 1.8 or Fig. 1.11. The results of Step 7 are shown in Table 1.11.

Step 8. Velocity Pressure q_z. Values for q_z are obtained from Eqs. (1.14) and (1.15). The results of Step 8 are shown in column 3 and 5 of Table 1.11.

Step 9. Design Pressure p. With the known values of q, G, and C_p, the design pressure is obtained by the chain equation

$$P = q \times G \times C_p$$

Wind pressures and suctions on windward and leeward walls are shown in Table 1.11. Summation of the two is used is used in determining the pressure for the design of main wind-force-resisting system.

Step 10. Floor-by-Floor Wind Load. This is obtained by multiplying the exposed area tributary to the level by the corresponding value of design pressure at floor height.

Calculations for Wind Pressures (ASCE 7-02): 30-Story Building.
Given.

- Location of building: Houston, Texas, use $V = 110$ mph
- Terrain: Flat, open country with scattered obstructions having the size of single-family dwellings, height generally less than 30 ft
- Plan dimensions: 98.5 ft × 164 ft
- Building height: 394 ft
- Building lateral-load-resisting system: moment frame with shear walls. Fundamental period $T = 3.12$ secs. Therefore, frequency $f = 1/3.12 = 0.32$ Hz.
- The building is regular, as defined in ASCE 7-02 Section 6.2. It does not have unusual geometric irregularities.
- It does not have response characteristics that would subject the building to across-wind loading, vortex shedding, or instability due to galloping or flutter. The building does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.
- Damping factor: 1.5% of critical
- Topographic factor $K_{zt} = 1.0$
- The building is for typical office occupancy

Required. Wind pressures for the design of MWFRS using ASCE 7-02. Include ample explanation to emphasize the essential requirements of ASCE provisions. Use $K_{zt} =$ 1.0 for the basic problem and compare results by using the following topographic factors for a 2-D ridge. (Refer to Fig. 1.11 for definitions.)

$L_h = 100$ ft, $H = 100$ ft, $x = 50$ ft

Solution. The design wind loads for lateral analysis of the building will be based on method 2, the analytical procedure of ASCE 7-02 Section 6.5. This method is applicable to the example building, as it satisfies the two conditions set forth in Section 6.5.1. of ASCE 7.

The design wind load is calculated after determining the following quantities:

- The basic wind speed V (6.5.4)
- A wind directionality factor K_d (6.5.4.4)
- An importance factor I_w (6.5.5)
- An exposure category and velocity pressure coefficient K_z or K_h, as applicable (6.5.6)
- A topographic factor K_{zt} (6.5.7)
- A gust for G or G_f as applicable (6.5.8)
- An enclosure classification (6.5.9)
- Internal pressure coefficient GC_{pi} (6.5.11.1)
- External pressure coefficients C_p or GC_{pf}, or force coefficients C_f as applicable, (6.5.11.2 or 6.5.11.3)
- Velocity pressure q_z or q_h as applicable (6.5.10)
- Design wind pressure P (6.5.12)

The numbers in parentheses indicate section numbers of the ASCE 7-02 Standard.

Enclosure Classification Determination of enclosure classification and internal pressure coefficient is not required for the calculation of overall wind loads for typical diaphragmed buildings, because internal pressures acting in opposite directions on the windward and leeward walls cancel out.

Basic Wind Speed From Figure 1.10d, the wind velocity $V = 110$ mph for Houston, Texas. The wind must be assumed to come from any direction.

Wind Directionality Factor From ASCE 7-02 Table 6-6 (Table 1.8 of this text), $K_d = 0.85$.

Importance Factor The example office building can be placed under category II.

Importance factor $I_w = 1.00$ (Table 1.7a)

Exposure Category The terrain for the example building consists of flat, open country with scattered obstructions having heights generally less than 30 ft (9.1 m).

Therefore, exposure category = C (ASCE 7-02, Section 6.5.9)

Velocity Pressure Exposure Coefficients K_z and K_h Case 2 of ASCE 7-02, Table 6-3, is applicable for determining K_z and K_h. These may be calculated using the general equations

$$K_z = 2.01\ (z/_{zg})^{2/\alpha} \quad \text{for} \quad 15\text{ ft} \le z \le z_g \tag{1.44}$$

and

$$K_z = 2.01\ (15/Z_g)^{2/\alpha} \quad \text{for} \quad Z \le 15\text{ ft} \tag{1.44a}$$

where

Z = height above ground level, ft
Z_g = gradient height
α = 3-sec gust speed power law exponent

The values for Z_g and α are given in Table 6.2 of ASCE 7-02 (Table 1.9 of this text). From this table, for exposure category C:

α = 3-sec. Gust speed pow\er law exponent = 9.5

Z_g = nominal height of the boundary layer = 900 ft

The magnitudes of K_z at different heights and K_h at the roof are shown in Table 1.6. These values are the same as those given in Table 6.3 of AISC 7-02, except only the values required for determining wind loads on MWFRS are given, and the height z aboveground has been extended to 1500 ft.

Topographic Effects This effect is given by a factor K_{zt}

$$K_{zt} = (1 + K_1K_2K_3)^2 \quad \text{[ASCE 7-02 Eq. (6.13)]} \tag{1.44b}$$

where K_1, K_2, and K_3 are given in Fig. 1.11 for various topographic factors. The example building is situated on level ground. Therefore, the K_{zt} factor may be taken equal to 1.

Calculation of Flexibility of Structure A structure is considered flexible per commentary Section 6.2 of ASCE if it has a fundamental natural frequency of less than 1 Hz (i.e., a fundamental period $T > 1$ sec.). To find the period of our building, we will use an approximate formula normally used in seismic design. This formula gives the period T, in terms of number of stories N present in the building by the relation: $T = 0.1N$. For the subject building, then, $T = 0.1 \times 30 = 3.0$ sec with a corresponding fundamental frequency of 0.333 Hz. This is considerably less than 1 Hz. Therefore, the building is considered flexible for purpose of determining the gust effect factor. It should be noted that a dynamic analysis of buildings typically gives building periods longer than those from the approximate formula. For a moment frame building, for example, the approximate period is usually in the range of $T = 0.15N$. However, for our building, which has a combination of shear walls and moment frames, we use the given fundamental period of $T = 3.12$ sec. with a corresponding frequency $f = 0.32$ Hz.

TABLE 1.12 Internal Pressure Coefficient GC_{pi} for Main Wind-Force-Resisting System/Components and Cladding (Walls and Roofs)

Enclosure classification	GC_{pi}[b,c]
Open buildings	0.00
Partially enclosed buildings	+0.55[a]
	−0.55
Enclosed buildings	+0.18[a]
	−0.18

[a] Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.
[b] Values of GC_{pi} shall be used with q_z or q_h as specified in 1.4.2.2.
[c] Two cases shall be considered to determine the critical load requirements for the appropriate condition:
1. A positive value of GC_{pi} applied to all internal surfaces.
2. A negative value of GC_{pi} applied to all internal surfaces.

(From ASCE 7-02 Fig. 6.5.)

Enclosure Classifications and Internal Pressure Coefficients For purposes of determining internal pressure coefficients, all buildings are classified as enclosed, partially enclosed, or open (see Table 1.12, ASCE 7-02 Fig. 6.5). However, internal pressures do not come into play in the determination of overall wind loads for lateral load analysis of buildings. The overall wind loads can therefore be determined using only the external pressures and suctions on the windward and leeward faces.

Velocity pressure Velocity pressure q_z, evaluated at height z, is calculated by the following equation:

$$q_z = 0.00256\, K_z,\, K_{zt}, K_d V^2\, \mathrm{I} \tag{1.45}$$

Design Wind Pressure *P* for Enclosed Flexible Buildings is determined from the following general equation:

$$P = qG_fC_p - q_i\,(GC_{pi}) \tag{1.46}$$

where

$q = q_z$ for windward walls evaluated at height z
$q = q_h$ for leeward walls, side walls, and roofs, evaluated at height h
$q_i = q_h$ for internal pressure evaluation in enclosed buildings
GC_{pi} = internal pressure coefficient (Table 1.12)

Since internal pressures do not effect the overall wind loads, q_i may be eliminated from the preceding equation, giving the wind pressure and suction for MWFRS, as follows.

$P_{\text{pressure}} = q_z C_f C_p$ (pressure on windward wall calculated at height z)

$P_{\text{suction}} = q_h G_f C_p$ (suction on leeward wall, side walls, and roof calculated at height h)

With this explanation, we now proceed to calculate the design pressures for the example building. Instead of hand calculations, a spreadsheet[a] has been used to obtain the results shown in Tables 1.13 and 1.14.

[a] The author wishes to acknowledge gratitude to his colleague Mr. Ryan Wilkerson, S.E., who developed the spreadsheets and reviewed this chapter and made valuable suggestions.

TABLE 1.13 Main Wind-Resisting-System (No Topographic Effects)

Exposure category	C	Bldg. frequency	0.32	(Flexible)
Building height	394	ft		
Wind speed	110	mph		
Width (B)	164	ft		
Length (L)	98.5	ft		
Importance	1			
K_d	0.85			
Bldg. period	3.1	(seconds)		
β	0.015	(damping ratio)		

Terrain Exposure Constants

α	z_g	$\hat{a}$	$\hat{b}$	$\bar{\alpha}$
9.5	900	0.1053	1	0.1538
$\bar{b}$	c	ℓ	$\bar{\varepsilon}$	$z_{\min}$
0.65	0.2	500	0.2	15

$L_{\bar{z}}$	$\bar{z}$	$l_{\bar{z}}$	$V_{\bar{z}}$	g_r	N_1	R_n
741.3	236.4	0.144	142.0	3.911	1.685	0.099
Q	η_h	R_h	η_a	R_a	η_L	R_L
0.809	4.118	0.213	1.714	0.419	3.447	0.248

R	G_t	G
0.616	0.96	0.84

Height	K_3	K_{zt}	K_z	q_z
0	1	1	0.85	22.35
17	0.77	1	0.87	22.95
30	0.64	1	0.98	25.86
43	0.52	1	1.06	27.90
56	0.43	1	1.12	29.49
69	0.36	1	1.17	30.82
82	0.29	1	1.21	31.96
95	0.24	1	1.25	32.97
108	0.20	1	1.29	33.87
121	0.16	1	1.32	34.69
134	0.13	1	1.35	35.44
147	0.11	1	1.37	36.14
160	0.09	1	1.40	36.79
173	0.07	1	1.42	37.40
186	0.06	1	1.44	37.97
199	0.05	1	1.46	38.52
212	0.04	1	1.48	39.03
225	0.03	1	1.50	39.53
238	0.03	1	1.52	40.00
251	0.02	1	1.54	40.45
264	0.02	1	1.55	40.88
277	0.02	1	1.57	41.30
290	0.01	1	1.58	41.70

(*Continued*)

TABLE 1.13 (Continued)

Height	K_3	K_{zt}	K_z	q_z
303	0.01	1	1.60	42.08
316	0.01	1	1.61	42.46
329	0.01	1	1.63	42.82
342	0.01	1	1.64	43.17
355	0.00	1	1.65	43.51
368	0.00	1	1.67	43.84
381	0.00	1	1.68	44.16
394	0.00	1	1.69	44.47

	Windward wind				Leeward wind				Total wind		
Height	C_p	Wind, psf	Load, plf	OTM, kip-ft	C_p	Wind, psf	Load, plf	OTM, kip-ft	Wind, psf	Load, plf	OTM, kip-ft
0	0.8	17.1	146.4	0.0	0.5	21.3	180.9	0.0	38.4	327.2	0.0
17	0.8	17.6	266.1	4.5	0.5	21.3	319.1	5.4	38.8	585.3	9.9
30	0.8	19.8	256.3	7.7	0.5	21.3	276.6	8.3	41.1	532.8	16.0
43	0.8	21.4	277.1	11.9	0.5	21.3	276.6	11.9	42.6	553.7	23.8
56	0.8	22.6	293.1	16.4	0.5	21.3	276.6	15.5	43.9	569.7	31.9
69	0.8	23.6	306.4	21.1	0.5	21.3	276.6	19.1	44.9	583.0	40.2
82	0.8	24.5	317.9	26.1	0.5	21.3	276.6	22.7	45.7	594.4	48.7
95	0.8	25.2	327.9	31.2	0.5	21.3	276.6	26.3	46.5	604.5	57.4
108	0.8	25.9	336.9	36.4	0.5	21.3	276.6	29.9	47.2	613.5	66.3
121	0.8	26.6	345.1	41.8	0.5	21.3	276.6	33.5	47.8	621.7	75.2
134	0.8	27.1	352.6	47.2	0.5	21.3	276.6	37.1	48.4	629.2	84.3
147	0.8	27.7	359.5	52.9	0.5	21.3	276.6	40.7	48.9	636.1	93.5
160	0.8	28.2	366.0	58.6	0.5	21.3	276.6	44.3	49.4	642.6	102.8
173	0.8	28.6	372.1	64.4	0.5	21.3	276.6	47.9	49.9	648.7	112.2
186	0.8	29.1	377.8	70.3	0.5	21.3	276.6	51.4	50.3	654.4	121.7
199	0.8	29.5	383.2	76.3	0.5	21.3	276.6	55.0	50.8	659.8	131.3
212	0.8	29.9	388.4	82.3	0.5	21.3	276.6	58.6	51.2	665.0	141.0
225	0.8	30.3	393.3	88.5	0.5	21.3	276.6	62.2	51.5	669.9	150.7
238	0.8	30.6	398.0	94.7	0.5	21.3	276.6	65.8	51.9	674.6	160.5
251	0.8	31.0	402.5	101.0	0.5	21.3	276.6	69.4	52.2	679.0	170.4
264	0.8	31.3	406.8	107.4	0.5	21.3	276.6	73.0	52.6	583.4	180.4
277	0.8	31.6	410.9	113.8	0.5	21.3	276.6	76.6	52.9	687.5	190.4
290	0.8	31.9	414.9	120.3	0.5	21.3	276.6	80.2	53.2	691.5	200.5
303	0.8	32.2	418.7	126.9	0.5	21.3	276.6	83.8	53.5	695.3	210.7
316	0.8	32.5	422.5	133.5	0.5	21.3	276.6	87.4	53.8	699.1	220.9
329	0.8	32.8	426.1	140.2	0.5	21.3	276.6	91.0	54.1	702.7	231.2
342	0.8	33.0	429.5	146.9	0.5	21.3	276.6	94.6	54.3	706.1	241.5
355	0.8	33.3	432.9	153.7	0.5	21.3	276.6	98.2	54.6	709.5	251.9
368	0.8	33.6	436.2	160.5	0.5	21.3	276.6	101.8	54.8	712.8	262.3
381	0.8	33.8	439.4	167.4	0.5	21.3	276.6	105.4	55.1	716.0	272.8
394	0.5	34.0	220.9	87.0	0.5	21.3	138.3	54.5	55.3	359.2	141.5
		Totals:	11125.4	2390.8		Totals:	8383.0	1651.4	Totals:	19508.4	4042.3

Comparison of Gust Effect Factors and Along-Wind Responses. ASCE 7-02 permits a single gust effect factor of 0.85 for rigid buildings and as an option, specific features of the building size and wind environment may be incorporated to more accurately calculate a gust effect factor. The gust effect factor accounts for the loading effects due to wind turbulence structure interaction, and along-wing loading effects due to dynamic amplification for flexible buildings.

TABLE 1.14 Main Wind-Resisting-System (Topographic Factors: $L_h = 100$ ft, $H = 100$ ft, $x = 50$ ft)

Exposure category	C		Bldg. frequency	0.32	(Flexible)
Building height	395	ft	K_h	1.69	
Wind speed	110	mph	K_1	0.725	
Width (B)	164	ft	K_2	0.83	
Length (L)	98.5	ft	K_{3h}	0.00	
Importance	1		K_{ht}	1.003231	
K_d	0.85		q_i	44.64 psf	
Bldg. period	3.1	(seconds)			
β	0.015	(Damping ratio)			
GC_{pi}	0	(Internal pressure)			

Terrain Exposure Constants

α	z_g	$\hat{a}$	$\hat{b}$	$\bar{\alpha}$
9.5	900	0.1053	1	0.1538
$\bar{b}$	c	ℓ	$\bar{\varepsilon}$	z_{min}
0.65	0.2	500	0.2	15

Bldg location relative to crest: ○ Upwind ◉ Downwind

Topographic factors	2-D ridge/valley ▼			
L_h	100 ft		γ	3
H	100 ft (+/– ridge/valley)		μ	1.5
x	50 ft		$K_1/(H/L_h)$	0.725

$L_{\bar{z}}$	$\bar{z}$	$I_{\bar{z}}$	$V_{\bar{z}}$	g_r	N_1	R_n
741.7	237	0.144	142.0	3.911	1.685	0.099
Q	η_h	R_h	η_B	R_B	η_L	R_L
0.809	4.127	0.213	1.714	0.419	3.446	0.248

R	G_f	G
0.616	0.957	0.84

Height	K_3	K_{zt}	K_z	q_z
0	1	2.573351	0.85	57.52
17	0.60	1.857222	0.87	42.62
30	0.41	1.551609	0.98	40.13
43	0.28	1.360278	1.06	37.95
56	0.19	1.237881	1.12	36.51
69	0.13	1.158287	1.17	35.70
82	0.09	1.105898	1.21	35.34
95	0.06	1.071117	1.25	35.31
108	0.04	1.047883	1.29	35.49
121	0.03	1.032297	1.32	35.81
134	0.02	1.021811	1.35	36.21
147	0.01	1.014741	1.37	36.67
160	0.01	1.009969	1.40	37.16
173	0.01	1.006744	1.42	37.65
186	0.00	1.004564	1.44	38.15
199	0.00	1.003089	1.46	38.64
212	0.00	1.002091	1.48	39.12
225	0.00	1.001415	1.50	39.58
238	0.00	1.000958	1.52	40.04
251	0.00	1.000649	1.54	40.47

(*Continued*)

TABLE 1.14 (Continued)

Height	K_3	K_{zt}	K_z	q_z
264	0.00	1.000439	1.55	40.90
277	0.00	1.000297	1.57	41.31
290	0.00	1.000201	1.58	41.70
303	0.00	1.000136	1.60	42.09
316	0.00	1.000092	1.61	42.46
329	0.00	1.000062	1.63	42.82
342	0.00	1.000042	1.64	43.17
355	0.00	1.000029	1.65	43.51
368	0.00	1.000019	1.67	43.84
381	0.00	1.000013	1.68	44.16
395	0.00	1.000009	1.69	44.48

		Windward wind				Leeward wind			Total wind		
Height	C_p	Wind, psf	Load, plf	OTM, kip-ft	C_p	Wind, psf	Load, plf	OTM, kip-ft	Wind, psf	Load, plf	OTM, kip-ft
0	0.8	44.0	349.9	0.0	0.5	21.4	181.5	0.0	65.4	531.4	0.0
17	0.8	32.6	510.4	6.7	0.5	21.4	320.3	5.4	54.0	830.6	14.1
30	0.8	30.7	399.6	12.0	0.5	21.4	277.6	8.3	52.1	677.2	20.3
43	0.8	29.0	378.5	16.3	0.5	21.4	277.6	11.9	50.4	656.0	28.2
56	0.8	27.9	364.0	20.4	0.5	21.4	277.6	15.5	49.3	641.6	35.9
69	0.8	27.3	355.7	24.5	0.5	21.4	277.6	19.2	48.7	633.3	43.7
82	0.8	27.0	352.0	28.9	0.5	21.4	277.6	22.8	48.4	629.6	51.6
95	0.8	27.0	351.5	33.4	0.5	21.4	277.6	26.4	48.4	629.1	59.8
108	0.8	27.2	353.2	38.1	0.5	21.4	277.6	30.0	48.5	630.8	68.1
121	0.8	27.4	356.3	43.1	0.5	21.4	277.6	33.6	48.8	633.9	76.7
134	0.8	27.7	360.3	48.3	0.5	21.4	277.6	37.2	49.1	637.9	85.5
147	0.8	28.1	364.9	53.6	0.5	21.4	277.6	40.8	49.4	642.4	94.4
160	0.8	28.4	369.7	59.1	0.5	21.4	277.6	44.4	49.8	647.2	103.6
173	0.8	28.8	374.6	64.8	0.5	21.4	277.6	48.0	50.2	652.2	112.8
186	0.8	29.2	379.5	70.6	0.5	21.4	277.6	51.6	50.5	657.1	122.2
199	0.8	29.6	384.4	76.5	0.5	21.4	277.6	55.2	50.9	661.9	131.7
212	0.8	29.9	389.1	82.5	0.5	21.4	277.6	58.8	51.3	666.7	141.3
225	0.8	30.3	393.6	88.6	0.5	21.4	277.6	62.5	51.6	671.4	151.1
238	0.8	30.6	398.3	94.8	0.5	21.4	277.6	66.1	52.0	675.9	160.9
251	0.8	31.0	402.6	101.1	0.5	21.4	277.6	69.7	52.3	680.2	170.7
264	0.8	31.3	406.9	107.4	0.5	21.4	277.6	73.3	52.7	684.4	180.7
277	0.8	31.6	410.9	113.8	0.5	21.4	277.6	76.9	53.0	688.5	190.7
290	0.8	31.9	414.9	120.3	0.5	21.4	277.6	80.5	53.3	692.5	200.8
303	0.8	32.2	418.7	126.9	0.5	21.4	277.6	84.1	53.6	696.3	211.0
316	0.8	32.5	422.4	133.5	0.5	21.4	277.6	87.7	53.8	700.0	221.2
329	0.8	32.6	426.0	140.2	0.5	21.4	277.6	91.3	54.1	703.6	231.5
342	0.8	33.0	429.5	146.9	0.5	21.4	277.6	94.9	54.4	707.1	241.8
355	0.8	33.3	432.9	153.7	0.5	21.4	277.6	98.5	54.6	710.4	252.2
366	0.8	33.6	436.1	160.5	0.5	21.4	277.6	102.1	54.9	713.7	262.7
381	0.8	33.8	439.3	167.4	0.5	21.4	277.6	105.8	55.1	716.9	273.1
395	0.8	34.0	220.8	87.0	0.5	21.4	138.8	54.7	55.4	359.6	141.7
395	0.8	0.0	—	—	0.5	0.0	—	—	0.0	—	—
395	0.8	0.0	—	—	0.5	0.0	—	—	0.0	—	—
395	0.8	0.0	—	—	0.5	0.0	—	—	0.0	—	—
395	0.8	0.0	—	—	0.5	0.0	—	—	0.0	—	—
		Totals:	12046.8	2422.8		Totals:	8412.8	1657.3	Totals:	20459.6	4080.1

TABLE 1.15 Buildings' Characteristics and Wind Environment

Problem #	Exposure category	Basic wind speed at exposure C V, mph	Height h, ft	Base B, ft	Depth L, ft	Frequency Hz (period, sec)	Damping ratio β	Building density, slugs/cu ft[a]
1	A	90	600	100	100	0.2 Hz (5 sec)	0.01	0.3727
2	C	90	600	100	100	0.2 Hz (5 sec)	0.01	0.3727
3	B	120	394	98.5	164	0.222 Hz (4.5 sec)	0.01	0.287
4	C	130	788	164	164	0.125 Hz (8 sec)	0.015	0.3346

[a] 1 slug = 32.17 lbs.

The two along-wind responses—the along-wind displacements and along-wind accelerations of typical buildings—are due entirely to the action of the turbulence of the longitudinal component of the wind velocity, superimposed on their corresponding mean values. As discussed presently, the most important criterion for the comfort of the building's occupants is the peak or maximum accelerations they are likely to experience in a windstorm. Human perception of building motion is influenced by many cues, such as the movement of suspended objects; noise due to ruffling between building components; and, if the building twists, apparent movement of objects at a distance viewed by the occupants. Although at present there are no comprehensive comfort criteria, a generally accepted benchmark value in North American practice is to limit the acceleration at the upper floor of a building to 20 milli-g. This limit applies to both human comfort and motion perception.

A windstorm postulated to occur at a frequency of once every 10 years is used as the design event. The threshold of accelerations for residential occupancies is somewhat more stringent—about 15 milli-g for a 10-year windstorm. The rationale is that occupants are likely to remain longer in a given location of a residence, than in a typical office setting.

With this background, it is of interest to evaluate the gust effect factors and along-wind responses for some example buildings. Four buildings are considered here. Example 1 is a building located in wind terrain exposure category A, a category that is no longer recognized in ASCE 7-02, but included in its commentary as a numerical example. We use the results of this example solely to compare the wind response characteristics with the three other buildings.

Table 1.15 gives in summary form the buildings' characteristics and their wind environment. Given in Table 1.16 are the values of various parameters such as $z_{\min}$, $\bar{\varepsilon}$, etc., obtained from ASCE 7-02 Table 6.2 (Table 1.9 of this text). These values serve as starting points for the determination of gust effect factor, maximum lateral displacements, and accelerations.

Instead of presenting all examples in excruciating detail, only the final values of the derived parameters (as many as 24 for each example) are given in Table 1.17. However, for Building No. 3, the worked example follows the step-by-step procedure using the worksheet introduced in Section 1.4.2.7.

Discussions of Results. Because the example buildings are chosen randomly, it is impractical to make a comprehensive qualitative comparison. However, it may be appropriate to record the following observations regarding their wind-induced response characteristics.

TABLE 1.16 Design Parameters for Example Buildings

Problem #	1	2	3	4
z_{min}	60 ft	15 ft	30 ft	15 ft
$\bar{\varepsilon}$	0.5	0.2	0.333	0.2
c	0.45	0.20	0.30	0.20
$\bar{b}$	0.3	0.65	0.45	0.65
$\bar{\alpha}$	0.33	0.1538	0.25	0.1538
$\bar{b}$	0.64	1.0	0.84	1.0
$\bar{\alpha}$	0.2	0.1053	0.143	0.1053
ℓ	180	500	320	500
C_{fx}	1.3	1.3	1.3	1.3
ξ	1	1.0	1.0	1.0

(Values obtained from ASCE 7-02, Table 6.2, or Table 1.9 of this text.)

- Building No. 1 has a rather large height-to-width ratio of 600/100 = 6. Yet, because it is located in exposure category A, the most favorable wind terrain (per ASCE 7-98), and is subjected to a relatively low wind velocity of 90 mph, its lateral response to wind does not appear to be overly sensitive. The calculated acceleration at the top floor is 26 milli-g, as compared to the threshold value of 20 milli-g (2% of g).

TABLE 1.17 Comparison of Dynamic Response to Wind Loads

Calculated values	Problem 1	Problem 2	Problem 3	Problem 4
V	132 ft/s	132 ft/sec	176 ft/sec	191 ft/sec
$\bar{z}$	360	360 ft	236 ft	473 ft
$I_{\bar{z}}$	0.302	0.1343	0.216	0.128
$L_{\bar{z}}$	594.52 ft	806 ft	616 ft	852 ft
Q^2	0.589	0.634	0.64	0.596
$\bar{V}_{\bar{z}}$	87.83 ft/s	124 ft/s	130 ft/s	187 ft/s
$\hat{V}_{\bar{z}}$	136.24 ft/s	170 ft/s	195 ft/s	253 ft/s
N_1	1.354	1.30	1.051	0.5695
R_n	0.111	0.114	0.128	0.171
η	1.047	0.742	0.773	0.504
R_B	0.555	0.646	0.6360	0.74
η	6.285	4.451	3.095	2.423
R_h	0.146	0.1994	0.271	0.328
η	3.507	2.484	4.31	1.688
R_L	0.245	0.322	0.205	0.423
R^2	0.580	1.00	1.381	2.01
G	1.055	1.074	1.20	1.204
K	0.502	0.50	0.501	0.50
m_1	745,400 slugs	745,400 slugs	608,887 slugs	236,4000 slugs
g_R	3.787	3.787	3.813	3.66
X_{max}	0.78 ft	1.23 ft	1.16 ft	5.3 ft
$g_{\ddot{x}}$	3.786	3.786	3.814	3.66
$\sigma_{\ddot{x}}$	0.19	0.22	0.363	0.463
$\ddot{X}_{max}$	0.72 ft/sec^2 (22.39 milli-g)	0.834 ft/sec^2 (26 milli-g)	1.385 ft/sec^2 (43 milli-g)	1.68 ft/sec^2 (52.26 milli-g)

- Building No. 2 has the same physical characteristics as Building No. 1, but now is sited in exposure category C, the second most severe exposure category, consisting of open terrain with scattered obstructions. Because the basic wind is the same as for the first, (90 mph at exposure C), its peak acceleration is only slightly higher than for Building 1. Pushing exposure category from A to C does not appear to unduly alter the wind sensitivity of the building.
- Building No. 3, in contrast to the other three, is not that tall. It is only 394 ft, in height, equivalent to a 30-story office building at a floor-to-floor height of 12 ft-6 in. At a fundamental frequency of 4.5 sec, its lateral stiffness is quite in line with buildings designed in high seismic zones. But because it is subjected to hurricane winds of 120 mph, its peak acceleration is a head-turning 43 milli-g—more than twice the threshold value of 20 milli-g.
- Building No. 4 is the tallest of the four, equivalent to a 60-plus story building. Its height-to-width ratio is not very large (only 4.8) but it appears to be quite flexible at a fundamental frequency of 8 sec and a calculated peak acceleration of 52.26 milli-g. It is doubtful that even with the addition of a supplemental damping system such as a tuned mass damper (TMD) or a simple pendulum damper (discussed in Chapter 8), the building oscillations can be tamed. Consulations with an engineering expert who specializes in performing wind-tunnel tests and in designing damping systems for dynamically sensitive structures would be recommended before finalizing the structural system.

Although in North American practice, the determination of wind-motion characteristics of buildings is primarily the domain of wind engineering consultants, the author strongly recommends that an analytical study be undertaken, as given in this section. The result will help in communication with owners of buildings, architects, and wind engineering experts in identifying problems associated with motion perception and human comfort.

Building No. 3: Calculations for Gust Effect Factor, Maximum Along-Wind Deflection, and Acceleration.

Given.

Building height h	= 394 ft
Building depth L	= 164 ft
Building width B	= 98.5 ft
Building natural frequency	= 0.222 Hz (period $T = 4.5$ s)
Damping ratio	= 0.01
Building density	= 0.287 slugs/ft^3
Exposure category	= B
Basic wind speed	= 120 mph
Air density	= 0.0024 slugs/cu ft
Mode exponent φ	= 1.0
Coefficient C_{fx}	= 1.3

Required.

Gust factor G

Maximum lateral load deflection at top, $X_{\max\ (h)}$

Maximum along-wind acceleration top, $\ddot{X}_{\max(h)}$

Solution.

ASCE 7-02 Formulas	Commentary
V = wind speed in ft/s $= V_{\text{mph}} \times 1.467 = 120 \times 1.467 = 176$ ft/s	V = basic wind speed = 120 mph, given
$\bar{z} = 0.6\,h = 0.6 \times 394 = 236\ \text{ft} > \bar{z}_{\min} = 15\ \text{ft}$	h = building height = 394 ft; $\bar{z}_{\min} = 15$ ft from Table 6.4
$I_z = C\left(\dfrac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\dfrac{33}{236}\right)^{1/6} = 0.216$	$c = 30$ from Table 6.4
$L_{\bar{z}} = l\left(\dfrac{\bar{z}}{33}\right)^{\varepsilon} = 320\left(\dfrac{236}{33}\right)^{0.333} = 616$	$l = 320$ ft, $\varepsilon = 1/5$ both from Table 6.4
$Q = \sqrt{\dfrac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\dfrac{1}{1 + 0.63\left(\frac{98.5+394}{616}\right)^{0.63}}} = 0.804$	Q = is called background response; B = building width perpendicular to wind = 98.5 ft, given
$\bar{V}_{\bar{z}} = \bar{b}\left(\dfrac{\bar{z}}{33}\right)^{\bar{\alpha}} V = 0.45\left(\dfrac{236}{33}\right)^{0.25} \times 176 = 130$ ft/s	$\bar{b} = 0.65$, $\bar{\alpha} = 1/6.5$, both from Table 6.4
$\hat{V}_{\bar{z}} = \hat{b}\left(\dfrac{\bar{z}}{33}\right)^{\hat{\alpha}} V = 0.84\left(\dfrac{236}{33}\right)^{0.142} \times 176 = 195$ ft/s	$\hat{b} = 1.0$, $\hat{\alpha} = 1.0$, both from Table 6.4
$N_1 = \dfrac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \dfrac{0.222 \times 616}{130} = 1.051$	n_1 = natural frequency of the building = 0.222 Hz, given

$$R_n = \frac{7.47\, N_1}{(1 + 10.3\, N_1)^{5/3}}$$
$$= \frac{7.47 \times 1.051}{(1 + 10.3 \times 1.051)^{5/3}}$$
$$= 0.128$$

$$\eta_b = \frac{4.6\, n_1 B}{\bar{v}_z}$$
$$= \frac{4.6 \times 0.222 \times 98.5}{130}$$
$$= 0.773$$

$$R_B = \frac{1}{\eta_b} - \frac{1}{2\eta_b^2}(1 - e^{-2\eta_b})$$
$$= \frac{1}{0.773} - \frac{1}{2 \times 0.773^2}(1 - 2.71^{-2\times 0.773})$$
$$= 0.6370$$

$e = 2.71$

$$\eta_h = \frac{4.6\, n_1 h}{\bar{v}_{\bar{z}}}$$
$$= \frac{4.6 \times 0.222 \times 394}{130}$$
$$= 3.095$$

$$R_h = \frac{1}{\eta_h} - \frac{1}{2\eta_h^2}(1 - e^{-2\eta_h})$$
$$= \frac{1}{3.095} - \frac{1}{2 \times 3.095^2}(1 - 2.71^{-2\times 3.095})$$
$$= 0.271$$

$$\eta_L = \frac{15.4\, n_1 L}{\bar{v}_{\bar{z}}}$$
$$= \frac{15.4 \times 0.222 \times 164}{130}$$
$$= 4.31$$

L = building breadth parallel to wind = 164 ft given

$$R_L = \frac{1}{\eta_L} - \frac{1}{2\eta_L^2}(1 - e^{-2\eta})$$
$$= \frac{1}{4.31} - \frac{1}{2 \times 4.31^2}(1 - 2.71^{-2\times 4.31})$$
$$= 0.205$$

$$R^2 = \frac{1}{\beta} R_n R_h R_B (0.53 + 0.47\, R_L)$$
$$= \frac{1}{0.01} \times 0.128 \times 0.271 \times 0.6360(0.53 + 0.47 \times 0.205)$$
$$= 1.381$$

β = damping ratio = 0.015, given

$$R = \sqrt{1.381} = 1.175$$

$$g_R = \sqrt{2 \ln(3600\, n_1)} + \frac{0.577}{\sqrt{2 \ln(3600\, n_1)}}$$
$$= \sqrt{2 \ln(3600 \times 0.222)} + \frac{0.577}{\sqrt{2 \ln(3600 \times 0.222)}}$$
$$= 3.813$$

ln means logarithm to base $e = 2.71$.

G_f is gust effect factor

$$\begin{aligned} G_f &= 0.925\left(\frac{1 + 1.7\, I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7\, g_v I_{\bar{z}}}\right) \\ &= 0.925\left(\frac{1 + 1.7 \times 0.216\sqrt{3.4^2 \times 0.64 + 3.813^2 \times 1.381}}{1 + 1.7 \times 3.4 \times 0.216}\right) \\ &= 1.203 \end{aligned}$$

This is the gust factor required for calculating design wind pressures for the main wind-force-resisting system of the building.

$$\begin{aligned} m_1 &= \mu_z \times \frac{h}{3} \\ &= 4636 \times \frac{394}{3} \\ &= 608,887 \text{ slugs} \end{aligned}$$

ξ = first mode exponent = 1.0

$$\begin{aligned} K &= \frac{1.65^{\hat{\alpha}}}{(\hat{\alpha} + \xi + 1)} \\ &= \frac{1.65^{0.142}}{(0.142 + 1 + 1)} \\ &= 0.501 \end{aligned}$$

Maximum Along-Wind Displacement.

$$\begin{aligned} X_{\max(z)} &= \frac{\phi_{(z)}\rho B h C_{fx}\hat{V}_{\bar{z}}^2}{2m_1(2\pi n_1)^2} KG \\ &= \frac{1 \times 0.0024 \times 98.5 \times 394 \times 1.3 \times 195^2}{2 \times 608,887(2\pi \times 0.222)^2} \times 0.501 \times 1.203 \end{aligned} \tag{1.47}$$

= 1.16 ft. This is the maximum deflection at the building top due to wind pressures.

$$\begin{aligned} g_{\ddot{x}} &= \sqrt{2\ln(n_1 T)} + \frac{0.5772}{\sqrt{2\ln(n_1 T)}} \\ &= \sqrt{2\ln(0.222 \times 3600)} + \frac{0.5772}{\sqrt{2\ln(0.222 \times 3600)}} \\ &= 3.6567 + 0.1579 \\ &= 3.813 \text{ (Note: } g_{\ddot{x}} \text{ is the same as } g_R) \end{aligned}$$

$$\begin{aligned} \sigma_{\ddot{x}(z)} &= \frac{0.85\phi_z \rho B h C_{fx}\bar{V}_{\bar{z}}^2}{m_1} I_{\bar{z}} K R \\ &= \frac{0.85 \times 0.0024 \times 98.5 \times 394 \times 1.3 \times 130^2}{608,887} \times 0.216 \times 0.501 \times 1.175 \\ &= 0.363 \end{aligned} \tag{1.48}$$

Maximum Along-Wind Acceleration.

$$
\begin{aligned}
X_{\max(z)} &= g_{\ddot{x}}\sigma_{\ddot{x}(z)} \\
&= 3.814 \times 0.363 \\
&= 1.384 \text{ ft/sec}^2 \\
&= 1.384 \times 31.11 \text{ milli-g} \\
&= 43 \text{ milli-g} \quad \text{This is the maximum along-wind acceleration}
\end{aligned} \tag{1.49}
$$

This is more than twice the threshold value of 20 milli-g. Therefore, it is important to verify the results by conducting wind tunnel tests. If the test results confirm the likelihood of the building experiencing high accelerations, adding dampers to the building to reduce building oscillations is an option.

This is discussed in Chapter 8.

1.4.2.9. Formal Definitions of Exposure Categories

Exposure B: Exposure B applies to urban and suburban areas or other terrain with numerous closely spaced obstructions the size of single-family dwellings or larger. To appropriately assign this exposure category, this topography must prevail in the upwind direction for a distance of at least 2630 ft (800 m) or 10 times the height of the buildings, whichever is greater.

Exposure C: Exposure C applies to terrain that consists of scattered obstructions of height generally less than 30 ft (9.1 m). This category includes flat open country, grasslands, and all water surfaces in hurricane-prone regions. It also applies to all areas where exposures B and D do not apply.

Exposure D: Exposure D consists of unobstructed areas and water surfaces outside hurricane-prone regions. This category includes smooth mud flats, salt flats, and unbroken ice extending in the upwind direction for a distance of at least 5000 ft (1524 m) or 10 times the buildings' height, whichever is greater. Exposure D extends inland from the shoreline for a distance of 660 ft (200 m) or 10 times the height of the building, whichever is greater.

The proper assessment of exposure is a matter of good engineering judgment, particularly because the exposure may change in one, wind direction or more as a result of future development and/or demolition. Figures 1.13a–e are aerial photographs representative of some of the exposure types.

1.4.3. National Building Code of Canada (NBCC 1995): Wind Load Provisions

The reader may be wondering why, after an arguably extensive coverage of the ASCE 7-02 wind load provisions, the author would burden the text with yet another building code provision. The reason is simple: Although extensive in its treatment of wind, the ASCE 7-02 does not provide an analytical procedure for estimating across-wind response of tall, flexible buildings. To the best of the author's knowledge, NBCC is the only code in North America that presents an analytical method for computing across-wind response. It is perhaps the most comprehensive standard for wind because it takes into consideration characteristics such as building dimensions, shape, stiffness, damping ratios, site topography, climatology, boundary layer meteorology, bluff body aerodynamics, and probability theory.

Figure 1.13a. Exposure B; suburban residential area predominated by single-family dwellings.

Figure 1.13b. Exposure B; urban area with numerous closely spaced buildings the size of single-family homes or larger.

Figure 1.13c. Exposure B; urban area with numerous closely spaced buildings the size of single-family homes or larger.

Figure 1.13d. Structure in the foreground is located in exposure B. Structures in the rear, adjacent to the clearing, are located in exposure C when wind flows from the left over the clearing.

Figure 1.13e. Structures on a shoreline with wind flowing over open water for a distance of at least one mile are located in exposure D.

Three different approaches for determining wind loads on buildings are given: 1) simple procedure; 2) experimental procedure; and 3) detailed procedure.

1.4.3.1. Simple Procedure

The simple procedure is applicable for determining structural wind loads for a majority of low- and medium-rise buildings and also for cladding design of low-, medium-, and high-rise buildings. The method is similar to other code approaches in which the dynamics action of wind is dealt with by equivalent static loads defined independently of the dynamic properties of wind.

The recurrence intervals used for evaluating wind loads are

1. 1 in 10 years for the design of cladding and structural members designed for deflection and vibration limits.
2. 1 in 30 years for the design of structural members of all, except post-disaster buildings, for strength.
3. 1 in 100 years for the design of structural members of post-disaster buildings for strength.

The external pressure or suction on the building surface is given by the equation

$$P = qC_eC_gC_p \tag{1.50}$$

where

P = design static pressure or suction, acting normal to the surface: kilo pascals

q = reference velocity pressure; kilo pascals

C_e = exposure factor that reflects the changes in wind speed with height and variations in the surrounding terrain: dimensionless

C_g = gust factor, with a value of 2.0 for the primary structural system, and 2.50 for cladding: dimensionless

C_p = external pressure coefficient averaged over the area of the surface considered: dimensionless

Reference Pressure *q*. The reference velocity pressure q, in kilo pascals, is determined from referenced wind speed V by the equation:

$$q = C\overline{V}^2 \tag{1.51}$$

The factor C depends on the atmospheric pressure and air temperature. If the wind speed $\overline{V}$ is in meters per second, the design pressure, in kilo pascals, is obtained by using a value of $C = 650 \times 10^{-6}$. The reference wind pressure q, is given for three different levels of probability being exceeded per year (1/10, 1/30, and 1/100), that is, for return periods for 10, 30, and 100 years, respectively. A 10-year recurrence pressure is used for the design of cladding an for the serviceability check of structural members for deflection and vibration. A 30-year wind pressure is used for the strength design of structural members of all buildings except those classified as post-disaster buildings. A 100-year wind is used for the design of post-disaster buildings such as hospitals, fire stations, etc. The 10-, 30-, and 100-year mean hourly wind pressures in Montreal, Quebec are 0.31 kPa (6.5 psf), 0.37 kPa (7.72 psf), and 0.44 kPa (9.2 psf), respectively, with corresponding wind speeds of 22 m/s (49.2 mph), 24 m/s (54 mph), and 26 m/s (58 mph).

Exposure Factor C_e. The exposure factor C_e is based on the 1/5 power law corresponding to wind gust pressures in open terrain. An averaging period of 3 to 5 seconds is used in determining the gust factor. It represents a 'parcel' of wind assumed to be effective over the entire building. For tall buildings, the reference height for pressures on the windward face corresponds to the actual height aboveground, and for suctions on the leeward face, the reference height is half the height of the structure.

The exposure factor C_e reflects the changes in wind speed and height, and the effects of variations in the surrounding terrain and topography. Hills and escarpments that can significantly amplify wind speeds are reflected in the exposure factor.

The exposure factor C_e may be obtained from any of the following three methods:

1. The value shown in Table 1.
2. The value of the function $(h/10)^{\frac{1}{5}}$ but not less than 0.9, where h is the reference height above grade, in meters.
3. If a dynamic approach is used, an appropriate value depending on both the height and shielding.

Gust Effect Factor (Dynamic Response Factor) C_g. This factor accounts for the increase in the mean wind loads due to the following factors:

- Random wind gusts acting for short durations over entire or part of structure.
- Fluctuating pressures induced in the wake of a structure, including vortex shedding forces.
- Fluctuating forces induced by the motion of a structure.

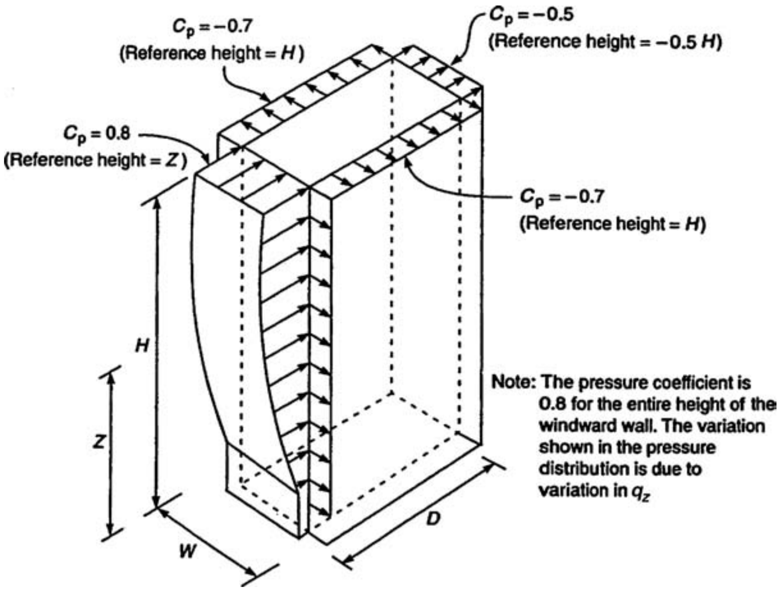


Figure 1.14. External wind pressure coefficient C_p; flat-roofed buildings $H > W$. (Adapted from NBCC 1995.)

All buildings are affected to some degree by their dynamic response. The total response may be considered as a summation of the mean component without any structural dynamic magnification, and a resonant component due to building vibrations close to its natural frequency. For the majority of buildings less than 120 m (394 ft) tall, and with height-to-width ratio less than 4, the resonant component is small. The only added loading is due to gusts that can be dealt with in a simple static manner.

For buildings and components that are not particularly tall, long, slender, lightweight, flexible, or lightly damped, a simplified set of dynamic gust factors is given as follows:

$C_g = 2.5$ for building components and cladding

$C_g = 2.0$ for the primary structural system including anchorages to foundation

Pressure coefficient C_p. C_p is a nondimensional ratio of wind-induced pressure on a building to the velocity pressure of the wind speed at the reference height (see Fig. 1.14). It depends on the shape of the building, wind direction, and profile of the wind velocity, and can be determined most reliably from wind-tunnel tests. However, for the simple procedure, based on some limited measurements on full-scale buildings supplemented by wind-tunnel tests, NBC gives the following values of C_p for simple building shapes:

Windward wall:	$C_p = +0.8$ (positive pressure)
	Reference height = Z aboveground
Side wall and roof:	$C_p = -1.0$ (negative pressure, suction)
	Reference height = H aboveground
Leeward wall:	$C_p = -0.5$ (negative pressure, suction)
	Reference pressure = $0.5H$ aboveground

1.4.3.2. Experimental Procedure

The second approach is to use the results of wind-tunnel or other experimental procedures for buildings likely to be susceptible to wind-induced vibrations. Included in this category are tall, slender structures for which wind loading plays a major role in the structural design. A wind-tunnel test is also recommended for determining exterior pressure coefficients for cladding design of buildings whose geometry deviates markedly from more common shapes for which information is already available.

1.4.3.3. Detailed Procedure

In this method, a series of calculations is performed to determine more accurate values for the gust factor C_g, the exposure factor C_e, and the pressure coefficient C_p. The end product of the calculations yields a static design pressure, which is expected to produce the same peak effect as the actual turbulent wind, with due consideration for building properties such as height, width, natural frequency of vibration, and damping. This approach is primarily for determining the overall wind loading and response of tall slender structures, and is not intended for determining exterior pressure coefficients for cladding design.

The code gives procedures for calculating the dynamic effects of vortex shedding for slender cylindrical towers and for tapered structures. Since the available data are limited for slender structures with cross sections other than circular, wind-tunnel tests are recommended for estimating the likely response. To limit the cracking of masonry and interior finishes, the total drift per story under specified wind and gravity loads is limited to 1/500 of the story height, unless a detailed analysis is made and precautions taken to permit larger movements.

The code recognizes that maximum accelerations of a building leading to possible human perception of motion or discomfort may occur in a direction perpendicular to the wind. A tentative acceleration limit of 1 to 3% of gravity for a 10-year return wind is recommended to limit the possibility of perception of motion.

Exposure Factor C_e. The exposure factor C_e is based on the mean wind speed profile, which depends on the roughness of terrain over which the wind has traveled before reaching the building. Three wind profile categories are used in building design.

Exposure A. This is the exposure on which the reference wind speeds are based. The exposure is defined as open, level terrain with only scattered buildings, trees or other obstructions, and open water or shorelines. C_e is given by

$$C_e = \left(\frac{z}{10}\right)^{0.28}, \qquad C_e \geq 1.0 \tag{1.52}$$

Exposure B. Suburban and urban areas, wooded terrain, or centers of large towns with terrain roughness extending in the upwind direction for at least 1.5 km. C_e is given by

$$C_e = 0.5\left(\frac{z}{12.7}\right)^{0.50}, \qquad C_e \geq 0.5 \tag{1.53}$$

Exposure C. Centers of large cities with heavy concentrations of buildings extending in the upwind direction for at least 1.5 km, with at least 50% of the buildings exceeding four stories in height. C_e is given by

$$C_e = 0.4\left(\frac{z}{30}\right)^{0.72}, \qquad C_e \geq 0.4 \tag{1.54}$$

Exposure factor C_e can be calculated from Eq. (1.54) or obtained directly from the graph in Fig. 1.15.

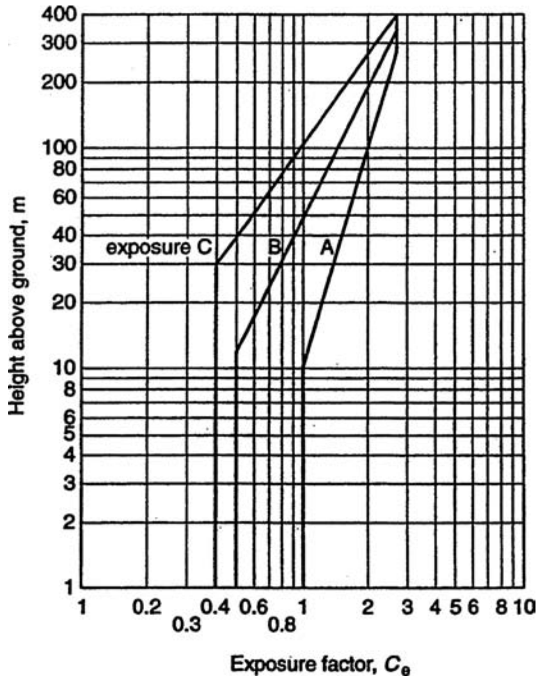


Figure 1.15. Exposure factor C_e as a function of terrain roughness and height aboveground. (From NBCC 1995.)

Gust Effect Factor C_g (Detailed Procedure). A general expression for the maximum or peak load effect, denoted W_p, is given by

$$W_p = \mu + g_p \sigma \tag{1.55}$$

where

μ = the mean loading effect
σ = the root-mean square loading effect
g_p = a peak factor for the loading effect

The dynamic gust response factor is defined as the ratio of peak loading to mean loading,

$$\begin{aligned} C_g &= W_p/\mu \\ &= 1 + g_p\left(\frac{\sigma}{\mu}\right) \end{aligned} \tag{1.56}$$

The parameter σ/μ is given by the expression

$$\frac{\sigma}{\mu} = \sqrt{\frac{K}{C_{eH}}\left(B + \frac{sF}{\beta}\right)} \tag{1.57}$$

where

K = a factor related to the surface roughness coefficient of the terrain
K = 0.08 for exposure A
K = 0.10 for exposure B
K = 0.14 for exposure C