Low-Rise Building Wind Load Provisions—Where Are We and Where Do We Need To Go?*

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ABSTRACT

The National Standard ASCE 7 was revised and published in January 1996 following many years of study by the Wind Loads Subcommittee of ASCE 7. Contained for the first time are a complete set of provisions for assessing wind loads for the design of low-rise buildings. The provisions are based, for the most part, on the research conducted at the University of Western Ontario during the late 1970s (funded, in part by MBMA), the following extensive research reported from Concordia University, and the very recent activity at the University of Western Ontario supported by the timber industry (as yet not subject to proper peer review). The new low-rise provisions reflect rather substantial reductions in loading requirements for some design applications.

The object of this communication is to briefly review the changes incorporated in ASCE 7-95 for Buildings of All Heights, Other Structures and Low-rise Buildings and compare the new wind load provisions with those contained in other codes and standards currently in use. The troublesome items that may impede acceptance of the new Standard and should be addressed in the future are cited.

INTRODUCTION

A revision of the National Standard, "Minimum Design Loads for Buildings and Other Structures," ASCE 7-95, was published in January 1996. The past standard ASCE 7-93 (revision of ANSI A58.1-1982) had been under review since 1993. Participation by the various industrial groups including the National Association of Home Builders was intense and unquestionably contributed in a positive manner to the provisions contained in the final document. At issue were the following changes:

- a new Wind Speed Map based on 3-second gust speeds and reflecting additional measured wind speed data obtained since 1979,
- new provisions for wind speed-up due to topographical effects (wind flowing over escarpments, hills),
- substantial increases in the internal pressure coefficients for typical low-buildings sited in hurricane zones,
- decreases in design wind pressures for low-buildings embedded in suburban terrain, and
- two distinct, separate approaches for assessing the wind loads for buildings having heights less or equal to 60 ft (18 m).

Additionally, the movement toward developing deemed-to-comply documents for residential construction and changes in the South Florida Building Code (1993) mandating engineering design of residential construction, has heightened interest and activity in the code arena.

The new standard ASCE 7-95 reflects some substantial changes in assessing wind loads for low-rise buildings. For some applications and specific areas of the country, the design loads will increase while for other cases there will be a considerable reduction in the wind load requirements, particularly if the new low-rise provisions are used. The flurry of activity among the code groups who promulgate the specific codes adopted by the 40,000 or so jurisdictions in the United States (Perry, 1992c) will continue to intensify as the three model codes (ICBO, BOCA and SBCCI) attempt to agree on a common national code by the year 2000. Before the end of 1996 it is quite possible that 12 different documents will be available for adoption for the design of low-rise, commercial structures and/or one and two family dwellings:

- ASCE 7-95, revision of ANSI/ASCE 7-93
- ANSI/ASCE 7-93, revision of ANSI/ASCE 7-88, (essentially the same provisions as ANSI, A58.1-1982)
- BOCA-1996, containing ASCE 7-95 as an alternate procedure
- UBC-1997, which may (or may not) contain revisions of the wind load provisions consistent with ASCE 7-95
- SBCCI-1997, with no changes over the alternate procedure of the 1994 edition

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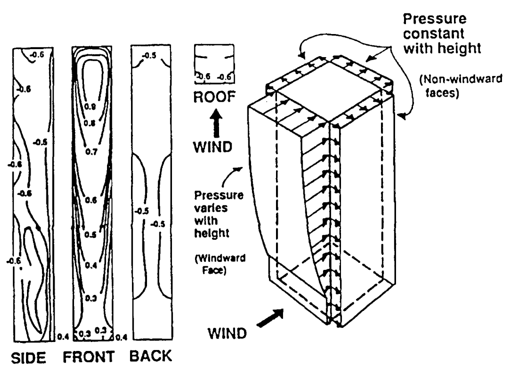
$$\begin{bmatrix}\text{estimated}\\ \text{wind}\\ \text{load}\end{bmatrix} = \begin{bmatrix}\text{reference}\\ \text{velocity}\\ \text{pressure}\end{bmatrix} \bullet \begin{bmatrix}\text{importance}\\ \text{factor}\end{bmatrix} \bullet \begin{bmatrix}\text{exposure}\\ \text{height}\\ \text{factor}\end{bmatrix} \bullet \begin{bmatrix}\text{aerodynamic}\\ \text{shape}\\ \text{factor}\end{bmatrix} \bullet \begin{bmatrix}\text{gust}\\ \text{effect}\\ \text{factor}\end{bmatrix} \qquad \text{Equation (1)}$$

$$p_z = 1/2\rho(V_{33})^2 \bullet I \bullet K_z \bullet (C_p - C_{pi}) \bullet G$$

- South Florida Building Code (SFBC-1993)
- MBMA-1986 Low-Rise Systems Manual
- ICBO ER 3018, Design Wind Load Criteria for Metal Building Systems (1996)
- TM 4-809-1/AFM 8803, "Load Assumptions for Buildings," developed by the Army and Air Force (1986)
- SBCCI SSTD 10-93, "Deemed-to-Comply Standard for Single and Multi-Family Dwellings in High Wind Regions"
- TDI-1996, "Building Code for Windstorm Resistant Construction"
- Blue Sky 1996, Deemed-to-Comply provisions based on performance criteria of SBCCI-1994 (alternate procedure)

A new non-consensus ASCE Report, "Wind Load Provisions for Structures Not Exceeding 160 Feet in Height" will also be published. This document was produced by a special ASCE Wind Load Task Committee whose charge was to develop a simplified version of ASCE 7-95 that was more "user friendly" and could be readily adopted by the model codes.

The primary objective of this paper is to describe the changes in ASCE 7-95 that will affect the design of low-rise buildings focusing on the two different approaches set forth in the Standard. Subsequently, these provisions are compared with those contained in other documents alluded to above which are currently in use.



(a) External pressure coefficients measured in wind tunnel. (Baines, 1965)

(b) Assumed distribution of external wind pressures. (Perry, 1987)

Fig. 1. Monitored mean pressure coefficients and assumed building code loads.

GENERAL CODE FORMAT

Traditionally, code and standard writers in the United States have used a simple relationship for the pressures induced on the surfaces of a building as a function of the free stream wind flow as given in Equation 1.

where

- p_z = pressure induced on a particular surface at height z above mean ground level, psf
- ρ = mass density of air
- I = importance factor reflecting risk based on nature of occupancy
- V_{33} = basic design wing speed in airport exposure at reference height 33 ft (10 m) above mean ground level, mph
- K_z = exposure coefficient to account for variation in velocity with height z above mean ground level as influenced by terrain exposure
- G = gust effect factor intended to account for load amplification due to turbulence in approaching wind and turbulence generated as result of interruption of flow pattern by a building or structure in wind path (in latter standards this term was expanded to include effect of resonant vibrations of structure)
- C_p = external mean pressure coefficient (or shape factor) averaged over some time interval
- C_{pi} = internal pressure coefficient

The degree of sophistication involved in evaluating the parameters contained in the above equation for a given design application, constitutes the point of departure in codifying wind effects. As our basic understanding of the effects of wind on structures has improved, the tendency has been to introduce refinements (and attendant complexities) in the evaluation of each of the terms. Obviously, some tradeoffs are necessary in order that reasonably simple wind load criteria can be developed which are applicable for a majority of conventional type structures encountered in practice.

Induced wind pressures are transient and fluctuate markedly with respect to both time and space. Thus, in order to codify the data generated from wind-tunnel experiments, it is necessary to time average and spatially average the pressure coefficients. The more recent codes and standards have accomplished this task by separating the overall (or global) forces (Figure 1) to be used in the design of the primary or main wind-force resisting systems (MWFRS) from those appropriate for the design of the fasteners, cladding, and

$$\begin{bmatrix}\text{estimated}\\ \text{wind}\\ \text{load}\end{bmatrix} = \begin{bmatrix}\text{reference}\\ \text{velocity}\\ \text{pressure}\end{bmatrix} \bullet \begin{bmatrix}\text{exposure}\\ \text{height}\\ \text{factor}\end{bmatrix} \bullet \begin{bmatrix}\text{gust}\\ \text{response}\\ \text{factor}\end{bmatrix} \bullet \begin{bmatrix}\text{aerodynamic}\\ \text{shape}\\ \text{factor}\end{bmatrix} \qquad \text{Equation (2)}$$

$$p_z = 1/2\rho(IV_{33})^2 \bullet K_z \bullet G_z \bullet (C_p - C_{pi})$$

Windward Face:

$$\begin{bmatrix}\text{estimated}\\ \text{wind}\\ \text{load}\end{bmatrix} = \begin{bmatrix}\text{velocity pressure}\\ \text{varies with}\\ \text{height above}\\ \text{ground}\end{bmatrix} \bullet \begin{bmatrix}\text{gust}\\ \text{response}\\ \text{factor}\end{bmatrix} \bullet \begin{bmatrix}\text{aerodynamic}\\ \text{shape}\\ \text{factor}\end{bmatrix} \qquad \text{Equation (3)}$$

$$p_z = q_z \bullet G_h \bullet (C_p - C_{pi})$$

Non-Windward Face:

$$\begin{bmatrix}\text{estimated}\\ \text{wind}\\ \text{load}\end{bmatrix} = \begin{bmatrix}\text{evaluated}\\ \text{at mean}\\ \text{roof height}\end{bmatrix} \bullet \begin{bmatrix}\text{evaluated}\\ \text{at mean}\\ \text{roof height}\end{bmatrix} \bullet \begin{bmatrix}\text{aerodynamic}\\ \text{shape}\\ \text{factor}\end{bmatrix} \qquad \text{Equation (4)}$$

$$p_h = q_h \bullet G_h \bullet (C_p - C_{pi})$$

$$\begin{bmatrix}\text{estimated}\\ \text{wind}\\ \text{load}\end{bmatrix} = \begin{bmatrix}\text{reference}\\ \text{velocity}\\ \text{pressure}\end{bmatrix} \bullet \begin{bmatrix}\text{exposure}\\ \text{height}\\ \text{factor}\end{bmatrix} \bullet \begin{bmatrix}\text{dynamic}\\ \text{gust}\\ \text{response}\\ \text{factor}\end{bmatrix} \bullet \begin{bmatrix}\text{aerodynamic}\\ \text{shape}\\ \text{factor}\end{bmatrix} \qquad \text{Equation (5)}$$

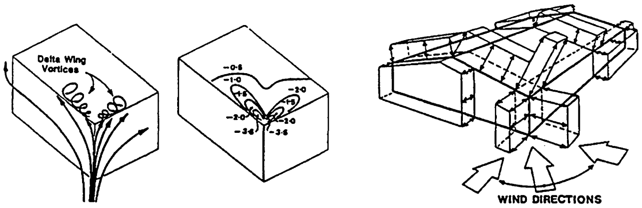
$$p_z = 1/2\rho(IV_{33})^2 \bullet K_z \bullet G_z \bullet (C_p - C_{pi})$$

$$\begin{bmatrix}\text{estimated}\\ \text{wind}\\ \text{load}\end{bmatrix} = \begin{bmatrix}\text{velocity pressure}\\ \text{evaluated at}\\ \text{mean roof height}\end{bmatrix} \bullet \begin{bmatrix}\text{peak}\\ \text{pressure}\\ \text{coefficient}\end{bmatrix} \qquad \text{Equation (6)}$$

$$p_z = q_h \bullet (GC_p) - (GC_{pi})$$

components of the structure (C&C) which must resist the much higher loadings induced over very small areas (Figure 2). In ANSI/ASCE 7-93 (revision of A58. 1-1982), the basic format was written for MWFRS as seen in Equations (2), (3), and (4), and for components and cladding as seen in Equations (5) and (6).

Thus, the importance factor I and exposure height factor



(a) Localized pressures due to oblique wind. (Newberry, Eaton 1974).

(b) Code prescriptions for localized loads (Perry, 1987)

Fig. 2. Monitored peak pressure coefficients and assumed building code loads.

K_z had been combined with $1/2\rho V_{33}^2$ to yield a velocity pressure q_z given by

$$q_z = 1/2\rho(IV)^2 \bullet K_z \tag{7}$$

The terms (GC_p) and (GC_{pi}) represent the peak pressure coefficients measured in the wind tunnel experiments and hence can be interpreted as the product of the gust response factor G and mean pressure coefficient C_p.

In the 1995 revision, an attempt was made to make the basic code format more consistent with the model codes. The importance factor I was not included in the velocity term. Additionally, a new term K_{zt} was added to reflect the wind speed-up due to topographical effects (Figure 3).

REVISIONS IN ASCE 7-95

Wind Speed Map

The basic wind speed map shown in Figure 4 reflects the following changes from that found in ANSI/ASCE 7-93:

- The speeds shown represent 3-second gust speeds ($V_{3sec} \cong 1.2V_{fm}$)
- for non-hurricane regions it was possible to specify de-

$$\text{ASCE 7-93}\quad \begin{bmatrix}\text{estimated}\\ \text{wind}\\ \text{load}\end{bmatrix} = \begin{bmatrix}\text{reference}\\ \text{velocity}\\ \text{pressure}\end{bmatrix} \bullet \begin{bmatrix}\text{importance}\\ \text{factor}\end{bmatrix} \bullet \begin{bmatrix}\text{exposure}\\ \text{height}\\ \text{factor}\end{bmatrix} \bullet \begin{bmatrix}\text{aerodynamic}\\ \text{shape}\\ \text{factor}\end{bmatrix} \bullet \begin{bmatrix}\text{gust}\\ \text{effect}\\ \text{factor}\end{bmatrix} \quad \text{Equation (8)}$$

$$p_z \quad = \quad 1/2\rho(IV_{33})^2 \quad \bullet \quad \quad \bullet \quad K_z \quad \bullet \quad (C_p - C_{pi}) \quad \bullet \quad G$$

$$\text{ASCE 7-95}\quad \begin{bmatrix}\text{estimated}\\ \text{wind}\\ \text{load}\end{bmatrix} = \begin{bmatrix}\text{reference}\\ \text{velocity}\\ \text{pressure}\end{bmatrix} \bullet \begin{bmatrix}\text{importance}\\ \text{factor}\end{bmatrix} \bullet \begin{bmatrix}\text{exposure}\\ \text{height}\\ \text{factor}\end{bmatrix} \bullet \begin{bmatrix}\text{aerodynamic}\\ \text{shape}\\ \text{factor}\end{bmatrix} \bullet \begin{bmatrix}\text{gust}\\ \text{effect}\\ \text{factor}\end{bmatrix} \quad \text{Equation (9)}$$

$$p_z \quad = \quad 1/2\rho(V_{33})^2 \quad \bullet \quad I^2 \quad \bullet \quad K_{zt}K_z \quad \bullet \quad (C_p - C_{pi}) \quad \bullet \quad G$$

sign wind speeds for the contiguous states in terms of only two zones: 85 mph (38 m/s) and 90 mph (40 m/s), thus eliminating contour lines (Figure 4)

- wind speed data available from 485 stations (129 stations for old map) were used; at some locations, the wind speeds have increased while for others the speeds decreased
- the difference between the importance factors along the hurricane-prone coastline and those appropriate for inland stations has been eliminated by incorporating the difference in the probability distribution of hurricane winds (Table 1, Figure 5) into the wind speed contours along the Gulf and Atlantic Coastlines, and
- a conversion table (Table 2) is provided in the commentary for determining wind speeds for other return periods (probability of exceedance)

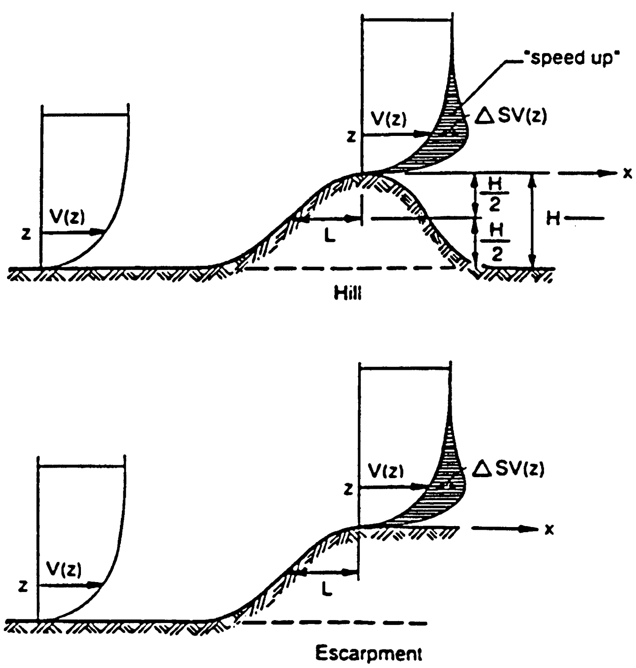


Fig. 3. *Wind speed effects (after ASCE, 1996).*

The U.S. National Weather Service no longer collects fastest-mile data and hence it was logical to re-define the basic wind speed in terms of either 3-second gust (used by United Kingdom, Australia); 1 minute sustained (National Hurricane Center); 10 minute sustained (European Code & Caribbean Uniform Building Code); or hourly-mean (Canadian) speeds. All options were considered. In the end, 3-second gust speed was chosen for the following reasons:

- A larger number of U.S. Stations record 3-second gust speeds making more data available for refining the map
- for anemometers that provide a voltage signal, the maximum peak gust will be recorded at a given location during a hurricane event even if the power source to drive the recording drum is lost
- it provides a consistent measure of wind speed to be used and understood by design professionals, building code

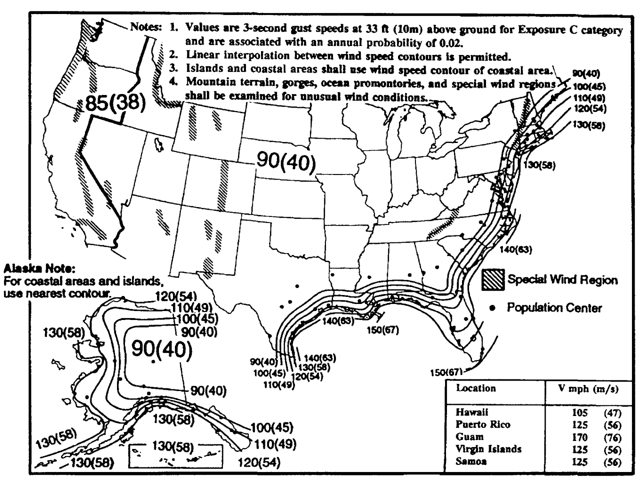


Fig. 4. Basic wind speed in mph (m/s) (after ASCE, 1996).

Table 1.
Importance Factors I

Classification	ASCE 7-93				ASCE 7-95	
	Category	Return Period (Years)	Importance Factor, I		Category	Importance Factor I
			100 miles Inland	Hurricane Oceanline		
All buildings and structures except those listed.	I	50	1.00	1.05	II	1.00
Buildings and structures where primary occupancy is more than 300 people in one area.	II	100	1.07	1.11	III	1.15
Buildings and structures designed as essential facilities including, but not limited to –Hospitals; Fire and police stations; –Disaster operation centers; National defense structures	III	100	1.07	1.11	IV	1.15
Buildings and structures that represent a low hazard to human life or to property	IV	25	0.95	1.00	I	0.87

officials, meteorologists, the news media, and John Doe public

In preparing the previous map (1979) for the interior of the contiguous United States, the data from only 129 stations were utilized in the analysis to ensure reasonable homogeneity of data (Simiu, et al 1979). Only those stations were used for which a minimum of ten years of continuous records were available, the terrain surrounding the recording station was representative of Exposure Category C (airport), the wind speeds recorded were fastest-mile, the anemometers were known to be located in open, unobstructed areas, and the history of anemometer height was known.

Peterka and Shahid (1992) in updating the wind speed map, utilized data from 485 stations where peak gust data were recorded and having at least 5 years of continuous data. For the non-hurricane regions of the contiguous United States, the data were fit into state-sized blocks to reduce sampling error. As in the earlier study (Batts, et al 1980), the assembled data were statistically reduced using extreme analyses procedures based on Fisher-Tippett Type I (Gumbel) distribution. The variation in 50-yr speeds over 75 percent of the eastern 48 states was too small to justify isotachs. The division between 90 mph (40.2 m/s) and 85 mph (38.0 m/s) regions was close enough to draw the line following state boundaries.

Because of the lack of data for hurricanes, the wind speed contours in the hurricane-prone region of Figure 4 continue to be based on a Monte Carlo simulation of hurricane storms striking the coastal region (Batts, et al 1980). The coastline was divided into discrete points spaced at 50 nautical miles. Thus, the total coastline of 2,900 nautical miles had 58 points. The results of the analysis provided wind speeds at each point for various probabilities of being exceeded. The wind speed values correspond to smooth terrain (Exposure C category) at a 10 m (33 ft) height above ground. Subsequently, the prediction of hurricane wind speeds was addressed by Georgiou (1983), Vickery and Twisdale (1993, 1994) and Peterka and

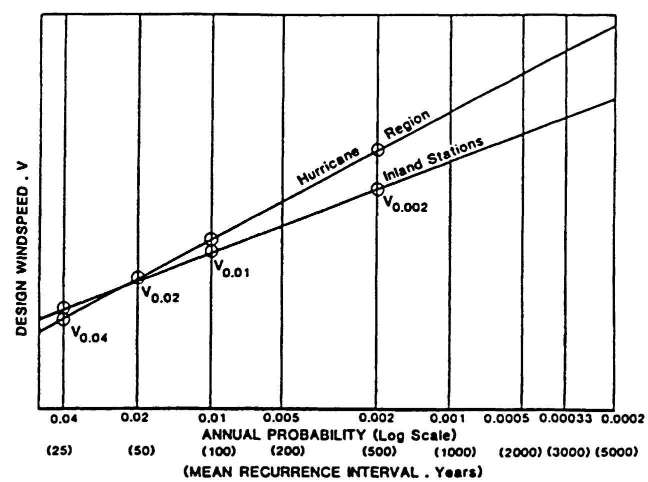


Fig. 5. Probability distributions of wind speeds (after Mehta, et al 1992).

Shahid (1992, 1993, 1994). The simulation estimates by Georgiou (1983) showed substantial agreement with the wind speeds generated by Batts, et al (1980) except along the Florida panhandle extending from Tampa to Jacksonville and for large return periods (500–2000 yrs). The research reported by Vickery and Twisdale (1992) appeared to indicate that the ASCE 7-93 map underestimated the speeds for the Texas coastline. This research was a very limited study, however, and considered only two locations in the Gulf area (Galveston, TX and Mobile, AL).

In developing the contours for the hurricane-prone coastline, Peterka (1994) corrected the fastest-mile speeds to 3-second gust speeds using the appropriate gust factors (Durst 1960; Krayer and Marshall 1989). Of particular significance to design professionals and building code officials is the fact that the need to consider separate importance factors for the coastline and inland regions was removed by incorporating the difference in the probability distribution of hurricane winds into the hurricane contours. The contours shown over the Atlantic Ocean are for interpolation and represent values for Exposure C over land. Additionally, Peterka (1994) prepared Table 2 (presented in the Commentary) which provides the appropriate conversion factors for determining design speeds for other than the 50-yr return period (probability of exceedance of 0.02) given in the map. This has been a problem in the past for design professionals and building code officials in assessing proper design speeds for construction loads, serviceability, design of temporary facilities, or other applications requiring different risk levels.

Table 2.
Conversion Factors for Other Mean Recurrance Intervals (ASCE, 1996)

MRI (yrs)	Continental U.S. $V = 85 - 100$ mph	Continental U.S. $V > 100$ mph (Hurricane)	Alaska
500	1.23	1.33	1.18
200	1.14	1.21	1.12
100	1.07	1.105	1.06
50	1.00	1.00	1.00
25	0.93	0.89 (84 mph min.)	0.94
10	0.84	0.73 (76 mph min.)	0.87
5	0.78	0.52 (70 mph min.)	0.81
1	0.61	0.48 (55 mph min.)	0.67

As noted above, the basic design wind speeds were increased for certain locations and decreased for others. For example, the 1993 version of the Standard specifies 50-yr wind speeds of 110 and 100 mph for South Florida and New Orleans, respectively. For structures sited adjacent to the coastline, these values would correspond to

$$(1.05)(110)(1.2) = 138.6 \text{ mph}$$

and

$$(1.05)(100)(1.2) = 126 \text{ mph}$$

3-second gust speeds when compared with ASCE 7-95 which lists 145 mph (4 percent increase) for South Florida and 140 mph (11 percent increase) for New Orleans. On the other hand, for the inland section of the contiguous United States wherein wind speeds of 84–108 mph (adjusted to 3-sec gust speeds) and are specified in ASCE 7-93, the new Standard

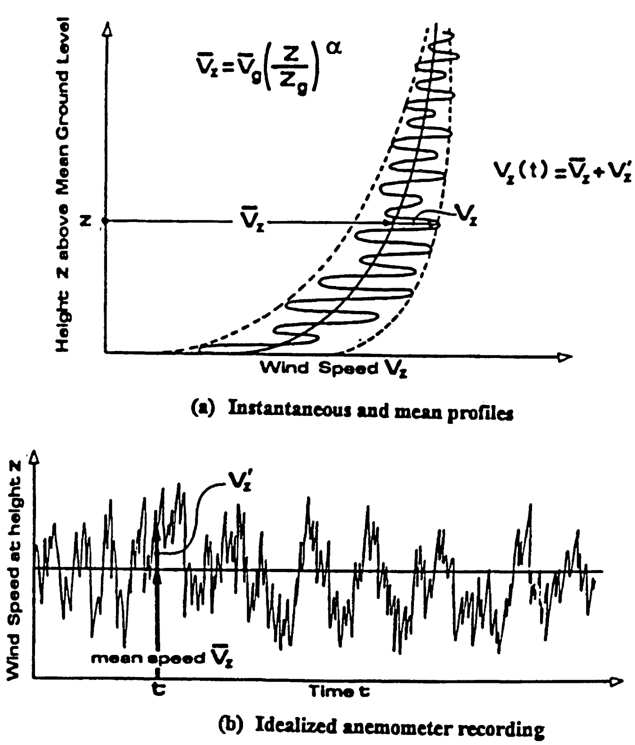


Fig. 6. Wind characteristics.

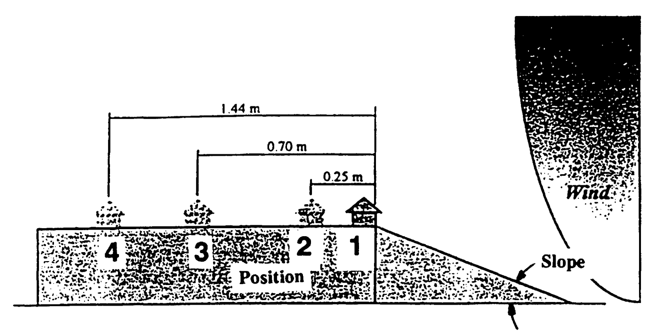


Fig. 7. Positions of house along test section (after Means, et al 1996).

Table 3. Power Law Exponent and Surface Roughness Length			
Exposure Category	α ASCE 7-93	α ASCE 7-95	z_g (ft)
A	3.0	5.0	1500
B	4.5	7.0	1200
C	7.0	9.0	900
D	10.0	11.5	700

specifies 90 mph for the entire region eastward of California, Oregon, and Washington, and 85 mph for these states.

Velocity Pressure Exposure Coefficients

Changing the basic design wind speed to a 3-second gust speed required corresponding changes in the wind profile as depicted in Figure 6. The exposure categories (A, B, C and D) and gradient heights z_g are consistent with those values given in ASCE 7-93 but the power law coefficient α values are changed (Table 3) to make the wind speed profiles flatter. The velocity pressure exposure coefficients K_z are given by

$$K_z = \begin{cases} 2.01\left(\dfrac{z}{z_g}\right)^{2/\alpha} & \text{for } 15\text{ ft} \le z \le z_g \\ 2.01\left(\dfrac{15}{z_g}\right)^{2/\alpha} & \text{for } z \le 15\text{ ft} \end{cases}$$

The new values of K_z agree favorably with those given in the Australian Standard (1989) which uses 3-second gust speeds.

Topographic Factor

For the first time in U.S. codes and standards, a topographic factor K_{zt} is introduced to account for wind speed-up as the wind flows over escarpments or hills as depicted in Figure 3. As the load varies as a function of the square of the wind speed, the increase in the overall load may be substantial for low-buildings sited near the crest and having a height that places the entire structure within the speed-up bubble.

A recent wind tunnel investigation (Means, et al 1996) determined the induced wind pressures on a typical one-story house (Figure 7). Figure 8 provides a comparison of pressure coefficients C_p for a house sited in position 1 (Figure 7) and an escarpment slope of 30° together with the flat terrain values. Figure 9 presents pressure coefficients normalized by the flat terrain values for various escarpment slopes. The study indicates that the provisions set forth in ASCE 7-95 are on the unconservative side for two reasons:

1. The pressure coefficients for the roof are underestimated by factors as high as 2.6 (Figure 8) for a house sited at the crest and as high as 6.2 for a house situated in position 1
2. the location of high suctions extend much farther from the leading edge of the roof than indicated by the Standard (Figure 9); the width of the perimeter zones (See Figure 2b) should be a function of escarpment slope and distance from the crest

Gust Effect Factor

With the change from a basic fastest-mile wind speed to a 3-second gust, a factor to account for the unsteady effects in the along-wind direction due to wind turbulence—structure interaction was needed. The new factor termed "Gust Effect Factor" in ASCE 7-95 is presented for three categories:

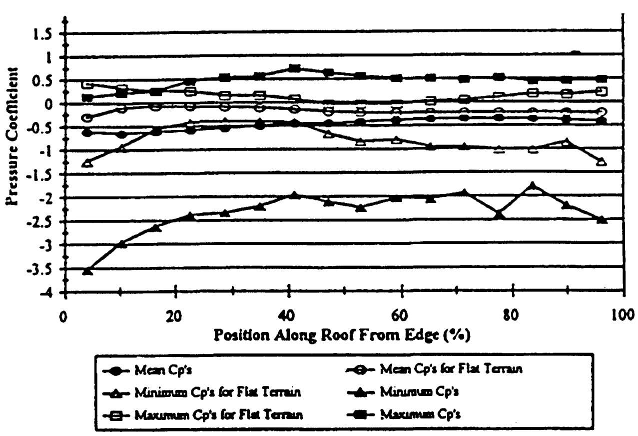


Fig. 8. Pressure coefficients with distance from roof edge (after Means, et al 1996).

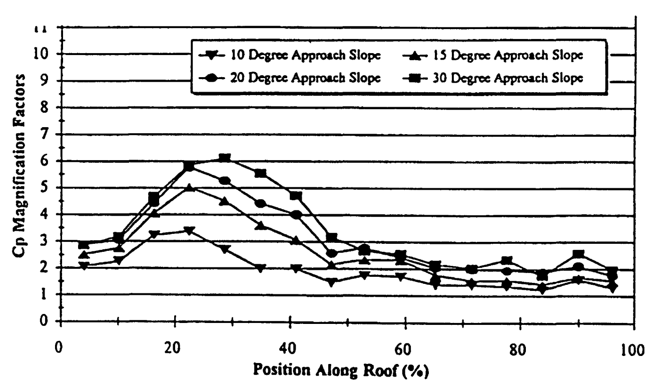


Fig. 9. C_p magnification factors (after Means, et al 1996).

- Category I: Rigid structure—simplified method
- Category II: Rigid structures—complete analysis
- Category III: Flexible or dynamically sensitive structures

For simplicity, the Gust Effect Factor G given in Section 6.6.1 of the Standard may be used for rigid structures. The values of G are 0.8 for exposures A and B and 0.85 for exposures C and D. Note that these factors are considered constant with height and that the larger values are associated with smoother terrain.

As the size of the structure increases, the gust effect factor reduces due to lack of correlation of the wind-induced loads over larger surface areas and a complete analysis (Category II) may be warranted. For this category and flexible or dynamically sensitive structures (Category III), a suitable method of analysis is given in the Commentary of the Standard based on the research reported by Solari (1993a, 1993b). Note that the gust effect factor does not account for across-wind loading effects, vortex shedding, instability due to galloping or flutter, or dynamic torsional effects.

Importance Factor

The importance factor I is used to set the level of risk. In the 1995 revision, the factor was moved outside of the $[1/2\rho(IV_{33})^2]$ in Equation 2, 5, 7, 8 and hence the values represent the square of the importance factors used in ASCE 7-93. The factor modifies the velocity pressure rather than the basic design speed. This was done to be consistent with the model codes. The importance factor values of 1.0, 1.15 and 0.87 are associated, respectively, with annual probability of exceedance of 0.02, 0.01 and 0.04 which correspond to 50, 100 and 25 yr. return periods, respectively.

Because the probability distribution of hurricane winds and extratropical winds are not the same (Weibull vs. Fisher Tippett Type I), different I values were used in ASCE 7-93 for the hurricane prone coastline and the inland regions (Figure 5 and Table 1). This led to some misinterpretations. Some of the codes properly specified the two sets of values, others did not (SBCCI, 1994; UBC, 1994). To avoid this confusion, Peterka (1994) wisely incorporated the difference into the hurricane contours of the map along with the difference in gust factors for the two types of winds. Additionally, note that the category designation was changed in ASCE 7-95.

Pressure Coefficients

From Equation 1, it is seen that pressure coefficients represent the non-dimensional ratio of the pressure induced at some point of the structure normalized by the velocity pressure referenced to some height above ground, i.e.,

$$C_p = \frac{p_z}{(1/2\rho V_{33}^2)K_z} \tag{10}$$

It is important to recognize at the outset that three distinctly different approaches have been used to generate these coefficients in the current Standard from data collected in wind tunnel and full-scale tests and some previously available literature. "Directional" and "envelope" (structural actions) approaches are used for MWFRS and the "envelope" (area averaging) approach is used for components and cladding.

Main Wind-Force Resisting Systems

For Main Wind-Force Resisting Systems the approaches are separated into two categories:

1. Buildings of all heights (directional approach)
2. Low-rise buildings having a height less than or equal to 60 ft (18m) (envelope approach)

The first approach is the more traditional method in which the pressure coefficients C_p reflect the mean pressures on each face of the buildings or structures for specific wind azimuths (usually relative to the principal directions of the structure for main frame loads, Figure 1). Thus, Figure 6-3 of ASCE 7-95 provides mean pressure coefficients C_p for wind directions perpendicular or parallel to the ridge line. These coefficients are multiplied by a gust effect factor based on the averaging time used in specifying the basic wind speeds. In the previous standard (1993), the gust response factor had values greater than unity to properly account for the additional loading effects due to wind turbulence over the fastest-mile-wind speed. In the recent standard, the gust effect factor based on 3-second gusts speeds have values less than unity except for flexible, buildings or other structures where dynamic amplification of the loading occurs due to wind gusts in resonance with along-wind vibrations of the structure. Additionally, as the size of the building increases, the gust effect factor may decrease below the values given earlier (0.8, 0.85) to account for lack of correlation of the wind-induced loads over larger-sized surfaces.

The second method, new in ASCE 7-95, represents an "envelope" approach in which the values of external pressure coefficients GC_{pf} represent "pseudo" loading conditions. When applied to the building, they envelope the desired structural actions (bending moment, shear, thrust) independent of wind direction and exposure. To capture all appropriate structural actions, the building must be designed for all wind directions by considering in turn each building corner as the windward corner shown in the sketches of Figure 10. Note also that for all roof slopes, load case A and load case B must be considered individually in order to determine the critical loading for a given structural assemblage or component thereof. These same two loading conditions are required for each of the windward corners to generate the wind actions including torsion, to be resisted by the structural systems. Note that the building "end zones" must be aligned in accordance with the assumed windward corner (Figure 10).

To develop the appropriate "pseudo" values of GC_{pf}, investigators at the University of Western Ontario (Davenport, et al 1978) used an approach which consisted essentially of permitting the building model to rotate in the wind tunnel through a full 360 degrees while simultaneously monitoring the loading conditions on each of the surfaces (Figure 11). Both exposures B and C were considered. Using influence coefficients for rigid frames it was possible to spatially average and time average the surface pressures to ascertain the maximum induced external and internal force components to be resisted. More specifically, the following structural actions were evaluated:

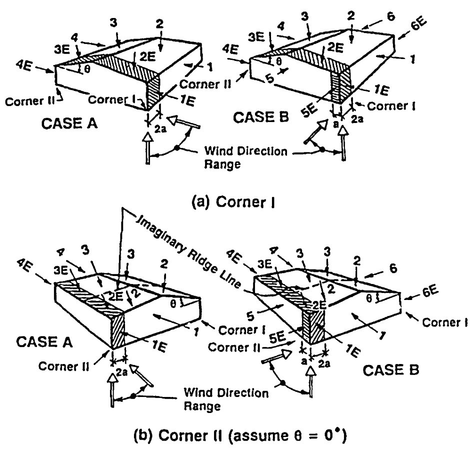
1. total uplift
2. total horizontal shear
3. bending moment at knees (two-hinged frame)
4. bending moment at knees (three-hinged frame)
5. bending moment at ridge (two-hinged frame)

Note that the shear induced at eave height was not considered as such would ordinarily not control the design of rigid frames. More will be said about this omission and its significance in a latter section.

The next step involved developing sets of"pseudo" pressure coefficients to generate loading conditions which would envelope the maximum induced force components to be resisted for all possible wind directions and exposures. Note, for example, the wind azimuth producing the maximum bending moment at the knee would not necessarily produce the maximum total uplift. The maximum induced external force components determined for each of the above five categories were used to develop the coefficients. The result was a set of coefficients that represent fictitious loading conditions, but conservatively envelope the maximum induced force components (bending moment, shear, and thrust) to be resisted, independently of wind direction.

The original set of coefficients was generated for the framing of conventional pre-engineered metal buildings, i.e., single-story moment-resisting frames in one of the principal directions and rod bracing in the other principal direction. The approach was later extended to single-story, moment-resisting frames with interior columns (Kavanagh, et al 1983).

Subsequent wind tunnel studies (Isyumov, et al 1995) suggested that the GC_{pf} values of Figure 10 were also applicable to low-rise buildings with structural systems other than moment-resisting frames. That work examined the instantaneous wind pressures on a low-rise building with a 4:12 pitched gable roof and the resulting wind-induced forces on



CASE A

Roof Angle θ (degrees)	Building Surface							
	1	2	3	4	1E	2E	3E	4E
0-5	0.40	-0.69	-0.37	-0.29	0.61	-1.07	-0.53	-0.43
20	0.53	-0.69	-0.48	-0.43	0.80	-1.07	-0.69	-0.64
30-45	0.56	0.21	-0.43	-0.37	0.69	0.27	-0.53	-0.48
90	0.56	0.56	-0.37	-0.37	0.69	0.69	-0.48	-0.48

CASE B

Roof Angle θ (degrees)	Building Surface											
	1	2	3	4	5	6	1E	2E	3E	4E	5E	6E
0-90	-0.45	-0.69	-0.37	-0.45	0.40	-0.29	-0.48	-1.07	-0.53	-0.48	-0.61	-0.43

Fig. 10. Low-rise external pressure coefficients (GC_{pf}) for MWFRS (after ASCE, 1996).

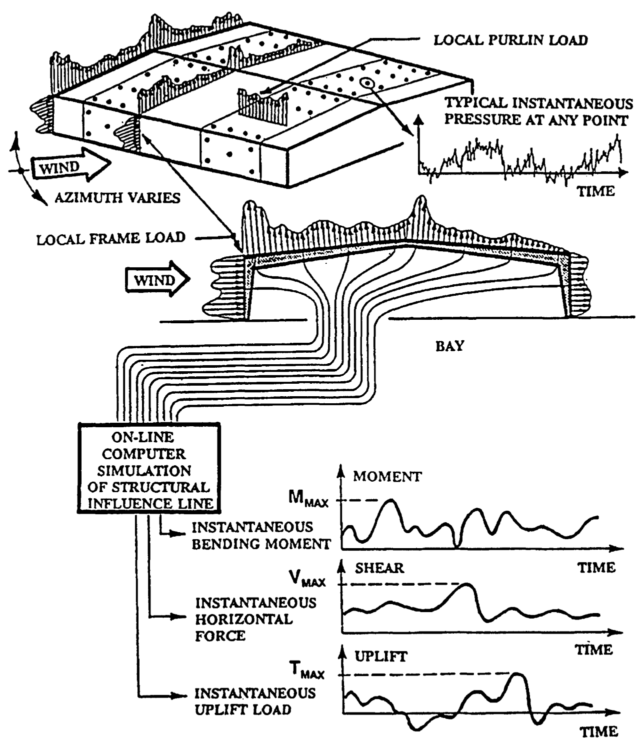


Fig. 11. Unsteady wind loads on low building (after Davenport, et al 1978).

its main wind-force resisting system. Two different main wind-force resisting systems were evaluated. One consisted of shear walls and roof trusses at different spacings. The other had moment resisting frames in one direction, positioned at the same spacings as the roof trusses, and diagonal wind bracing in the other direction. Wind tunnel tests were conducted for both Exposures B and C. The findings of this study showed that the GC_{pf} values of Figure 10 produced satisfactory estimates of the wind forces for both types of structural systems. This work was accepted by the Standard's Wind Load Task Committee as confirming the validity of Standard Figure 6-4, which reflects the combined action of wind pressures on different external surfaces of a building and thus takes advantage of spatial averaging.

Both B and C exposure terrains were checked. In these experiments, B exposure did not include nearby buildings. In general, the force components, bending moments, etc. were found comparable in both exposures, although GC_{pf} values associated with Exposure B terrain would be higher than that for Exposure C terrain because of reduced velocity pressure in Exposure B. The GC_{pf} values are associated with Exposure C terrain as obtained in the wind tunnel; hence, they are to be used with velocity pressure for Exposure C, irrespective of surrounding terrain.

In recent wind tunnel studies conducted at the University of Western Ontario (Ho, 1992), it was determined that when low-buildings [$h < 60$ ft (18 m)] are embedded in suburban terrain, the pressures in most cases are lower than those currently used in existing standards and codes, although the values show a very large scatter. The results seem to indicate that some reduction in pressures for buildings located in Exposure B are justified; and hence a 15 percent reduction in calculated pressures is permitted for buildings sited in Exposure B. Stubbs and Perry (1996) have suggested a reduction in the wind hazard for portfolios of low buildings based on insurance loss data.

Figure 6-4 of the Standard is considered appropriate for low-buildings with widths greater than twice their height and a mean roof height that does not exceed 33 ft (10 m). The original data base included low-buildings with widths no greater than five times their eave heights, and eave heights did not exceed 33 ft (10 m). In the absence of more appropriate data, the Standard suggests that Figure 6-4 may also be used for buildings with mean roof heights that do not exceed the least horizontal dimension and are less than or equal to 60 ft (18 m). Beyond these extended limits, Figure 6-3 should be used. Recent correspondence with members of the Tri-States Structural Engineering Association (Sept. 1996), however, suggests that the mean roof height should not be extended beyond 40 ft (12 m) as the original data base was limited to heights of 33 ft (10 m). An additional concern raised by some is that the internal pressure coefficients are based on a somewhat directional approach and the internal coefficients

Table 4.
Internal Pressure Coefficients for Buildings, GC_{pi}

Code/Standard	ASCE 7-95	ASCE 7-93*	SBCCI-94* MBMA-90*
Open buildings	0.00	0.00	0.00
Partially enclosed buildings	0.80 –0.30	+0.54 –0.18	+0.43 –0.14
Buildings satisfying the following conditions: (1) sited in hurricane-prone regions, and (2) having glazed openings not designed nor protected to resist wind-borne debris impact.	+0.80 –0.30		
All buildings except those listed above	+0.18 –0.18	+0.18 –0.18	+0.14 –0.14

*Adjusted for 3-second gust speed.

C_{pi} (Table 4) may be larger than the external pressure coefficients C_{pf}.

The external pressure coefficients C_p appropriate for Buildings of All Heights given in Figure 6-3 of the Standard ASCE 7-95 and the force coefficients C_f given in Tables 6-6 to 6-10 for Other Structures are unchanged from ASCE 7-93 except where updated data from the Australian Standard (1989) have been utilized. The roof pressure coefficients in Figure 6-4 require double interpolation of positive and negative values for some roof angles and permit attenuation of the coefficients with horizontal distance from the windwall edge as a function of tributary areas for the "flat roof" case ($\theta < 10°$). The question of interpolation for intermediate roof angles when both positive and negative coefficients are given has been clarified with notes 2 and 3 that accompany the figures.

In specifying the external pressure coefficients (GC_{pf}) for loads on Low-rise Buildings, the Standard's Wind Loads Subcommittee elected to accept the coefficients given in the Canadian Standard (1980) modified as follows:

$$(GC_{pf})_{7-95} = (C_P C_g)_{NBCC} \bullet \left(\frac{1}{0.80}\right) \bullet \left(\frac{1}{1.53}\right)^2 \qquad (11)$$

where

$\left(\frac{1}{0.8}\right)$ increases the coefficient to the maximum peak values measured in the wind tunnel tests (no 20 percent reduction), and,

$\left(\frac{1}{1.53}\right) = \left(\frac{V_{3600}}{V_{3-sec}}\right)$ is the ratio of mean hourly V_{3600} to 3-second gust V_{3-sec} speeds (Figure 12).

For buildings sited within Exposure B, the coefficients may be reduced 15 percent as noted above. The 20 percent reduction issue is discussed in a latter section.

Loads on Components and Cladding

In developing the set of pressure coefficients applicable for the design of components and cladding as given in Figures 6-5 through 6-7 of ASCE 7-95, an envelope approach was again followed but using different methods than for the main wind-force resisting systems of Figure 10 (Figure 6-4 of Standard). Because of the small effective area which may be involved in the design of a particular component (consider, for example, the effective area associated with the design of a fastener), the point-wise pressure fluctuations may be highly correlated over the effective area of interest. Consider the local purlin loads shown in Figure 11. The approach involved spatial averaging and time averaging of the point pressures over the effective area transmitting loads to the purlin while the building model was permitted to rotate in the wind tunnel through 360 degrees. As the induced localized pressures may also vary widely as a function of the specific location on the building, height above ground level, exposure, and more importantly, local geometric discontinuities and location of element relative to the boundaries in the building surfaces (walls, roof lines), these factors were also enveloped in the wind tunnel tests. Thus, for the pressure coefficients given in the Standard Figures 6-5 through 6-7, the directionality of the wind and influence of exposure have been removed and the surfaces of the building "zoned" to reflect an envelope of the peak pressures possible for a given design application. Again, the pressure coefficients are all referenced to Exposure C.

As indicated in the discussion of Standard Figure 6-4, the wind tunnel experiments were performed for both B and C exposure terrains. Basically, (GC_p) values associated with Exposure B terrain would be generally higher than those for Exposure C terrain because of reduced velocity pressure in Exposure B terrain. The (GC_p) values given in the Standard Figures 6-5 through 6-7 are associated with Exposure C terrain as obtained in the wind tunnel; hence, they are required to be used with velocity pressure for Exposure C, irrespective of surrounding terrain.

The wind tunnel studies conducted by Ho (1992) determined that when low-buildings, $h < 60$ ft (18 m), are embedded in suburban terrain (Exposure B), the pressures on components and cladding in most cases are lower than those currently used in the current standards and codes, although the values show a very large scatter because of high turbulence and many variables. The results seem to indicate that some reduction in pressures for components and cladding of buildings located in Exposure B is justified; a 15 percent reduction in calculated pressures is permitted for buildings sited in Exposure B. This reduction is consistent with the damageability studies noted earlier by Stubbs and Perry (1996).

To reflect the changes in basic wind speed (3-second gust), the pressure coefficients (GC_p) were adjusted as follows:

$$(GC_p)_{7-95} = (GC_p)_{7-93} \bullet \frac{(1.3)^2}{(1.53)^2} = 0.72(GC_p)_{7-93} \qquad (12)$$

where

$$\frac{(1.3)}{(1.53)} = \frac{(V_{fm}/V_{3600})}{(V_{3-sec}/V_{3600})} \qquad (13)$$

reflects the ratio of fastest-mile speed V_{fm} to hourly mean speed V_{3600} in the numerator and the ratio of 3-second gust speed V_{3-sec} to hourly mean in the denominator (Figure 12).

Internal Pressure Coefficients

A major change in philosophy in the 1995 version of the Standard concerns consideration of the influence of fluctuations of internal pressure. A new category for determining the internal pressure coefficients (GC_{pi}) in Table 4 (Table 6-4 of ASCE 7-95) has been added to reflect the observed damage to glazed openings during hurricane events and subsequent increase in internal pressure fluctuations (Sparks, et al 1990; Perry, et al 1992a, 1992b). Glazed openings in the lower 60 ft (18 m) of buildings sited in hurricane-prone regions must be protected or designed for impact by wind-borne debris. Otherwise, the internal pressure coefficients (GC_{pi}) shall be based on those for partially enclosed buildings (0.80, –0.30).

The coefficients (+0.80, –0.30) are based on wind tunnel tests conducted at the University of Western Ontario (Stathopoulos, et al 1980) and full-scale data obtained from Texas Tech (Yeatts, et al 1993). The (+0.18 and –0.18) values

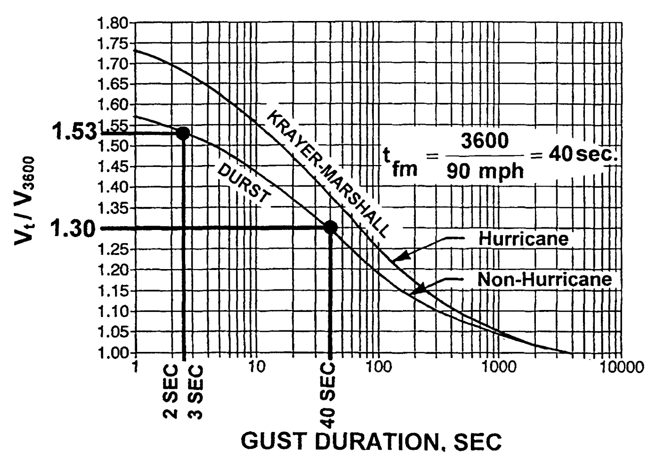


Fig. 12. Ratio of probable maximum speed averaged over t seconds V_t to hourly mean speed V_{3600} (after ASCE 7-95).

Table 5. Ratio of *(KG)* (ASCE 7-95)/*(KG)* (ASCE 7-93)				
Height Above Ground, *z* (ft)	Exposure A	Exposure B	Exposure C	Exposure D
0–15	0.91	0.75	0.68	0.64
30	0.88	0.74	0.67	0.64
60	0.84	0.72	0.67	0.64
90	0.82	0.71	0.67	0.65
140	0.81	0.70	0.67	0.65
200	0.79	0.68	0.65	0.65

Table 6. Ratio of *K* (ASCE 7-95)/*K* (ASCE 7-93)			
Height Above Ground, *z* (ft)	Exposure B	Exposure C	Exposure D
0–15	1.54	1.06	0.86
30	1.40	1.00	0.85
60	1.25	0.95	0.83
90	1.17	0.93	0.82
140	1.10	0.89	0.81
200	1.03	0.87	0.80

correspond to the coefficients given in the 1993 Standard multiplied by 0.72.

WIND LOAD COMPARISONS

During the balloting process, numerous parameter studies were presented, mostly by representatives from industry, to compare the impact of the new provisions on the market place. Most suggested the new provisions would increase the wind loads by upwards of 25 percent. Many of these efforts were flawed, however, for a number of important and significant reasons:

- They failed to recognize that the new data base reflecting wind speeds collected since 1979 suggesting that the basic design speed should be increased in some regions (and decreased in others)
- the change in importance (use) factors alluded to earlier was not properly included and/or the particular code/standard used for comparison did not include proper importance factors (e.g.; MBMA; SBCCI, Alternate Procedure; SFBC; ICBO ER 3018)
- the much needed changes in the internal pressure coefficients (GC_{pi}) were disputed (in some instances the values used were less than those required by ASCE 7-93)
- the comparison code or standard currently in use produced design loads 20–25 percent less than those based on ASCE 7-93

Each of these issues are addressed in comparing the provisions of a particular code/standard with those of ASCE 7-95.

ASCE 7-93 vs. ASCE 7-95 for Buildings of All Heights

MWFRS

Returning to Equation (8) and (9) a comparison of wind loads for Building of All Heights is given by

$$1993:\quad p_{93} = (\tfrac{1}{2}\rho)(I_{93}V_{93})^2 \bullet (KG)_{93} \bullet (C_p)_{93} \tag{14}$$

and

$$1995:\quad p_{95} = (\tfrac{1}{2}\rho)(V_{95})^2 \bullet I_{95} \bullet (KG)_{95} \bullet (C_p)_{95} \tag{15}$$

where the subscripts 93 and 95 denote the coefficients in the 1993 and 1995 versions of the Standard, respectively. The internal pressure coefficients C_{pi} have been taken as zero for this comparison.

Mehta (1996) has provided a comparison by assuming:

$V_{95} = 1.2V_{93}$

$I_{95} = I_{93}^2$ (not correct for Gulf, Atlantic hurricane zones)

$(C_p)_{95} = (C_p)_{93}$

As the intent of the Wind Loads Subcommittee was not to increase the loads (except as warranted by new data), p_{95} is taken equal to p_{93}:

$$\frac{p_{95}}{p_{93}} = 1 = \frac{(\tfrac{1}{2}\rho)(1.2V_{93})^2}{(\tfrac{1}{2}\rho)(V_{93})^2} \bullet \frac{I_{95}}{I_{95}} \bullet \frac{(KG)_{95}}{(KG)_{93}} \tag{16}$$

and

$$\frac{(KG)_{95}}{(KG)_{93}} = 0.69 \tag{17}$$

Substituting the appropriate values for K_z and G in exposures A, B, C, and D from the two Standards yields the values in Table 5.

Components and Cladding

Applying the same arguments for C&C loads, Mehta generated Table 6:

$$\frac{p_{95}}{p_{93}} = 1 = \frac{(\tfrac{1}{2}\rho)(1.2V_{93})^2}{\tfrac{1}{2}\rho(V_{93})^2} \bullet \frac{(I_{95})}{(I_{95})} \bullet \left(\frac{K_{95}}{K_{93}}\right) \bullet \left(\frac{0.72(GP_p)}{GC_p}\right)$$

where for this case

$(GC_p)_{95} = 0.72(GC_p)_{93}$

Hence

$$\frac{K_{95}}{K_{93}} = 0.96 \tag{14}$$

The comparison for Exposures B, C, and D are given in Table 6.

Again, it is seen the match is rather good for Exposures C and D. Exposure A was not included in the comparison as the Standards (1993 and 1995) require that Exposure B be used for assessing component and cladding loads for Buildings of All Heights and Other Structures sited in Exposure A. Additionally, for Low-buildings, Exposure C is used for all exposures. The loads have been increased for buildings having a height greater than 60 ft (18 m) in exposure B. Peterka (1992) found than the exposure coefficient K_z given in exposure B for the 1993 Standard was unconservative as proper turbulence effects had not been taken into consideration.

ASCE 7-95 Vs. ASCE 7-93 for Low-Buildings

It is difficult to make a simple and direct comparison between the MWFRS loads found in the new Standard for buildings less than 60 ft (18 m) in height and those in the 1993 version. If a designer elects to use the provisions for Buildings of all heights, the comparison given above is adequate for most purposes. If, on the other hand, the low-building provisions are used, two load cases (A & B) must be considered as each corner of the building is taken as the windward corner (Figure 10) as compared to just addressing the loads for each principal axis of the building (Figure 1).

Limited comparison based on base shear suggests a rather substantial decrease in the wind loads, particularly for buildings sited in exposure B where a 15 percent reduction in pressure coefficients (GC_p) and (GC_{pi}) is permitted when the low-rise provisions are used. Consider for example, a building having a height of 60 ft (18 m) and a length L to width B ratio $L/B < 1$. For this case, the new low-rise provisions predict a base shear of 79 percent of that following the provisions of Figure 6-3 of the Standard. If the height is reduced to 30 ft and 15 ft, (maintaining the same L/B ratio), the base shear predicted by the low-rise provisions would remain approximately 80 percent of those calculated from Figure 6-3.

BOCA-1996, SFBC-93, TM 4-809-1/AFM 8803

These codes/standards currently contain performance criteria for wind loads based on ASCE 7-88 (identical to ASCE 7-93). Additionally, because of the brief time available to review the new provisions, BOCA elected to adopt the new 1995 provisions as an alternate procedure. It is anticipated that South Florida Building Code (SFBC) will adopt the new ASCE 7-95 provisions in the immediate future. It is noteworthy that following Hurricane Andrew, this code group (SFBC) immediately updated their prescriptive requirements and adopted ASCE 7-88 as the performance criteria. Historically, the prescriptive provisions of SFBC for 1 and 2 family dwellings have set the standard for the United States. With the recent updates this continues to be the case. Additionally, SBFC requires an engineer's seal on residential plans.

SBCCI-1997, MBMA-1986, ICBO ER 3018-1996

The low-rise wind load provisions in SBCCI were first introduced in 1982 and were based on those contained in the guide produced by the Metal Building Manufacturers Association (MBMA). The SBCCI provisions were subsequently updated to include the new provisions contained in the 1986 version of this document and MBMA Supplement (1980). The evaluation report ICBO ER 3018 (1996) permits the use of the MBMA provisions for the design of "MBMA type" buildings in those areas of the country wherein the Uniform Building Code is the prevailing document.

The basic differences between the provisions contained in these documents and those of ASCE 7-95 are as follows:

- The (GC_p) pressure coefficients represent the combination of external and internal pressure coefficients, i.e.

$$(GC_p)_{\substack{\text{SBCCI}\\ \text{MBMA}}} = (GC_p - GC_{pi})$$

- the external pressure coefficients GC_{pf} and internal pressure coefficients (GC_{pi}) were reduced by 20 percent to reflect the joint probability of the most severe wind, approaching the building site from the most critical azimuth, occurring simultaneously with the worst combination of building geometry and terrain is lower than the probability of each event occurring individually
- the application of the "pseudo coefficients" ($GC_{pf} - GC_{pi}$) are based somewhat on "MBMA type" framing (rigid frames in one principal direction and "cable bracing" in the other). Thus, the load cases and design applications are different than those set forth in the low-rise provisions of ASCE 7-95
- to overcome the difficulties in applying the MBMA coefficients to all types of framing, SBCCI separated the MWFRS provisions into coefficients applicable for structural systems providing resistance in the

  —transverse direction
  —longitudinal direction

  as shown in Figure 13
- the MBMA provisions are based on a 50-yr return period and thus contain no importance factor (for either the inland or hurricane-prone coastline)
- the SBCCI provisions do not include separate importance factors I for the hurricane-prone coastline and thus, in effect, reduce the wind loads by 10 percent for buildings sited near the Atlantic or Gulf hurricane zones (for 50-yr return period)
- the updated internal pressure coefficients for Partially Enclosed Buildings or those sited in the hurricane-prone regions are not included

Recent conversations with SBCCI staff (Vognild, Battles 1996) indicate that code changes have been advanced for consideration at the mid-year hearings scheduled for July

1996 to adopt ASCE 7-95. No changes are anticipated for the Alternate Procedure (Section 1606.2) of SBCCI-1994 for Low-rise Buildings. If a new wind speed map is not adopted for the low-rise provisions, the wind loads predicted on the basis of ASCE 7-95 and SBCCI-1977 will differ for "enclosed buildings" by at least the ratio

$$P_{SBCCI} = (0.80)(090) = 0.72 p_{7\text{–}95}$$

for those structures sited along the hurricane coastline in exposure C where the basic design wind speed has not increased. For inland regions, the SBCCI provisions will predict loads 20 percent lower for low-rise buildings sited in exposure C and 6 percent lower for exposure B (reflecting 15 percent reduction in ASCE 7-95).

The arguments for and against the 20 percent reduction in pressure coefficients inherent in the Canadian Building Code (1990); SBCCI (1994), Alternate Procedure; and MBMA (1986), ICBO ER 3018 (1996), were revisited by the Standard's Wind Load Subcommittee. In the end, the arguments against the reduction considered earlier (Mehta, 1984) prevailed for buildings sited in Exposure C with the committee accepting a 15 percent reduction for buildings embedded within Exposure B based on the previously cited research of Ho (1992).

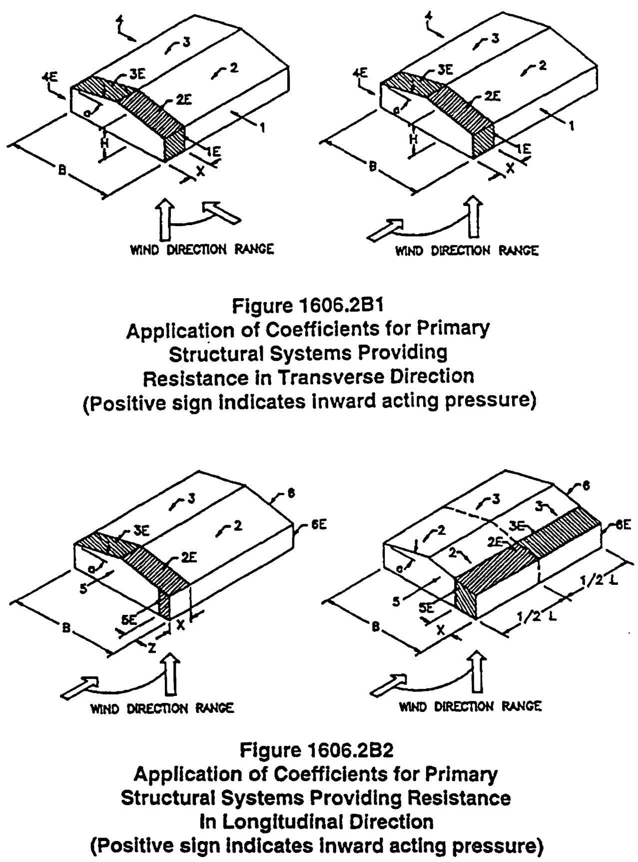


Fig. 13. Application of coefficients for primary structural systems (after SBCCI-1994).

To place the issue in proper light, one needs to recall the following:

- NBCC (1990) is based on a limit state approach in which a load factor of 1.5 is assigned for wind (with a corresponding 20 percent reduction in the pressure coefficients on the resistance side of the equation)
- the wind load provisions of NBCC (1990) assess wind loads for MWFRS and C&C based on wind speeds for 30-yr and 10-yr MRI's, respectively

whereas

- The LRFD approach of the U.S. employs a load factor for wind of 1.3, (1.3/1.5) = 0.87 with no reduction in the measured peak pressure coefficients
- the design wind speed is based on a 50-yr MRI for both MWFRS and C&C design

Additionally, when Allowable Stress Design is the design approach of choice, U.S. codes and standards permit an increase in allowable stress for wind of 33⅓ percent. It is also worthy of note that Ellingwood, et al (1980) included the influence of directionality in arriving at the 1.3 load factor for wind. Directionality was the dominant consideration for the 20 percent reduction included in the Canadian approach.

The more important issue will be the increase in internal pressure coefficients for "partially enclosed buildings" and buildings not designed for wind-borne debris in the hurricane areas. Recent conversations with SBCCI staff (Vognild, Battles 1996) indicate the SBCCI Wind Load Subcommittee has now been elevated to full Standing Committee status and is currently studying the new changes in ASCE 7-95 for low-buildings. They are also directing a great deal of attention to developing a hurricane resistant standard for glazed openings; this study could have a significant impact on the issue of wind-borne debris (Table 4).

UBC-1977

The Tri-states Structural Engineers Association (SEAOC, SEAO and SEAW) has been working at an ever increasing pace (Scott, 1996) to develop new performance requirements for inclusion in the 1997 code. The requirements will follow the same format as previous editions, and represent an ambitious attempt to produce a more "user friendly" version of ASCE 7-95. If their efforts are not successful in time for adoption in the 1997 code, one must rely on UBC-94, the wind load provisions of which, compare favorably with ASCE 7-93. There exist a number of differences, however, principally with respect to "zoning" the surfaces for localized pressures; but, they are relatively insignificant. Additionally, the UBC Code does not include an increase in the importance factor I for the hurricane zones. Thus, a proper comparison would follow, for the most part, the comparisons between the 1993 and 1995 versions of ASCE-7 given above.

SBCCI SSTD 10-93, TDI-96, BLUE SKY-96

The development of the prescriptive provisions contained in these "deemed-to-comply" documents is noteworthy and their adoption for the construction of single and multiple family dwellings in the high wind areas will arguably lead to a reduction in wind induced losses. Stubbs and Perry (1995) conducted a cost-benefit analysis of the TDI Code (1995) and found the provisions most cost-effective, although not optimal. Although the SBCCI and Blue Sky provisions are based on the alternate provisions of SBCCI (alternative method) and the TDI code was developed from the ASCE 7-93 provisions, and could be updated, the documents stand as our best hope in reducing damage due to future hurricane events. Given our lack of understanding of the load paths and modes of resistances of residential construction, there exists no need to update these provisions at the moment except to include proper provisions for the roof coverings and siding materials.

CLOSURE

Troublesome items still exist in ASCE 7-95 that will, unfortunately, influence the immediate acceptance of the Low-rise Buildings provisions:

- The "enveloping" of structural actions for main frame loads (MWFRS) as compared to "directional loads" will initially cause concern for the practitioner. Given the availability of software, this problem will go away with time
- the application of the low-rise provisions to building geometries (height, *h/B*, *h/L* ratios) beyond those considered in the wind tunnel experiments. The Tri-states SEA has recommended that the provisions be limited to a maximum building height of 40 ft (12.2 m)
- the question whether the provisions apply to all types of framing or just to "MBMA type" buildings will linger on,
- the anomaly that surfaces when buildings have roof slopes in the range of 20–30 degrees (roof level shear was not considered in the enveloping process)
- the rather substantial differences between the wind loads predicted by the provisions for Buildings of All heights (Figure 6-3) and Low-rise Buildings (Figure 6-4)
- the increase in internal pressure considerations for hurricane areas
- the trend towards developing new prescriptive ("deemed-to-comply") code requirements based on the provisions of ASCE 7

One is tempted to let time arbitrate these issues, but the year 2000 is fast approaching, and design professionals, industry, building code officials, and the consumer would be well served by a common U.S. Code.

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