



**National Institute of Standards and Technology**  
Technology Administration, U.S. Department of Commerce

## **NIST TECHNICAL NOTE 1738**

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# **An Assessment of Methods for Determining Wind Loads**

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February 2012

U.S. Department of Commerce  
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National Institute of Standards and Technology  
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## **Abstract**

The purpose of this work is to present an assessment of methods for determining wind loads on buildings and other structures that warrant comment, correction or improvement. The assessment is intended to serve as a resource as a new version of the American Society of Civil Engineers ASCE-7 Standard is being prepared. Issues discussed in the paper include: wind speeds in non-hurricane regions; alternative analytical methods for determining wind loads on Main Wind Force Resisting Systems and Components/Cladding; aerodynamic pressure coefficients; pressures on rooftop equipment; component and cladding pressures on arched roofs; and the wind tunnel procedure.

**Keywords:** Arched roofs; rooftop equipment; wind engineering; wind loads; wind pressures; wind speeds; wind tunnels.



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## 1. Introduction

The purpose of this work is to present an assessment of methods for determining wind loads on buildings and other structures. The assessment, while not exhaustive, presents facts and results intended to constitute a useful resource as a new version of the American Society of Civil Engineers ASCE-7 Standard is being prepared.

The following issues are discussed in this work:

- Wind speed maps for non-hurricane regions
- Alternative analytical methods for determining wind loads on Main Wind Force Resisting System and Component/Cladding
- Aerodynamic pressure coefficients
- Pressures on rooftop equipment
- Component and cladding pressures on arched roofs
- The wind tunnel procedure.

Improvements in the methods for determining wind loads on buildings and other structures can eliminate the underestimation of wind effects on some types of buildings in regions where wind speeds are higher than those estimated, for example, by Peterka and Shahid (1998); whereas in regions where wind speeds are lower, such improvements can reduce (a) unnecessary costs due to overestimates of wind effects, and (b) unnecessary consumption of embodied energy. Reference is made, where necessary, to procedures incorporated in ASCE (2010). Note that these procedures are referenced in the International Building Code (ICC 2012).

## 2. Wind Speeds for Non-Hurricane Regions

According to Simiu et al. (2003), methodological errors have led to the estimation by Peterka and Shahid (1998) of peak-gust wind speeds for non-hurricane regions that in some areas are larger, while in other areas are smaller than correct statistical estimates would indicate. Indeed, to obtain the wind speed estimates, the data sets for several stations were pooled to form superstation data sets, a procedure that is legitimate provided that it is applied correctly from a statistical point of view. However, in developing the peak gust speeds maps, 80 % of the superstations included stations that were also included in other superstations (see <http://www.nist.gov/wind>, I. Extreme Winds, Data Sets, 4. Texas Tech/CSU data). This led to the artificial smoothing out of geographical variations of the extreme wind climate. To see why this is the case, consider the following schematic example. Superstation A comprises stations a, b, c, d. Superstation B comprises stations e, f, and again c and d. Superstation C comprises stations g, h, d, and f. It is clear that this procedure for creating superstations results in correlations among speeds esti-

mated from different superstation data that do not exist in nature. In addition, stations with different physical geography and meteorology were in many cases included in the same superstation. One example is superstation 99927, which includes stations such as New York/Central Park, Albany, NY, Belmar, NJ, Providence, RI, Albany, NY, Boston, MA, and other 17 coastal and inland stations, located in the states of New Jersey, New York, Connecticut, Massachusetts, and Rhode Island (Simiu et al., 2003).

The following examples suggest that the uniformity of the extreme wind climate over vast regions of the country can be consequential from a structural engineering point of view. Within regions where the ASCE 7-02 Standard (ASCE, 2003) specifies a 3-s 50-year speed of 90 mph, the ASCE 7-88 Standard (ASCE, 1990) specifies a basic 50-year fastest-mile wind speed of 80 mph (36 m/s). This is equivalent to a 96 mph (43 m/s) 3-s peak gust of 10 m above ground. The square of the ratio 90/96 – which is relevant to the calculation of aerodynamic pressures or forces – is 0.88. The ASCE 7-88 map was developed from well-documented fastest-mile wind speed data samples with sizes typically larger than those of the 3-s peak wind speeds used to obtain the ASCE 7-2003 map. It may therefore be argued that the square of the ratio 90/96 provides at least a qualitative measure of the underestimation by ASCE (2003) of the basic 50-year wind speed over large areas of the U.S. (The underestimation in terms of wind effects is larger for flexible structures, for which the wind effect is proportional to the wind speed raised to a power larger than two.) In regions for which, according to ASCE (1990) the fastest-mile basic 50-year wind speed is 85 mph (38 m/s), which is equivalent to 101 mph (45 m/s), the square of the ratio 90/101 is 0.79. In some regions subjected to strong thunderstorms, ASCE (1990) specifies a basic 50-year fastest-mile wind speed of 90 mph (40 m/s), which is equivalent to a 3-s speed of about 106 mph (47 m/s). The square of the ratio 90/106 is 0.72. Similar or larger differences are inherent in wind maps obtained by calibration against the Peterka and Shahid (1998) map for mean recurrence intervals of the peak gust speed longer than 50 years. For additional details, see Simiu, Lombardo, and Yeo (2012).

New wind speed maps and databases are being developed by the National Institute of Standards and Technology (NIST)'s National Windstorm Impact Reduction Program and Statistical Engineering Division. These maps and databases will be provided to the ASCE 7 Subcommittee on Wind Loads for discussion and possible incorporation into the ASCE 7-16 Standard. The maps will be based on data measured at nearly 1200 Automated Surface Observing System (ASOS) stations with the majority of the stations having records approaching 30 to 40 years in length (by comparison, the wind speed map in ASCE 7-02 was developed from approximately 500 stations typically having 15 to 25 years of data). The methodology to extract ASOS data is described in Lombardo et al. (2009), which also describes in detail the analysis procedure, based on mixed probability distributions that account separately for thunderstorm and non-thunderstorm wind speeds. The improvements in climatological data volume, spatial resolution, and analysis are expected to lead to improved wind speed maps.

### 3. Analytical Methods for Determining Wind Loads

For some types of Main Wind Force Resisting Systems, and Components and Cladding, at least two, three, or even four alternative analytical methods are available for determining wind loads. For example, enclosed simple diaphragm low-rise buildings can be designed using four different methods to determine Main Wind Force Resisting System loads – the Directional Procedure, the Simplified Directional Procedure, the Envelope Procedure, and the Simplified Envelope Procedure (ASCE 2010).

Where more than one method is available the user needs to know to what extent the choice of method matters. In some instances some guidance is offered on this issue. Examples 1 and 2 below suggest that the guidance is not always dependable.

**Example 1.** *Main Wind Force Resisting Systems (MWFRS): Directional procedure for buildings of all heights, ASCE (2010), Sect. 27.4.1 vs. Envelope procedure for low-rise buildings, ASCE (2010), Sect. 28.4.1.*

Consider a rectangular office building with dimensions in plan of 45 ft × 40 ft (13.7 m × 12.2 m), eave height 15 ft (4.6 m), gable roof with slope  $\theta = 26.6^\circ$  and mean roof height  $h = 15 \text{ ft} + \frac{1}{2}(\frac{1}{2} \times 40 \text{ ft}) \tan 26.6^\circ = 20 \text{ ft}$  (6.05 m). (Since  $h < 60 \text{ ft}$  (18.3 m) and  $h$  /least horizontal dimension =  $20/40 < 1$ , the building is defined in current practice as a low-rise building.) Assume that the building is in flat terrain, has suburban exposure in all directions, and is fully enclosed. At some locations, for wind parallel to the short building dimension the envelope procedure yields pressures higher in absolute value by 25 % than the directional procedure; for wind parallel to long building dimension it yields pressures higher in absolute value by 50 % to 60 % than the directional procedure (for details see Simiu 2011, pp. 47, 59, 62).

It is stated in ASCE (2010), Sect. 28.2 that the envelope procedure “...generally yields the lowest wind pressure of all of the analytical methods specified in this standard.” Example 1, which was chosen at random, shows that this is not necessarily the case.

**Example 2.** *MWFRS: Directional procedure (regular approach for buildings of all heights, ASCE (2010), Sect. 27.4.1) vs. Envelope procedure (regular approach for low-rise buildings, ASCE (2010), Sect. 28.4.1) vs. Directional procedure (simplified approach for buildings of all heights, ASCE (2010), Sect. 27.6.*

**Case I.** For the building described in Example 1, assumed to be a simple diaphragm building, for flow direction parallel to the ridge, Zone 3 of the gable roof (ASCE 2010, Table 27.6-2, Directional Procedure, simplified approach) may be considered to correspond to Zone 2E, Load Case B (ASCE 2010, Fig. 28.4-1, Low-Rise Buildings, regular approach). Assuming that the basic wind speed is  $V = 115 \text{ mph}$  (51.4 m/s), the pressures on the roof for these zones are (for details see Simiu, 2011, pp. 47, 59, 82, 84):

Directional procedure, regular approach, buildings of all heights:  $p = -16.8 \text{ psf}$  (803 Pa)

Envelope procedure, regular approach, low-rise buildings:  $p = -25.1 \text{ psf}$  (1200 Pa)

Directional procedure, simplified approach, buildings of all heights:  $p = -19.0 \text{ psf}$  (708 Pa)

Note that in this example also the envelope procedure yields a larger pressure than the directional procedure.

**Case II.** The building to be designed has rectangular shape in plan [60 ft (18.3 m) × 125 ft (38.2 m)], eave height 95 ft (29 m), and flat roof; it is located at the southern tip of the Florida peninsula in flat, suburban terrain in all directions. The building is assumed to be an enclosed simple diaphragm building. It is assumed that for this building the fundamental natural frequency in Hz is not less than  $75/h$  ( $h$  in feet) [ $22.8/h$  ( $h$  in m)]. Since  $h > 60$  ft (18.3 m), the building is not Class 1 (ASCE 2010, Sect. 27.5.2). Since  $60 \text{ ft (18.3 m)} < h \leq 160 \text{ ft (48.8 m)}$ , and the ratio between the dimensions of the building in plan is not less than 0.5 nor more than 2.0 (ASCE 2010, Sect. 27.5.2), the building is a Class 2 building.

As determined by the simplified approach, for wind parallel to the long building dimension, the external pressures at the top of the windward, leeward, and side walls (internal pressures are not included for MWFRS design) are, respectively,

51.7, -19.1, and -45.3 psf (2472, -913, and -2166 Pa).

If determined by the regular approach the pressures are, respectively,

41.9, -15.7, and -36.7 psf (2000, -751, and -1755 Pa).

The pressures determined by the simplified approach are larger in absolute value than those determined by the regular approach by 20 to 25 %. For details, see Simiu (2011, pp. 38, 86, 90).

**Example 3. Components and Cladding: Regular approach for  $h \leq 60 \text{ ft (18.3 m)}$  (ASCE 2010, Sect. 30.4) vs. Simplified approach for  $h \leq 60 \text{ ft (18.3 m)}$  (ASCE 2010, Sect. 30.5) vs. Simplified approach for  $h \leq 160 \text{ ft (48.8 m)}$  (ASCE 2010 Sect. 30.7).** For an enclosed office building with height  $h = 60 \text{ ft (18.3 m)}$ , we assume: area of the cladding  $4 \text{ ft}^2$  ( $0.37 \text{ m}^2$ ) flat roof; suburban exposure; flat terrain; basic wind speed 115 mph (51.4 m/s). For wall Zones 4 and 5 (ASCE 2010), the calculated pressures, in psf, are listed in Table 1. The largest and smallest Zone 4 and Zone 5 pressures are shown in bold. The differences between those pressures are as high as about 20 % and 40 %, respectively (see Simiu 2011, pp. 100, 105, 106).

Table 1. Pressure Comparisons (1 psf = 47.88 Pa)

	Regul. proc., $h \leq 60 \text{ ft (18.3 m)}$	Simpl. proc., $h \leq 60 \text{ ft (18.3 m)}$	Simpl. proc., $h \leq 160 \text{ ft (48.8 m)}$
Zone 4	-28.7 psf	<b>-31.5 psf</b>	<b>-26.5 psf</b>
Zone 5	<b>-35.3 psf</b>	-38.9 psf	<b>-48.7 psf</b>

#### 4. Pressure Coefficients for Low-Rise Structures: Envelope Method versus Wind-Tunnel Measurements

We consider in this section only pressures on low-rise buildings, on which several studies are available. A procedure for low-rise buildings entails the use of tailored coefficients applicable to portal frames of industrial buildings and referred to as “pseudo-pressure” coefficients. These coefficients are based on wind tunnel data measured at the University of Western Ontario (UWO) mostly in the 1970s (Davenport, Stathopoulos, and Surry, 1978), and were developed with a view to enveloping the frame’s peak load effects: bending moments, resultant vertical uplift, and horizontal shear for about 15 distinct building geometries. St. Pierre et al. (2005) compared these quantities as obtained from pressure coefficient plots in ASCE (2003) (identical to the corresponding pressure plots in the later versions of the Standard) to their counterparts calculated from pressures measured also at UWO but by using state-of-the-art experimental techniques (Ho et al. 2005). They reported that the responses predicted by ASCE (2003) were in many cases lower in absolute value by about 30 % than the responses obtained using their recent pressure measurements. These discrepancies are attributed to the fact that the earlier experiments were conducted in flows with lower turbulence intensities, for wind directions in increments of mostly 45°, as opposed to 5° in the later tests, with a number of pressure taps lower by almost one order of magnitude, and with fixed distances between frames, thus disregarding the dependence of the loads on distance between frames. In addition, even if the coefficients resulted in estimates of the bending moments at the frame knees and ridge to within, say, 10 % of the actual values (which is not always the case), their suitability for calculating bending moments at other locations is questionable. The results obtained by St. Pierre et al. (2005) were confirmed by Coffman et al. (2010), who analyzed seven portal frame buildings with open terrain exposure for which pressure measurements by Ho et al. (2005) are recorded in the NIST aerodynamics database (<http://www.nist.gov/wind>). Coffman et al. (2010) found that, “depending on the building dimensions, the peak bending moments at the knee based on Database-Assisted design (DAD) techniques are generally larger by 10 to 30 % than their counterparts based on the ASCE (2006). (In one case with a relatively steep roof slope of 26.6° the discrepancies exceed 70 %.) For the buildings considered, the discrepancies increase with increasing roof slope and with increasing eave height.” From results reported by Fritz et al. (2008) it may be surmised that the discrepancies would be larger for buildings in suburban terrain.

ASCE’s Technical Council on Wind Engineering has identified the need for an “...extensive program of wind tunnel testing to establish design pressure coefficients for a wide range of different shapes...”, citing concerns with the existing ASCE 7 Standard, including the concern that existing pressure coefficients are based on tests done over 30 years ago, using wind tunnel technology far less advanced than available today (Irwin, 2011). Such a testing program has recently been performed by Tokyo Polytechnic University (TPU), which has issued a large public aerodynamic database that could be used as a coherent, traceable source of data for the development of improved provisions on wind pressure coefficients for a large variety of building configurations (Tamura, 2011; [www.wind.arch.t-kougei.ac.jp/system/eng/contents/code/w\\_it](http://www.wind.arch.t-kougei.ac.jp/system/eng/contents/code/w_it)). An evaluation of the TPU data is being performed through comparisons with existing data obtained in wind tunnel

(e.g., UWO and Colorado State University), full-scale, and large-scale facility measurements.

## 5. Wind Loads on Rooftop Equipment

Two procedures for estimating forces on rooftop equipment are currently available for widespread use in the United States (U.S.). The choice of procedure depends on building height. If the height of a building is over 60 ft (18.3 m) the procedure included in ASCE (2010) as Eq. 29.5-1 is applicable. For buildings with height less than or equal to 60 ft (18.3 m) the procedure included in ASCE (2010) as Eq. 29.5-2 is applicable. The two procedures are mutually inconsistent insofar as, for values close to 60 ft (18.3 m), they yield markedly different results.

**Example 4.** Rooftop equipment, Directional procedure: approach for  $h > 60$  ft (18.3 m) (ASCE Sect. 29.5) vs. approach for  $h \leq 60$  ft (18.3 m) (ASCE Sect. 29.5.1).

Two rectangular office buildings, located in Iowa in flat terrain with exposure B in all directions, have dimensions in plan 45 ft  $\times$  45 ft (13.7 m  $\times$  13.7 m) and a flat roof, but eave height 62.5 ft (19.1 m) and 60 ft (18.3 m), respectively. The rooftop equipment is a cube structure with 2.4 ft (0.73 m) sides. The procedure applicable to the building with  $h = 62.5$  ft (19.1 m) estimates a design lateral wind force of 180 lb (802 N), while the approach for the building with  $h = 60$  ft (18.3 m) results in a design lateral wind force of 400 lb (1780 N) and the design vertical uplift wind force of 220 lb (980 N). For details see Simiu (2011, p. 77). Note that the vertical uplift force on rooftop equipment is not considered in the procedure for buildings with height greater than 60 ft (18.3 m).

To correct this inconsistency, a possible option would be to apply conservatively the provision restricted to buildings with  $h \leq 60$  ft (18.3 m) of ASCE Sect. 29.5.1 to buildings of all heights. The proposed unified approach would also specify an uplift force on rooftop structures, regardless of building height. This approach is consistent with results of tests reported by Baines (1963). Note that code change recommendations for the Florida Building Code (FBC, 2010) to correct the inconsistency pertaining to wind loading on rooftop equipment were unanimously approved at the Florida Building Commission meeting on December 8, 2010 in Melbourne, Florida.

## 6. Wind Loads for Components and Cladding of Arched Roofs

A procedure for estimating wind loads for components and cladding of arched roofs is available in Fig. 27.4-3, ASCE (2010). The procedure specifies that the pressure coefficients for components and cladding should be equal to the pressure coefficients for the Main Wind Force Resisting Systems multiplied by the factor 0.87 (Fig. 27.4-3, Note 4). In fact, owing to the fact that the spatial coherence of the pressures is greater between pressures acting over small than over large surfaces, pressure coefficients for components and cladding should not be smaller than the pressure coefficients for Main Wind Force Resisting Systems.

## 7. Insufficient Specificity of Provisions on the Wind Tunnel Procedure

Largely because the U.S. provisions with respect to the wind tunnel method lack sufficient specificity, discrepancies can occur among estimates of wind effects obtained by various laboratories. This has been confirmed, for example, by a difference of more than 40 % between estimates by two independent laboratories of the response of the World Trade Center twin towers to wind (SOM 2004). For low-rise buildings differences among results obtained in six major wind tunnels in the U.S., Canada, France, and Japan can be even greater (Fritz et al. 2008).

The simulation of the natural wind at specific locations and the evaluation of wind induced loads and/or structural responses are performed by methods that may differ from wind tunnel to wind tunnel. Principal reasons for possible differences among response estimates by various wind engineering consultants are differences in (NIST 2005):

- Statistical models of the wind climate for the projects being considered.
- Predictive methods which combine wind tunnel test results with wind climate data to predict prototype wind loads and responses.
- Combining effects of wind action along the  $x$  principal axis *and* the  $y$  principal axis *and* in torsion, and/or interactions of internal forces and moments for member design (see, e.g., Yeo and Simiu 2011), and estimating combined wind effects with specified mean recurrence intervals needed for structural design purposes.
- Accounting for wind directionality in the prediction of wind. For example, (a) the sector-by-sector method, requires information on extreme wind speeds from different azimuths; (b) the method of up-crossings uses the parent distribution of wind speeds and directions; (c) the simulated storm passages method tracks the effects of storm passages in the time domain, and is particularly effective in the prediction of wind action during passages of hurricanes or typhoons (Isyumov et al. 2003).

Estimates of the local directional wind climate are particularly sensitive to the source of wind data, to the scrutiny of the data samples aimed at removing incorrect data (e.g., observations from malfunctioning instruments), and to analyses aimed at dealing with statistical outliers. While the design wind speed for a particular project can be mandated, criteria related to wind directionality are difficult to specify. This can result in appreciable differences between predictions of wind effects for buildings and structures whose response is sensitive to wind direction.

One possible approach for limiting differences due to wind directionality is to require that laboratories evaluate the reliability of their wind climate descriptions and make allowance for the uncertainties in those descriptions. For example, the effects of uncertainties in wind directionality can be reduced by rotating the extreme wind rosette by, say, 15 degrees clockwise as well as counter-clockwise when determining the wind effects. Another useful approach is to put in place a lower bound on the ratio of predictions obtained with and without the use of wind directionality. The limitation on wind loads obtained by using the Wind Tunnel Procedure specified in Sect. 31.4.3 of ASCE 7-10 is helpful in eliminating possibly unconservative results.

Progress in the resolution of the issues mentioned in this section would contribute to the necessary improvement of provisions on the wind tunnel procedure and to the reduction of considerable differences that might occur among results obtained by various laboratories.

## 8. Conclusions

It is noted in the paper that, for non-hurricane regions, inadequately differentiated wind speed maps can result in the underestimation of basic wind speeds in some regions of the country and in their overestimation in other regions. New wind speed maps and databases are being developed by NIST's National Windstorm Impact Reduction Program and Statistical Engineering Division. These maps and databases will be provided to the ASCE 7 Subcommittee on Wind Loads for discussion and possible incorporation into the ASCE 7-16 Standard.

It is also noted that, according to recent studies, pressure coefficients for low-rise buildings used in the U.S. within the framework of the so-called "envelope procedure" are in many cases lower by as much as 30 % than values obtained by state-of-the-art wind tunnel testing methods. The effects of the underestimation of basic wind speeds, and of pressure coefficients for low-rise buildings, can lead in some situations to designs that do not meet intended minimum requirements for wind loads.

Results of calculations shown in the paper demonstrate that alternative analytical methods for the determination of design wind loads can produce significantly different results. In particular, the "envelope method" (ASCE 2010) can yield design wind pressures that differ from -- and are typically lower than -- values yielded by recent, state-of-the-art measurements obtained at the University of Western Ontario's Boundary Layer Wind Tunnel Laboratory.

It is pointed out in the paper that current provisions for roof-top equipment and for components/cladding for arched roofs are inadequate and, where feasible, suggestions are presented for an improvement of those provisions.

With respect to existing provisions on the wind tunnel procedure, it is noted that they are not sufficiently specific, and that this can explain large discrepancies that have been found to exist between estimates of wind effects on buildings performed by different wind tunnels.

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