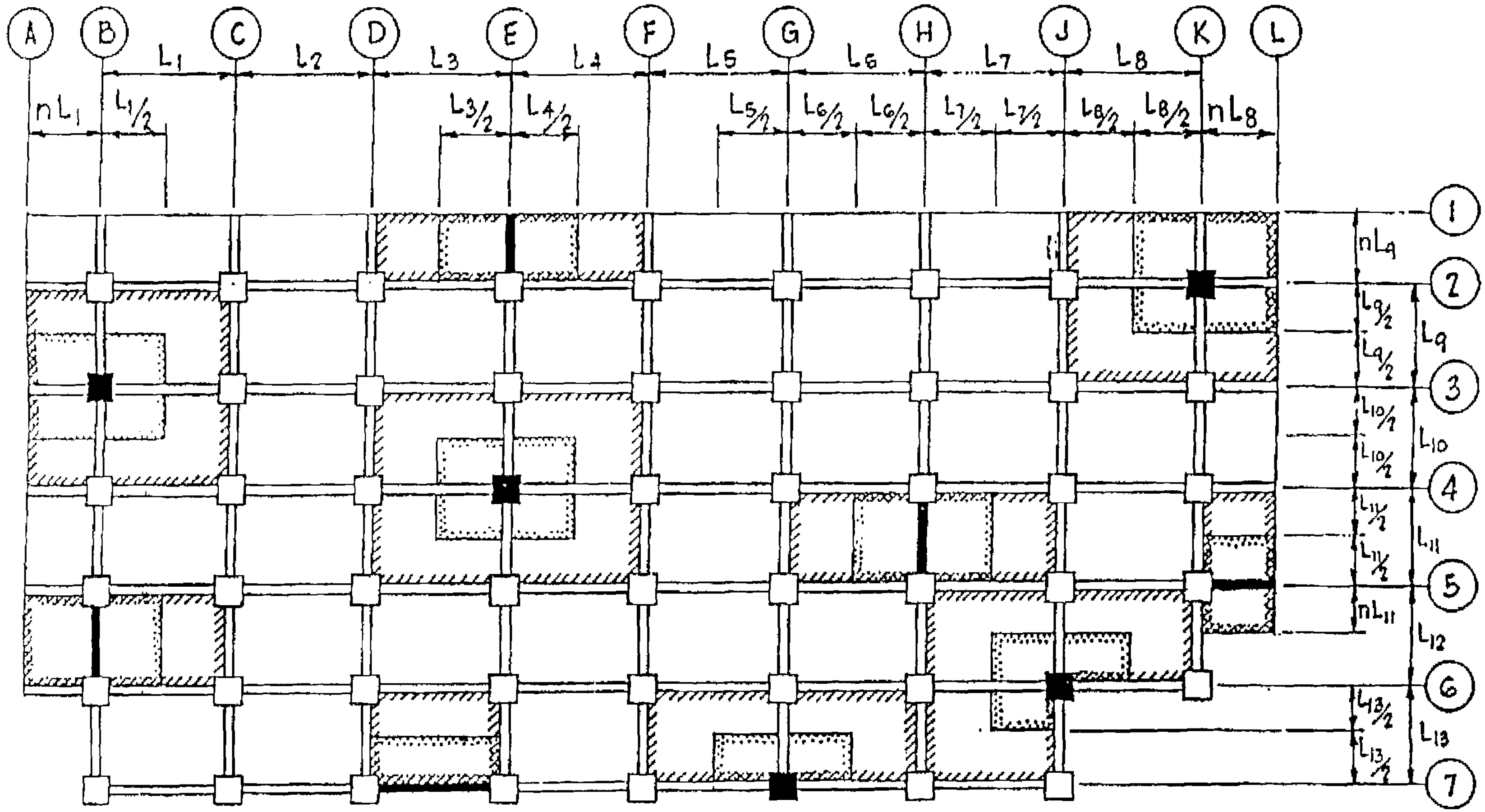

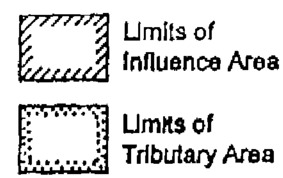

Element	Actual KLL Members w/o Cantilevers	Actual KLL Members w/ cantilevers n = 0	n = 0.5	n = 1.0	KLL (Table 4-2)	Example Member (Fig. C4)
Interior Columns	4	-	-	-	4	E4
Exterior Columns w/o cantilevers	4	-	-	-	4	G7, J8
Edge Columns w/ cantilevers	-	4	3	2.67	3	B3
Corner Columns w/ cantilevers	-	4	2.25	1.78	2	K2
Edge Beams w/o cantilever slabs	2	-	-	-	2	D7 - E7
Interior Beams	2	-	-	-	2	H4 - H5
Edge Beams w/ cantilever slabs	-	2	1.5	1.33	1	B5 - B6
Cantilever Beams w/o cantilever slabs	2	-	-	-	1	E1 - E2

* The value of n for member K5-L5 is used to calculate the distance nL_{11}

FIG. C4.

C5. Soil and Hydrostatic Pressure and Flood Loads

C5.1 Pressure on Basement Walls. Table 5-1 includes high earth pressures, 85 pcf (13 36 kN/m^2) or more, to show that certain soils are poor backfill material In addition, when walls are unyielding the earth pressure is increased from active pressure toward earth pressure at rest, resulting in 60 pcf (9 43 kN/m^2) for granular soils and 100 pcf (15.71 kN/m^2) for silt and clay type soils (see reference [4]). Examples of light floor systems supported on shallow basement walls mentioned in Table 5-1 are floor systems with wood joists and flooring, and cold formed steel joists without cast in place concrete floor attached.

Expansive soils exist in many regions of the United States and may cause serious damage to basement walls unless special design considerations are provided Expansive soils should not be used as backfill because they can exert very high pressures against walls Special soil testing is required to determine the magnitude of these pressures It is preferable to excavate expansive soil and backfill with non-expansive freely-draining sands or gravels The excavated backslope adjacent to the wall should be no steeper than 45 degrees from the horizontal in order to minimize the transmission of swelling pressure from the expansive soil through the new backfill Other special details are recommended, such as a cap of non-pervious soil on top of the backfill and provision of foundation drains Refer to current reference books on geotechnical engineering for guidance

C5.2 Uplift on Floors and Foundations. If expansive soils are present under floors or footings large pressures can be exerted and must be resisted by special design. Alternatively, the expansive soil can be excavated to a depth of at least two feet (0 60 m) and backfilled with non-expansive freely-draining sands or gravel. A geotechnical engineer should make recommendations in these situations

C5.3 Flood Loads. This section presents information for the design of buildings and other structures in areas prone to flooding. Much of the impetus for flood-resistant design has come about from the federal government sponsored initiatives of flood insurance and flood-damage mitigation

The National Flood Insurance Program (NFIP) is based on an agreement between the federal government and participating communities that have been identified as being floodprone The Federal Emergency Management Agency (FEMA) through the Federal Insurance Administration (FIA), makes flood insurance available to the residents of communities provided that the community adopts and enforces adequate floodplain management regulations that meet the minimum requirements. Included in the NFIP requirements, found under Title 44 of the U S Code of Federal Regulations, are minimum building design and construction standards for buildings and other structures located in Special Flood Hazard Areas (SFHA)

Special Flood Hazards Areas (SFHA) are those identified by FEMA's Mitigation Directorate as being subject to inundation during the 100 year flood SFHA are shown on Flood Insurance Rate Maps (FIRM), which are produced for floodprone communities. SFHA are identified on FIRM as zones A, A1-30, AE, AR, AO, AH and coastal high hazard areas as V1-30, V and VE The SFHA is the area in which communities must enforce NFIP-complaint, flood damage resistant design and construction practices.

Prior to designing a structure in a floodprone area, design professionals should contact the local building official to determine if the site in question is located in a SFHA or other floodprone area that is regulated under the community's floodplain management regulations If the proposed structure is located within the regulatory floodplain, local building officials can explain the regulatory requirements.

Answers to specific questions on flood-resistant design and construction practices may be directed to the Mitigation Division of each of FEMA's regional offices. FEMA has regional offices that are available to assist design professionals

C.5.3.1 Definitions. Three new concepts are added with ASCE 7-98 First, the concept of the Design Flood is introduced. The Design Flood will, at a minimum, be equivalent to the flood having a one-percent chance of being equaled or exceeded in any given year (i e., the Base Flood or 100-year flood, which served as the load basis in ASCE 7-95) In some instances, the Design Flood may exceed the Base Flood in elevation or spatial extent – this will occur where a community has designated a greater flood (lower frequency, higher return period) as the flood to which the community will regulate new construction

Many communities have elected to regulate to a flood standard higher than the minimum requirements of the National Flood Insurance Program (NFIP). Those communities may do so in a number of ways. For example a community may require new construction to be elevated a specific vertical distance above the Base Flood Elevation (this is referred to as freeboard), a community may select a lower frequency flood as its regulatory flood, a community may conduct hydrologic and hydraulic studies, upon which Flood Hazard Maps are based, in a manner different than the Flood Insurance Study prepared by the NFIP (the community may complete flood hazard studies based upon development conditions at build-out, rather than following the NFIP procedure which uses conditions in existence at the time the studies are completed; the community may include watersheds smaller than one square mile in size in its analysis, rather than following the NFIP procedure which neglects watersheds smaller than one square mile)

Use of the Design Flood concept in ASCE 7-98 will ensure that the requirements of this standard are not less restrictive than a community's requirements where that community has elected to exceed minimum NFIP requirements In instances where a community has adopted the NFIP minimum requirements, the Design Flood described in this standard will default to the Base Flood.

Second, this standard also introduces the terms, Flood Hazard Area and Flood Hazard Map, to correspond to and show the areas affected by the Design Flood. Again in instances where a community has adopted the minimum requirements of the NFIP, the Flood Hazard Area defaults to the NFIP's Special Flood Hazard Area and the Flood Hazard Map defaults to the Flood Insurance Rate Map

Third, the concept of a Coastal A Zone is introduced, to facilitate application of revised load combinations contained in Section 2 Coastal A zones lie landward of V zones, or landward of an open coast shoreline where V zones have not been mapped (e g , the shorelines of the Great Lakes). Coastal A Zones are subject to the effects of waves, high-velocity flows and erosion, although not to the extent that V Zones are Like V zones, flood forces in coastal A zones will be highly correlated with coastal winds or coastal seismic activity.

C5.3.2.1 Design loads. Wind loads and flood loads may act simultaneously at coastlines, particularly during hurricanes and coastal storms. This may also be true during severe storms at the shorelines of large lakes, and during riverine flooding of long duration

C5.3.3.1 Load basis. Water loads are the loads or pressures on surfaces of buildings and structures caused and induced by the presence of floodwaters These loads are of two basic types: hydrostatic and hydrodynamic Impact loads result from objects transported by floodwaters striking against buildings and structures or part thereof. Wave loads can be considered a special type of hydrodynamic load

C5.3.3.2 Hydrostatic Loads. Hydrostatic loads are those caused by water either above or below the ground surface, free or confined, which is either stagnant or moves at velocities less than 5 feet per second (1 52 m/s) These loads are equal to the product of the water pressure multiplied by the surface area on which the pressure acts

Hydrostatic pressure at any point is equal in all directions and always acts perpendicular to the surface on which it is applied. Hydrostatic loads can be subdivided into vertical downward loads, lateral loads and vertical upward loads (uplift or buoyancy). Hydrostatic loads acting on inclined, rounded or irregular surfaces may be resolved into vertical downward or upward loads and lateral loads based on the geometry of the surfaces and the distribution of hydrostatic pressure.

C5 3.3.3 Hydrodynamic Loads. Hydrodynamic loads are those loads induced by the flow of water moving at moderate to high velocity above the ground level. They are usually lateral loads caused by the impact of the moving mass of water and the drag forces as the water flows around the obstruction Hydrodynamic loads are computed by recognized engineering methods. In the coastal high hazard area the loads from high velocity currents due to storm surge and overtopping are of particular importance. Reference [1] is one source of design information regarding hydrodynamic loadings.

Potential sources of information regarding velocities of floodwaters include local, state and federal government agencies and consulting engineers specializing in coastal engineering, stream hydrology or hydraulics.

C.5.3.3.4 Wave Loads. The magnitude of wave forces (lbs/sq.ft)(kN/m^2) acting against buildings or other structures can be 10 or more times higher than wind forces and other forces under design conditions Thus, it should be readily apparent that elevating above the wave crest elevation is crucial to the survival of buildings and other structures. Even elevated structures, however, must be designed for large wave forces that can act over a relatively small surface area of the foundation and supporting structure.

Wave load calculation procedures in section 5.3 3 4 are taken from References [1] and [5] The analytical procedures described by equations 5-2 through 5-10 should be used to calculate wave heights and wave loads unless more advanced numerical or laboratory procedures permitted by this Standard are used.

Wave load calculations using the analytical procedures described in this Standard all depend upon the initial computation of the wave height, which is determined using equations 5-2 and 5-3 These equations result from the assumptions that waves propagating into shallow water break when the wave height equals 78 percent of the local stillwater depth and that 70 percent of the wave height lies above the local stillwater level. These assumptions are identical to those used by FEMA in its mapping of coastal flood hazard areas on FIRMs.

It should be pointed out that present NFIP mapping procedures distinguish between A Zones and V Zones by the wave heights expected in each zone. Generally speaking, A Zones are designated where wave heights less than three feet (0.91 m) in height are expected; V Zones are designated where wave heights equal to or greater than three feet (0.91 m) are expected. Designers should proceed cautiously, however. Large wave forces can be generated in some A Zones, and wave force calculations should not be restricted to V Zones. Present NFIP mapping procedures do not designate V Zones in all areas where wave heights greater than three feet (0 91 m) can occur during base flood conditions. Rather than rely exclusively on flood hazard maps, designers should investigate historical flood damages near a site to determine whether or not wave forces can be significant

C.5.3.3.4.2 Breaking Wave Loads on Vertical Walls. Equations used to calculate breaking wave loads on vertical walls contain a coefficient, C_p. Reference [5] provides recommended values of the coefficient as a function of probability of exceedance. The probabilities given by Reference [5] are not annual probabilities of exceedance, but probabilities associated with a distribution of breaking wave pressures measured during laboratory wave tank tests. Note that the distribution is independent of water depth. Thus, for any water depth, breaking wave pressures can be expected to follow the distribution described by the probabilities of exceedance in Table 5-2.

This Standard assigns values for C_p according to Building Category, with the most important buildings having the largest values of C_p. Category II buildings are assigned a value of C_p corresponding to a 1% probability of exceedance, which is consistent with wave analysis procedures used by FEMA in mapping coastal flood hazard areas and in establishing minimum floor elevations. Category I buildings are assigned a value of C_p corresponding to a 50% probability of exceedance, but designers may wish to choose a higher value of C_p. Category III buildings are assigned a value of C_p corresponding to a 0 2% probability of exceedance, while Category IV buildings are assigned a value of C_p corresponding to a 0 1% probability of exceedance

Breaking wave loads on vertical walls reach a maximum when the waves are normally incident (direction of wave approach perpendicular to the face of the wall, wave crests are parallel to the face of the wall). As guidance for designers of coastal buildings or other structures on normally dry land (i e , flooded only during coastal storm or flood events), it can be assumed that the direction of wave approach will be approximately perpendicular to the shoreline. Therefore, the direction of wave approach relative to a vertical wall will depend upon the orientation of the wall relative to the shoreline. Section 5 3.3 4.4 provides a method for reducing breaking wave loads on vertical walls for waves not normally incident

C5.3.3.5 Impact Loads. Normal impact loads are those which result from isolated occurrences of logs, ice floes and other objects normally encountered striking buildings, structures or parts thereof. Special impact loads are those which result from large objects, such as broken up ice floats and accumulations of debris, either striking or resting against a building, structures or parts thereof. Extreme impact loads are those which result from very large objects such as boats, barges, or collapsed buildings striking the building, structure or component under consideration [2]

Impact load is calculated as follows:

$$F_i = ma \quad \text{(Eq C5-1)}$$

Where

F_i = impact load in pounds (kN)
$m = w/g$ = mass in slugs (kg) (Eq. C5-2)
w = weight of object in pounds (kN)
g = acceleration due to gravity 32 2 feet per second per second (9 81 m/s^2)
$a = \Delta V/\Delta t$ (Eq. C5-3)
ΔV = change from V_b to zero velocity
V_b = velocity of object in feet per second (m/s)
Δt = time to decelerate object in seconds

Assume that the velocity of the object is reduced to zero in one second resulting in a minimum impact load

$$F = 31 \times V_b \text{ in pounds} \quad \text{(Eq. C5-4)}$$

$$[\text{ in SI } F_i = 0\ 453 \times V_b \text{ in kN }]$$

Where larger than normal impact loads are likely to occur the minimum impact load presented in this section is inadequate and special consideration in the design of the building or other structure is required. For example, rigid structures of concrete or steel may reduce the velocity of the object to zero within a time interval of 0 1 to 0 5 second

C.5.3 4 Special Flood Hazard Areas – A Zone. Note that the elevation requirements specified in Table 5-3 should apply to A Zones identified on FIRMs, and to other community-identified flood hazard areas with a 1% or greater chance of flooding in any given year. Exception community-identified V Zones not identified as such on FIRMs.

C5.3 5 Coastal High Hazards Areas - V Zones. In coastal flood hazard areas the velocity of flow of the water erodes soil supporting the foundation elements, such as piles, piers, and columns. Adequate embedment must be provided to resist the effects of the design flood as well as the accumulated scour from a number of smaller flood events. Similar design requirements should be met in other flood hazard areas including coastal A zones, and along river areas and lakefronts where water depths, wind speeds and fetch lengths are sufficient to generate damaging waves and high velocity flows capable of causing erosion and scour

Note that the elevation requirements specified in Table 5-4 should apply to V Zones identified on FIRMs, and to other community-identified flood hazard areas subject to high velocity wave action, and with a 1% or greater chance of flooding in any given year

C5.3.5.1 Elevation. Shear walls have not been included as a means of elevating buildings or other structures in coastal high hazard areas. They can be very efficient in resisting loads in the plane of the shear wall from wind, etc There is concern about serious damage or collapse from wave forces normal to the plane of the shear wall However it is possible to design shear walls to resist such wave forces and special impact forces. When shear walls are used, they should be oriented to minimize the total surface area exposed to potential hydrodynamic and impact loads

C5.3.5.3 Erosion and Scour. Scour is an important consideration in the design of foundations in coastal high hazard areas or V Zone. In coastal areas, scour can be significant due to area erosion resulting from the effects of storm surge and wave action. Local scour around foundation elements such as a pile or a column, mat foundation or grade beam must also be considered in determining the required depth of embedment or anchorage. Along the coastline the erosion from many small to large storms must be considered through evaluation of historical records. The required embedment of piles and foundations must include consideration of both areal erosion and local scour at the foundation.

References

[1] U.S Army Corps of Engineers. Coastal Engineering Research Center Waterways Experiment Station, Shore Protection Manual. 2 Vols 4th Ed , *1984*

[2] U.S Army Corps of Engineers Office of the Chief of Engineers. Flood Proofing Regulations, EP 1165-2-314, March 1992.

[3] Federal Emergency Management Agency, National Flood Insurance Program. 44 CFR Ch. I Parts 59 and 60, (10-1-90 Edition).

[4] Terzaghi. K and Peck. R B.. Soil Mechanics in Engineering Practice, Wiley. 2nd Edition, 1967.

[5] Walton. T L Jr J P Ahrens. C.L. Truitt and R.G. Dean. *Criteria for Evaluating Coastal Flood Protection Structures*. Technical Report CERC 89-15. U S. Army Corps of Engineers Waterways Experiment Station, 1989

C6. Wind Loads

C6.1 General. The ASCE 7-98 version of the wind load standard provides three methods from which the designer can choose. A new "simplified method" (Method 1) for which the designer can select wind pressures directly without any calculation when the building meets all the requirements for application of the procedure; and two other methods (Analytical Method and Wind Tunnel Procedure) which are essentially the same methods as previously given in the standard except for some changes that are noted.

The ASCE 7-98 version of the standard has been entirely reformatted into a more "user friendly" format. Specific step-by-step design procedures are listed for application of Methods 1 and 2 to aid the user in applying the standard.

Temporary bracing should be provided to resist wind loading on structural components and structural assemblages during erection and construction phases.

C6.2 Definitions. New definitions have been added for "approved," "building envelope," "regular shaped building," "rigid building and other structure," "simple diaphragm building," "escarpment," "glazing," "impact resistant glazing," "impact resistant covering," "hill," "hurricane prone regions," "mean roof height," "openings," "ridge," and "wind borne debris region." These terms are used throughout the standard and are provided to clarify application of the standard provisions.

Main wind-force resisting system can consist of a structural frame or an assemblage of structural elements that work together to transfer wind loads acting on the entire structure to the ground. Structural elements such as cross-bracing, shear walls, roof trusses and roof diaphragms are part of the main wind-force resisting system when they assist in transferring overall loads [87].

Building, enclosed, open, partially enclosed: These definitions relate to the proper selection of internal pressure coefficients, GC_{pi}. Building, open and Building, partially enclosed are specifically defined. All other buildings are considered to be enclosed by definition, although there may be large openings in two or more walls. An example of this is a parking garage through which the wind can pass. The internal pressure coefficient for such a building would be ± 0.18, and the internal pressures would act on the solid areas of the walls and roof.

Components and cladding: Components receive wind loads directly or from cladding and transfer the load to the main wind-force resisting system. Cladding receives wind loads directly. Examples of components include fasteners, purlins, girts, studs, roof decking and roof trusses. Components can be part of the main wind-force resisting system when they act as shear walls or roof diaphragms, but they may also be loaded as individual components. The engineer needs to use appropriate loadings for design of components, which may require certain components to be designed for more than one type of loading, e.g., long span roof trusses should be designed for loads associated with main wind-force resisting systems, and individual members of trusses should also be designed for component and cladding loads [87]. Examples of cladding include wall coverings, curtain walls, roof coverings, exterior windows (fixed and operable) and doors, and overhead doors.

Effective wind area is the area of the building surface used to determine GC_p. This area does not necessarily correspond to the area of the building surface contributing to the force being considered. Two cases arise. In the usual case the effective wind area does correspond to the area tributary to the force component being considered. For example, for a cladding panel, the effective wind area may be equal to the total area of the panel; for a cladding fastener, the effective wind area is the area of cladding secured by a single fastener. A mullion may receive wind from several cladding panels; in this case, the effective wind area is the area associated with the wind load that is transferred to the mullion.

The second case arises where components such as roofing panels, wall studs or roof trusses are spaced closely together; the area served by the component may become long and narrow. To better approximate the actual load distribution in such cases, the width of the effective wind area used to evaluate GC_p need not be taken as less than one third the length of the area. This increase in effective wind area has the effect of reducing the average wind pressure acting on the component. Note however that this effective wind area should only be used in determining the GC_p in Figures 6.5 through 6.8. The induced wind load should be applied over the actual area tributary to the component being considered.

For membrane roof systems, the effective wind area is the area of an insulation board (or deck panel if insulation is not used) if the boards are fully adhered (or the membrane is adhered directly to the deck). If the insulation boards or membrane are mechanically attached or partially adhered, the effective wind area is the area of the board or membrane secured by a single fastener or individual spot or row of adhesive.

Flexible buildings and other structures: A building or other structure is considered flexible if it contains a significant dynamic resonant response. Resonant response depends on the gust structure contained in the approaching wind, on wind loading pressures generated by the wind flow about the building, and on the dynamic properties of the building or structure Gust energy in the wind is smaller at frequencies above about 1 Hz, therefore the resonant response of most buildings and structures with lowest natural frequency above 1 Hz will be sufficiently small that resonant response can often be ignored. When buildings or other structures have a height exceeding four times the least horizontal dimension or when there is reason to believe that the natural frequency is less than 1 Hz (natural period greater than 1 second), natural frequency for it should be investigated. A useful calculation procedure for natural frequency or period for various building types is contained in Section 9

Regular shaped buildings and other structures: Defining the limits of applicability of the analytical procedures within the standard is a difficult process, requiring a balance between the practical need to use the provisions past the range for which data has been obtained and restricting use of the provisions past the range of realistic application Wind load provisions are based primarily on wind-tunnel tests on shapes shown in Figures 6-3 through 6-8. Extensive wind-tunnel tests on actual structures under design show that relatively large changes from these shapes can, in many cases, have minor changes in wind load, while in other cases seemingly small changes can have relatively large effects, particularly on cladding pressures Wind loads on complicated shapes are frequently smaller than those on the simpler shapes of Figures 6-3 through 6-8, and so wind loads determined from these provisions reasonably envelope most structure shapes. Buildings which are clearly unusual should use the provisions of 6 4 for wind-tunnel tests.

Rigid buildings and other structures: The defining criteria for rigid, in comparison to flexible, is that the natural frequency is greater than or equal to 1 Hz. A general guidance is that most rigid buildings and structures have height to minimum width less than 4 Where there is concern about whether or not a building or structure meets this requirement, the provisions of section 9 provide a method for calculating natural frequency (period = 1/ natural frequency).

C6.3 Symbols and Notation. The following additional symbols and notation are used herein

- A_{oo} = average area of open ground surrounding each obstruction
- n = reference period, in years;
- P_a = annual probability of wind speed exceeding a given magnitude (see Eq C6-1);
- P_n = probability of exceeding design wind speed during n years (see Eq C6-1),
- s_{ob} = average frontal area presented to the wind by each obstruction
- V_t = wind speed averaged over t seconds (see Fig C6-1), in miles per hour (meters per second),
- V_{3600} = mean wind speed averaged over 1 hour (see Fig C6-1), in miles per hour (meters per second);
- β = structural damping coefficient (percentage of critical damping)

C6.4 Method 1 - Simplified Procedure. Method 1 has been added to the standard for a designer having the relatively common low-rise ($h \leq 30$ ft) regular shaped, simple diaphragm building case (see new definitions for "simple diaphragm building" and "regular shaped building") where pressures for the roof and walls can be selected directly from a table. Two tables are provided; Table 6-2 for the main wind force resisting system and Table 6-3A and 6-3 B for components and cladding For components and cladding, values are provided for enclosed and partially enclosed buildings. Note that for the main wind force resisting system in a diaphragm building, the internal pressure cancels for loads on the walls, but must be considered for the roof This is true because when wind forces are transferred by horizontal diaphragms (such as floors and roofs) to the vertical elements of the main wind force resisting system (such as shear walls, X-bracing, or moment frames), the collection of wind forces from windward and leeward sides of the building occurs in the horizontal diaphragms. Once transferred into the horizontal diaphragms by the wall systems, the wind forces become a net horizontal wind force that is delivered to the vertical elements The equal and opposite internal pressures on the walls cancel in the horizontal diaphragm. Method 1 combines the windward and leeward pressures into a net horizontal wind pressure, with the internal pressures canceled The user is cautioned to consider the precise application of windward and leeward wall loads to members of the roof diaphragm where openings may exist and where particular members such as drag struts are designed The design of the roof members of the main wind force resisting system is still influenced by internal pressures, but for the limitations imposed on the simple diaphragm building type, it can be assumed that the maximum uplift, produced by a positive internal pressure, is the controlling load case For the designer to use Method 1, the building must conform to all seven requirements in 6 4 1 otherwise Method 2 or 3 must be used. Values are tabulated for Exposure B; multiplying factors are provided for other common exposures. The use of the Simplified Procedure for low-rise buildings in Exposure A is not recommended because of the greater uncertainty of wind load distribution in such an environment. The following values have been used in preparation of the tables:

h = 30 ft   Exposure B $K_z = 0.70$
$K_d = 0.85$ G = 0.85 $K_{zt} = 1.0$ I = 1.0
$GC_{pi} = \pm 0.18$ (enclosed building)
$GC_{pi} = \pm 0.55$ (partially enclosed building)
Pressure coefficients from Figure 6-3 and Figure 6-5

C6.5 Method 2 - Analytical Procedure

C6.5.1 Scope. The analytical procedure provides wind pressures and forces for the design of main wind-force resisting systems and for the design of components and cladding of buildings and other structures. The procedure involves the determination of wind directionality and a velocity pressure, the selection or determination of an appropriate gust effect factor, and the selection of appropriate pressure or force coefficients. The procedure allows for the level of structural reliability required, the effects of differing wind exposures, the speed-up effects of certain topographic features such as hills and escarpments, and the size and geometry of the building or other structure under consideration. The procedure differentiates between rigid and flexible buildings and other structures, and the results generally envelope the most critical load conditions for the design of main wind-force resisting systems as well as components and cladding

C6.5.2 Limitations of Analytical Procedure. The provisions given under 6 5.2 apply to the majority of site locations and buildings and structures, but for some locations, these provisions may be inadequate. Examples of site locations and buildings and structures (or portions thereof) that require use of recognized literature for documentation pertaining to wind effects, or the use of the wind tunnel procedure of 6 6 include

1 Site locations which have channeling effects or wakes from upwind obstructions. Channeling effects can be caused by topographic features (e g , mountain gorge) or buildings (e.g., a cluster of tall buildings). Wakes can be caused by hills or by buildings or other structures.
2 Buildings with unusual or irregular geometric shape, including domes, barrel vaults, and other buildings whose shape (in plan or profile) differs significantly from a uniform or series of superimposed prisms similar to those indicated in Figures 6-3 through 6-8 Unusual or irregular geometric shapes include buildings with multiple setbacks, curved facades, irregular plan resulting from significant indentations or projections, openings through the building, or multi-tower buildings connected by bridges.
3 Buildings with unusual response characteristics, which result in across-wind and/or dynamic torsional loads, loads caused by vortex shedding, or loads resulting from instabilities such as flutter or galloping. Examples of buildings and structures which may have unusual response characteristics include flexible buildings with natural frequencies normally below 1 Hz, tall slender buildings (building height-to-width ratio exceeds 4), and cylindrical buildings or structures Note Vortex shedding occurs when wind blows across a slender prismatic or cylindrical body Vortices are alternately shed from one side of the body and then the other side, which results in a fluctuating force acting at right angles to the wind direction (across-wind) along the length of the body.
4 Bridges, cranes, electrical transmission lines, guyed masts, telecommunication towers and flagpoles

C6.5.2.1 Shielding. Due to the lack of reliable analytical procedures for predicting the effects of shielding provided by buildings and other structures or by topographic features, reductions in velocity pressure due to shielding are not permitted under the provisions of 6.5. However, this does not preclude the determination of shielding effects and the corresponding reductions in velocity pressure by means of the wind tunnel procedure in 6 6

C6.5.2.2 Air-permeable Cladding Air-permeable roof or wall claddings allow partial air pressure equalization between their exterior and interior surfaces Examples include siding, pressure-equalized rain screen walls, shingles, tiles, concrete roof pavers, and aggregate roof surfacing

The design wind pressures derived from 6 5 represent the pressure differential between the exterior and interior surfaces of the exterior envelope (wall or roof system). Because of partial air-pressure equalization provided by air permeable claddings the pressures derived from 6 5 can over estimate the load on air-permeable cladding elements The designer may elect either to use the loads derived from 6.5 or to use loads derived by an approved alternative method If the designer desires to determine the pressure differential across the air-permeable cladding element, appropriate full-scale pressure measurements should be made on the applicable cladding element or reference be made to recognized literature [9], [16], [37], [73] for documentation pertaining to wind loads

C6.5.4 Basic Wind Speed. The ASCE 7 wind map proposed for the 1998 standard has been updated from the map in ASCE 7-95 based on a new and more complete analysis of hurricane wind speeds [78, 79] This new hurricane analysis yields predictions of 50 and 100 year return period peak gust wind speeds along the coast which are generally similar to those given in [88] and [89] The decision within the Task Committee on Wind Loads to update the map relied on several factors important to an accurate wind specification

1 The new hurricane results include many more predictions for sites away from the coast than have been available in the past It is desirable to include the best available decrease in speeds with inland distance. Significant reductions in wind speeds occur in inland Florida for the new analysis.

2. The distance inland to which hurricanes can influence wind speed increases with return period. It is desirable to include this distance in the map for design of ultimate events (working stress multiplied by an appropriate load factor).
3. A hurricane coast importance factor of 1.05 acting on wind speed was included explicitly in past ASCE 7 standards (1993 and earlier) to account for the more rapid increase of hurricane speeds with return period in comparison to non-hurricane winds. The hurricane coast importance factor actually varies in magnitude and position along the coast and with distance inland. In order to produce a more uniform risk of failure, it is desirable to include the effect of the importance factor in the map by first mapping an ultimate event and then reducing the event to a design basis.

The Task Committee on Wind Loads chose to use a map which includes the hurricane importance factor in the map contours. The map is specified so that the loads calculated from the standard, after multiplication by the load factor, represent an ultimate load having approximately the same return period as loads for non-hurricane winds. (An alternative not selected was to use an ultimate wind speed map directly in the standard, with a load factor of 1.0.)

The approach required selection of an ultimate return period. A return period of about 500 years has been used previously for earthquake loads. This return period can be derived from the non-hurricane speeds in ASCE 7-95. A factor of 0.85 is included in the load factor of ASCE 7-95 to account for wind and pressure coefficient directionality [76]. Removing this from the load factor gives an effective load factor F of 1.30/0.85 = 1.529 (round to 1.5). Of the uncertainties affecting the wind load factor, the variability in wind speed has the strongest influence [77], such that changes in the coefficient of variation in all other factors by 25 percent gives less than a 5 percent change in load factor. The non-hurricane multiplier of 50-year wind speed for various return periods averages $F_c = 0.36 + 0.1 \ln(12T)$, with T in years [74]. Setting $F_c = \sqrt{F} = \sqrt{1.5} = 1.225$ yields T = 476 years. On this basis, a 500-year speed can reasonably represent an approximate ultimate limit state event.

A set of design level hurricane speed contours, which include the hurricane importance factor, were obtained by dividing 500-year hurricane wind speed contours by $\sqrt{F} = 1.225$. The implied importance factor ranges from near 1.0 up to about 1.25 (the explicit value in ASCE 7-93 is 1.05).

The design level speed map has several advantages. First, a design using the map results in an ultimate load (loads inducing the design strength after use of the load factor) which has a more uniform risk for buildings than occurred with earlier versions of the map. Second, there is no need for a designer to use and interpolate a hurricane coast importance factor. It is not likely that the 500-year event is the actual speed at which engineered structures are expected to fail, due to resistance factors in materials, due to conservative design procedures which do not always analyze all load capacity, and due to a lack of a precise definition of "failure".

The wind speed map of Fig. 6-1 presents basic wind speeds for the contiguous United States, Alaska and other selected locations. The wind speeds correspond to 3-second gust speeds at 33 ft (10 m) above ground for exposure category C. Because the National Weather Service has phased out the measurement of fastest-mile wind speeds, the basic wind speed has been redefined as the peak gust which is recorded and archived for most NWS stations. Given the response characteristics of the instrumentation used, the peak gust is associated with an averaging time of approximately 3 seconds. Because the wind speeds of Fig. 6-1 reflect conditions at airports and similar open-country exposures, they do not account for the effects of significant topographic features such as those described in 6.5.7. Note that the wind speeds shown in Fig. 6-1 are not representative of speeds at which ultimate limit states are expected to occur. Allowable stresses or load factors used in the design equation(s) lead to structural resistances and corresponding wind loads and speeds that are substantially higher than the speeds shown in Fig. 6-1.

The hurricane wind speeds given in Figure 6-1 replace those given in ASCE-7-95 which were based on a combination the data given in [5], [15], [54], [88], and [20], supplemented with some judgement. The non hurricane wind speeds of Fig. 6-1 was prepared from peak gust data collected at 485 weather stations where at least 5 years of data were available [29], [30], [74]. For non-hurricane regions, measured gust data were assembled from a number of stations in state-sized areas to decrease sampling error, and the assembled data were fit using a Fisher-Tippett Type I extreme value distribution. This procedure gives the same speed as does area-averaging the 50-year speeds from the set of stations. There was insufficient variation in 50-year speeds over the eastern 3/4 of the lower 48 states to justify contours. The division between the 90 and 85 mph (40.2 and 38.0 m/s) regions, which follows state lines, was sufficiently close to the 85 mph (38.0 m/s) contour that there was no statistical basis for placing the division off political boundaries. This data is expected to follow the gust factor curve of Figure C6-1 [13].

Limited data were available on the Washington and Oregon coast; in this region, existing fastest-mile wind speed data were converted to peak gust speeds using open-country gust factors [13]. This limited data indicates that a speed of 100 mph is appropriate in some portions of the special coastal region in Washington and 90 mph in the special coastal region in Oregon; these speeds do not include that portion of the special wind region in the Columbia River Gorge where higher speeds may be justified. Speeds in the Aleutian Islands and in the interior of Alaska were established from gust data. Contours in Alaska are modified slightly from ASCE 7-88 based on measured data, but insufficient data were available for a detailed coverage of the mountainous regions.

C6.5.4.1 Special Wind Regions. Although the wind-speed map of Fig. 6-1 is valid for most regions of the country, there are special regions in which wind-speed anomalies are known to exist. Some of these special regions are noted in Fig. 6-1. Winds blowing over mountain ranges or through gorges or river valleys in these special regions can develop speeds that are substantially higher than the values indicated on the map. When selecting basic wind speeds in these special regions, use of regional climatic data and consultation with a wind engineer or meteorologist is advised.

It is also possible that anomalies in wind speeds exist on a micrometeorological scale. For example, wind speed-up over hills and escarpments is addressed in 6.5.7. Wind speeds over complex terrain may be better determined by wind-tunnel studies as described in 6.6. Adjustments of wind speeds should be made at the micrometeorological scale on the basis of wind engineering or meteorological advice and used in accordance with the provisions of 6.5.4.2 when such adjustments are warranted.

C6.5.4.2 Estimation of Basic Wind Speeds from Regional Climatic Data. When using regional climatic data in accordance with the provisions of 6.5.4.2 and in lieu of the basic wind speeds given in Fig. 6-1, the user is cautioned that the gust factors, velocity pressure exposure coefficients, gust effect factors, pressure coefficients, and force coefficients of this Standard are intended for use with the 3-second gust speed at 33 ft (10 m) above ground in open country. It is necessary, therefore, that regional climatic data based on a different averaging time, for example hourly mean or fastest mile, be adjusted to reflect peak gust speeds at 33 ft (10 m) above ground in open country. The results of statistical studies of wind-speed records, reported by [13] for extratropical winds and for hurricanes [78], are given in Fig. C6-1 which defines the relation between wind speed averaged over t seconds, V_t, and over one hour, V_{3600}. New research cited in [78] indicates that the old Krayer Marshall curve [20] does not apply in hurricanes. Therefore it has been removed in Figure C6-1 in ASCE 7-98. The gust factor adjustment to reflect peak gust speeds is not always straightforward and advice from a wind engineer or meteorologist may be needed.

In using local data, it should be emphasized that sampling errors can lead to large uncertainties in specification of the 50-year wind speed. Sampling errors are the errors associated with the limited size of the climatological data samples (years of record of annual extremes). It is possible to have a 20 mph (8.9 m/s) error in wind speed at an individual station with a record length of 30 years. It was this type of error that led to the large variations in speed in the non-hurricane areas of the ASCE 7-88 wind map. While local records of limited extent often must be used to define wind speeds in special wind areas, care and conservatism should be exercised in their use.

If meteorological data are used to justify a wind speed lower than 85-mph 50-year peak gust at 10 m, an analysis of sampling error is required to demonstrate that the wind record could not occur by chance. This can be accomplished by showing that the difference between predicted speed and 85 mph contains 2 to 3 standard deviations of sampling error [67]. Other equivalent methods may be used.

C6.5.4.3 Limitation. In recent years, advances have been made in understanding the effects of tornadoes on buildings. This understanding has been gained through extensive documentation of building damage caused by tornadic storms and through analysis of collected data. It is recognized that tornadic wind speeds have a significantly lower probability of occurrence at a point than the probability for basic wind speeds. In addition, it is found that in approximately one-half of the recorded tornadoes, gust speeds are less than the gust speeds associated with basic wind speeds. In intense tornadoes, gust speeds near the ground are in the range of 150-200 mph (67-89 m/s). Sufficient information is available to implement tornado resistant design for above-ground shelters and for buildings that house essential facilities for post-disaster recovery. This information is in the form of tornado risk probabilities, tornadic wind speeds, and associated forces. Several references provide guidance in developing wind load criteria for tornado-resistant design [1], [2], [24] through [28], [57].

Tornadic wind speeds, which are gust speeds, associated with an annual probability of occurrence of 1×10^{-5} (100 000 year mean recurrence interval) are shown in Figure C6-1A. This map was developed by the American Nuclear Society committee ANS 2.3 in the early 1980s. Tornado occurrence data of the last 15 years can provide a more accurate tornado hazard wind speed for a specific site.

C6.5.4.4 Wind Directionality Factor. The existing wind load factor 1.3 in ASCE 7-95 includes a "wind directionality factor" of 0.85 [76, 77]. This factor accounts for two effects, (1) The reduced probability of maximum winds coming from any given direction (2) the reduced probability of the maximum pressure coefficient occurring for any given wind direction. The wind directionality factor (identified as K_d in the new standard) has been hidden in previous editions of the standard and has generated renewed interest in establishing the design values for wind forces determined by using the standard. Accordingly, the Task Committee on Wind Loads, working with the Task Committee on Load Combinations, has decided to separate the wind directionality factor from the load factor and include its effect in the equation for velocity pressure. This has been done by developing a new factor, K_d, that is tabulated in the new Table 6-6 for different structure types. As new research becomes available, this factor can be directly modified without changing the wind load factor. Values for the factor were established from references in the literature and collective committee judgment. It is noted that the k_d value for round chimneys, tanks and similar structures is given as 0.95 in recognition of the fact that the wind load resistance may not be exactly the same in all directions as implied by a value of 1.0. A value of 0.85 might be more appropriate if a triangular trussed frame is shrouded in a round cover. 1.0 might be more appropriate for a round chimney having a lateral load resistance equal in all directions. The designer is cautioned by the footnote to Table 6-6 and the statement in 6.5.4.4 where reference is made to the fact that this factor is only to be used in conjunction with the load combination factors specified in 2.3 and 2.4.

C6.5.5 Importance factor. The importance factor is used to adjust the level of structural reliability of a building or other structure to be consistent with the building classifications indicated in Table 1-1. The importance factors given in Table 6-1 adjust the velocity pressure to different annual probabilities of being exceeded. Importance-factor values of 0.87 and 1.15 are, for the non hurricane winds, associated, respectively, with annual probabilities of being exceeded of 0.04 and 0.01 (mean recurrence intervals of 25 and 100 years). In the case of hurricane winds, the annual exceedance probabilities implied by the use of the importance factors of 0.77 and 1.15 will vary along the coast, however, the resulting risk levels associated with the use of these importance factors when applied to hurricane winds will be approximately consistent with those applied to the non-hurricane winds.

The probability P_n that the wind speed associated with a certain annual probability P_a will be equaled or exceeded at least once during an exposure period of n years is given by

$$P_n = 1 - (1 - P_a)^n \qquad \text{(C6-1)}$$

and values of P_n for various values of P_a and n are listed in Table C6-2. As an example, if a design wind speed is based upon $P_a = 0.02$ (50 year mean recurrence interval), there exists a probability of 0.40 that this speed will be equaled or exceeded during a 25-year period, and a 0.64 probability of being equaled or exceeded in a 50-year period.

For applications of serviceability, design using maximum likely events, or other applications, it may be desired to use wind speeds associated with mean recurrence intervals other than 50 years. To accomplish this, the 50-year speeds of Fig. 6-1 are multiplied by the factors listed in Table C6-3. Table C6-3 is strictly valid for the non-hurricane winds only (V<100 mph for continental U.S. and all speeds in Alaska), where the design wind speeds have a nominal annual exceedance probability of 0.02. Using the factors given in Table C6-3 to adjust the hurricane wind speeds will yield wind speeds and resulting wind loads that are approximately risk consistent with those derived for the non-hurricane prone regions. The true return periods associated with the hurricane wind speeds cannot be determined using the information given in this standard.

The difference in wind speed ratios between continental U.S. (V < 100 mph) and Alaska were determined by data analysis and probably represent a difference in climatology at different latitudes.

C6.5.6 Exposure Categories. Revisions have been made to definitions for Exposure C and Exposure D in recognition of new research [86]. The definitions of Exposures A and B remain unchanged. Proper application of exposure requires the designer to consider the following:

2. Terrain roughness for the particular surface area surrounding the site including height and density of topographic features and other structures
2. For each wind direction assumed, the vertical frontal area of each obstruction to the wind for any selected upwind fetch surface area.

The Task Committee on Wind Loads has made a judgment that 1500 feet or 10 times the structure height, whichever is greater, is the appropriate fetch distance to consider for Exposure B. One half mile or 10 times the structure height, whichever is greater, remains the fetch distance to consider for Exposure A.

In an effort to assist the designer in the proper selection of the Exposure B category, the following guidance is provided. Terrain roughness corresponding to Exposure B may be defined by consideration of the frontal area of each obstruction to the wind in the upwind fetch surface area [75]. For Exposure B to apply, on average the vertical frontal area of each obstruction to the wind in the upwind fetch surface area should be at least 5% of the area of open ground surrounding it. Mathematically this may be expressed as follows:

$$\frac{s_{ob}}{A_{ob}} \geq 0.05 \qquad \text{(C6-2)}$$

where

s_{ob} = average frontal area presented to the wind by each obstruction, and

A_{ob} = average area of open ground surrounding each obstruction

Vertical frontal area is defined as the area of the projection of the obstruction onto a vertical plane normal to the wind direction. The area s_{ob} may be estimated by summing the vertical frontal areas of all obstructions within a selected area of upwind fetch and dividing the sum by the number of obstructions in the area. Likewise A_{ob} may be estimated by dividing the size of the selected area of upwind fetch by the number of obstructions in it

Trees and bushes are porous and are deformed by strong winds, which reduces their effective frontal area. For conifers and other evergreens no more than 50% of their gross frontal area can be taken to be effective in obstructing the wind For deciduous trees and bushes, no more than 15% of their gross frontal area can be taken to be effective in obstructing the wind Gross frontal area is defined in this context as the projection onto a vertical plane (normal to the wind) of the area enclosed by the perimeter envelope of the tree or bush

The "upwind fetch surface area" for evaluation is currently left to engineering judgment For component and cladding design, where a single exposure is to be selected representing the most severe condition (highest wind loads), it is suggested that 30 degree sectors be considered in assigning the most severe terrain exposure for the site

A recent study [66] has estimated that the majority of buildings (perhaps as much as 60-80%) have a terrain exposure corresponding to Exposure B

A new requirement in ASCE 7-98 states that if a site is located in a transition zone between categories, the category resulting in the largest wind forces shall apply.

Exposure C has been expanded to include the shoreline in hurricane prone regions based on a recent study of the subject [86] It is recommended that Exposure C be used for the shoreline in non hurricane prone regions where open water extends at least 600 feet but less than one mile upwind from the building or other structure. If less than 600 feet of open water extends upwind then the terrain further upwind becomes a factor in exposure selection.

Aerial photographs, representative of each exposure type, have been included in the commentary to aid the user in establishing the proper exposure for a given site Obviously, the proper assessment of exposure is a matter of good engineering judgment.

A significant improvement in ASCE 7-98 is that wind forces for both the main wind force resisting system and components and cladding are now based on the actual exposure that the structure is judged to be in rather than the concept of using only Exposure C or using a modifying exposure factor for Exposure B. (See commentary discussion under C6 5.11) In the case of component and cladding elements and for low-rise buildings designed using Figure 6-4, wind profiles for Exposures A and B have been truncated in the bottom 100 feet and 30 feet, respectively. The truncation accounts for increased wind loading coefficients caused by local turbulence and from increased wind speeds near the surface associated with openings in the surface roughness such as parking lots, wide roads, road intersections, underdeveloped lots and tree clearings. Where such clearings adjacent to the building exceed 600 ft (183 m) the use of Exposure C is recommended. This effect has been accounted for by adjusting the velocity pressure exposure coefficient values in Table 6-5. The use of a truncated profile eliminates the underestimation in the wind loads which would occur in the absence of the truncation now that the actual exposures are used in the design.

In the case of component and cladding loads on high rise buildings, the negative pressures will not be affected by the truncation since these loads are referenced to the dynamic pressure at roof height. In the case of the positive pressures near the base of a high rise building (h>60ft) in an exposure B environment, the component and cladding loads will increase in the lowest 30 feet. This increase is observed in wind tunnel tests, and the estimate of the positive cladding pressures near the base of taller buildings is, and has in the past, been underestimated in most cases using a non-truncated profile.

In the case of overall loads on buildings and structures, the net effect of the changes in the characteristics of the turbulence is less important as the loads are integrated over the entire structure. These overall loads are reasonably well estimated when normalized by the peak gust wind speed obtained using the non-truncated profile

In general, when using Figure 6-3, it is reasonable to base main wind force resisting system loads on specific wind directions since the frame loads typically can be correlated with a particular wind direction and act in a direction very close to the wind direction However, it should also be recognized that the pressure coefficients in Figure 6-3 are based on two perpendicular wind directions along the building axis Pressures on components and cladding required for design have, from experience on many wind tunnel tests, not been consistently identified with a particular wind direction. In defining the exposure category for components and cladding in 6.5.6.3.1 and 6.5.6.3.2, the phrase "...based on the exposure resulting in the highest wind loads for any wind direction at the site" has been added to alert the user to the fact that there is no way to know, except for wind tunnel testing, which pressure zones are associated with which wind directions. Wind tunnel tests have shown high pressure zones to occur at unexpected wind directions. Therefore, a single exposure representing the most severe condition (highest