Tables 6-8 to 6-13. With the exception of Table 6-13, the pressure and force coefficient values in these tables are unchanged from ANSI A58 1-1972 and 1982, and ASCE 7-88 and 7-93. The coefficients specified in these tables are based on wind-tunnel tests conducted under conditions of uniform flow and low turbulence, and their validity in turbulent boundary layer flows has yet to be completely established. Additional pressure coefficients for conditions not specified herein may be found in two references [3], [36]. With regard to Table 6-10, local maximum and minimum peak pressure coefficients for cylindrical structures with $h/D<2$ are $GC_p = 1.1$ and $GC_p = -1.1$, respectively, for Reynolds numbers ranging from $1.1 \times 10^5$ to $3.1 \times 10^5$ [23]. The latter results have been obtained under correctly simulated boundary layer flow conditions.

With regard to Table 6-13, the force coefficients are a refinement of the coefficients specified in ANSI A58 1-1982 and in ASCE 7-93. The force coefficients specified are offered as a simplified procedure that may be used for trussed towers and are consistent with force coefficients given in ANSI/EIA/TIA-222-E-1991, Structural Standards for Steel Antenna Towers and Antenna Supporting Structures, and force coefficients recommended by Working Group No. 4 (Recommendations for Guyed Masts), International Association for Shell and Spatial Structures (1981).

It is not the intent of the Standard to exclude the use of other recognized literature for the design of special structures such as transmission and telecommunications towers. Recommendations for wind loads on tower guys are not provided as in previous editions of the Standard. Recognized literature should be referenced for the design of these special structures as is noted in C6 4.2.1. For the design of flagpoles, see ANSI/NAAMM FP1001-97, 4th Ed , Guide Specifications for Design of Metal Flagpoles.

**C6.5.11.1 Internal Pressure Coefficients** The internal pressure coefficient values in Table 6-7 were obtained from wind tunnel tests [38] and full scale data [59] Even though the wind-tunnel tests were conducted primarily for low-rise buildings, the internal pressure coefficient values are assumed to be valid for buildings of any height. The values $GC_{pi} = +0.18$ and $-0.18$ are for enclosed buildings. It is assumed that the building has no dominant opening or openings and that the small leakage paths that do exist are essentially uniformly distributed over the building's envelope. The internal pressure coefficient values for partially enclosed buildings assume that the building has a dominant opening or openings. For such a building, the internal pressure is dictated by the exterior pressure at the opening and is typically increased substantially as a result. Net loads, i.e., the combination of the internal and exterior pressures, are therefore also significantly increased on the building surfaces that do not contain the opening Therefore, higher $GC_{pi}$ values of $+0.55$ and $-0.55$ are applicable to this case These values include a reduction factor to account for the lack of perfect correlation between the internal pressure and the external pressures on the building surfaces not containing the opening [82] [83] Taken in isolation, the internal pressure coefficients can reach values of $\pm 0.8$, (or possibly even higher on the negative side).

For partially enclosed buildings containing a large unpartitioned space the response time of the internal pressure is increased and this reduces the ability of the internal pressure to respond to rapid changes in pressure at an opening. The gust factor applicable to the internal pressure is therefore reduced Equation 6-14 which is based on references [84] and [85] is provided as a means of adjusting the gust factor for this effect on structures with large internal spaces such as stadiums and arenas

Glazing in the bottom 60 ft of buildings that are sited in hurricane-prone regions that is not impact resistant glazing or is not protected by impact resistant coverings should be treated as openings. Because of the nature of hurricane winds [27], glazing in buildings sited in hurricane areas is very vulnerable to breakage from missiles, unless the glazing can withstand reasonable missile loads and subsequent wind loading, or the glazing is protected by suitable shutters. Glazing above 60 ft (18 m) is also somewhat vulnerable to missile damage, but because of the greater height, this glazing is typically significantly less vulnerable to damage than glazing at lower levels. When glazing is breached by missiles, development of high internal pressure results, which can overload the cladding or structure if the higher pressure was not accounted for in the design. Breaching of glazing can also result in a significant amount of water infiltration, which typically results in considerable damage to the building and its contents [33], [49], [50].

If the option of designing for higher internal pressure (versus designing glazing protection) is selected, it should be realized that if glazing is breached, significant damage from overpressurization to interior partitions and ceilings is likely. The influence of compartmentation on the distribution of increased internal pressure has not been researched. If the space behind breached glazing is separated from the remainder of the building by a sufficiently strong and reasonably air-tight compartment, the increased internal pressure would likely be confined to that compartment. However, if the compartment is breached (e.g., by an open corridor door, or by collapse of the compartment wall), the increased internal pressure will spread beyond the initial compartment quite rapidly The next compartment may contain the higher pressure, or it too could be breached, thereby allowing the high internal pressure to continue to propagate.

Because of the great amount of air leakage that often occurs at large hangar doors, designers of hangars should consider utilizing the internal pressure coefficients for partially enclosed buildings in Table 6-7

**C6.5.12 Design Wind Loads on Buildings.** This version of the standard states in the body of the standard specific wind pressure equations for both the main wind force resisting systems and components and cladding.

In equations 6-15, 6-17, and 6-19 a new velocity pressure term "$q_i$" appears that is defined as the "velocity pressure for internal pressure determination" The positive internal pressure is dictated by the positive exterior pressure on the windward face at the point where there is an opening The positive exterior pressure at the opening is governed by the value of q at the level of the opening, not $q_h$. Therefore the old provision which used $q_h$ as the velocity pressure is not in accord with the physics of the situation. For low buildings this does not make much difference, but for the example of a 300 ft tall building in Exposure B with a highest opening at 60 ft. the difference between $q_{300}$ and $q_{60}$ represents a 59% increase in internal pressure. This is unrealistic and represents an unnecessary degree of conservatism. Accordingly, $q_i = q_z$ for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure For buildings sited in wind borne debris regions, glazing that is not impact resistant or protected with an impact resistant covering, $q_i$ should be treated as an opening For positive internal pressure evaluation, $q_i$ may conservatively be evaluated at height h ($q_i = q_h$)

**C6.5.12.3 Full and Partial Loading.** Tall buildings should be checked for torsional response induced by partial wind loading and by eccentricity of the elastic center with respect to the resultant wind load vector and the center of mass. The load combinations described in Figure 6-9 reflect surface pressure patterns that have been observed on tall buildings in turbulent wind. Wind tunnel tests have demonstrated that even a 25% selective load reduction can underestimate the wind-induced torsion in buildings with a uniform rectangular cross-section [50]. In some structural systems, more severe effects are observed when the resultant wind load acts diagonally to the building or other structure. To account for this effect and the fact that many structures exhibit maximum response in the across-wind direction, a structure should be capable of resisting 75% of the design wind load applied simultaneously along the principal axes. Additional information on torsional response due to full and partial loading can be found in the literature [2], [4], [17].

**C6.6 Method 3 - Wind-Tunnel Procedure.** Wind tunnel testing is specified when a structure contains any of the characteristics defined in 6.5 2 or when the designer wishes to more accurately determine the wind loads. For some building shapes wind tunnel testing can reduce the conservatism due to enveloping of wind loads inherent in Methods 1 and 2 Also, wind tunnel testing accounts for shielding or channeling and can more accurately determine wind loads for a complex building shape than Methods 1 and 2 It is the intent of the standard that any building or other structure be allowed to use the wind tunnel testing method to determine wind loads. Requirements for proper testing are given in 6.6 2.

Wind-tunnel tests are recommended when the building or other structure under consideration satisfies one or more of the following conditions:

1. has a shape which differs significantly from a uniform rectangular prism or "box-like" shape,
2. is flexible with natural frequencies normally below 1 Hz.
3. is subject to buffeting by the wake of upwind buildings or other structures, or
4. is subject to accelerated flow caused by channeling or local topographic features

It is common practice to resort to wind-tunnel tests when design data are required for the following wind-induced loads

1. curtain wall pressures resulting from irregular geometry,
2. across-wind and/or torsional loads.
3. periodic loads caused by vortex shedding, and
4. loads resulting from instabilities such as flutter or galloping

Boundary-layer wind tunnels capable of developing flows that meet the conditions stipulated in 6.4 3.1 typically have test-section dimensions in the following ranges; width of 6-12 ft (2-4 m), height of 6-10 ft (2-3 m), and length of 50-100 ft (15-30 m). Maximum wind speeds are ordinarily in the range of 25-100 mph (10-45 m/s). The wind tunnel may be either an open-circuit or closed-circuit type.

Three basic types of wind-tunnel test models are commonly used. These are designated as follows (1) rigid pressure model (PM); (2) rigid high-frequency base balance model (H-FBBM), and (3) aeroelastic model (AM). One or more of the models may be employed to obtain design loads for a particular building or structure. The PM provides local peak pressures for design of elements such as cladding and mean pressures for the determination of overall mean loads. The H-FBBM measures overall fluctuating loads (aerodynamic admittance) for the determination of dynamic responses. When motion of a building or structure influences the wind loading, the AM is employed for direct measurement of overall loads, deflections and accelerations Each of these models, together with a model of the surroundings (proximity model), can provide information other than wind loads such as snow loads on complex roofs, wind data to evaluate environmental impact on pedestrians, and concentrations of air-pollutant emissions for environmental impact determinations Several references provide detailed information and guidance for the determination of wind loads and other types of design data by wind-tunnel tests [4], [7], [8], [33]

Wind tunnel tests frequently measure wind loads which are significantly lower than required by 6.5 due to the shape of the building, shielding in excess of that implied by exposure categories, and necessary conservatism in enveloping load coefficients in 6.5. In some cases, adjacent structures may shield the structure sufficiently that removal of one or two structures could significantly increase wind loads Additional wind tunnel testing without specific nearby buildings (or with additional buildings if they might cause increased loads through channeling or buffeting) is an effective method for determining the influence of adjacent buildings. It would be prudent for the designer to test any known conditions that change the test results and apply good engineering judgement in interpreting the test results. Discussion between the owner, designer and wind-tunnel laboratory can be an important part of this decision. However, it is impossible to anticipate all possible changes to the surrounding environment that could significantly impact pressure for the main wind force resisting system and for cladding pressures Also, additional testing may not be cost effective Suggestions, written in mandatory language for users (e.g., code writers) desiring to place a lower limit on the results of wind tunnel testing are shown below:

**Lower limit on pressures for main wind force resisting system.** Forces and pressures determined by wind tunnel testing shall be limited to not less than 80 percent of the design forces and pressures which would be obtained in 6.5 for the structure unless specific testing is performed to show that it is the aerodynamic coefficient of the building itself, rather than shielding from nearby structures, that is responsible for the lower values. The 80 percent limit may be adjusted by the ratio of the frame load at critical wind directions as determined from wind tunnel testing without specific adjacent buildings (but including appropriate upwind roughness), to that determined by 6.5.

**Lower limit on pressures for components and cladding.** The design pressures for components and cladding on walls or roofs shall be selected as the greater of the wind tunnel test results or 80 percent of the pressure obtained for Zone 4 for walls and Zone 1 for roofs as determined in 6.5, unless specific testing is performed to show that it is the aerodynamic coefficient of the building itself, rather than shielding from nearby structures, that is responsible for the lower values Alternatively, limited tests at a few wind directions without specific adjacent buildings, but in the presence of an appropriate upwind roughness, may be used to demonstrate that the lower pressures are due to the shape of the building and not to shielding

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Ambient Air Density Values for Various Altitudes

Table C6-1

| Altitude | | Ambient Air Density | | | | | |
|---|---|---|---|---|---|---|---|
| Feet | Meters | Minimum ($lbm/ft^3$) | Minimum ($kg/m^3$) | Average ($lbm/ft^3$) | Average ($kg/m^3$) | Maximum ($lbm/ft^3$) | Maximum ($kg/m^3$) |
| 0 | 0 | 0.0712 | 1.1392 | 0 0765 | 1 2240 | 0.0822 | 1.3152 |
| 1000 | 305 | 0.0693 | 1.1088 | 0.0742 | 1.1872 | 0.0795 | 1.2720 |
| 2000 | 610 | 0.0675 | 1.0800 | 0.0720 | 1.1520 | 0 0768 | 1.2288 |
| 3000 | 914 | 0.0657 | 1.0512 | 0.0699 | 1.1184 | 0.0743 | 1.1888 |
| 3281 | 1000 | 0.0652 | 1.0432 | 0.0693 | 1.1088 | 0.0736 | 1.1776 |
| 4000 | 1219 | 0.0640 | 1.0240 | 0.0678 | 1.0848 | 0.0718 | 1.1488 |
| 5000 | 1524 | 0.0624 | 0.9984 | 0 0659 | 1.0544 | 0 0695 | 1.1120 |
| 6000 | 1829 | 0.0608 | 0.9728 | 0.0639 | 1.0224 | 0.0672 | 1.0752 |
| 6562 | 2000 | 0.0599 | 0.9584 | 0.0629 | 1 0064 | 0 0660 | 1.0560 |
| 7000 | 2134 | 0.0592 | 0.9472 | 0.0620 | 0.9920 | 0 0650 | 1.0400 |
| 8000 | 2438 | 0.0577 | 0.9232 | 0.0602 | 0.9632 | 0.0628 | 1.0048 |
| 9000 | 2743 | 0.0561 | 0.8976 | 0.0584 | 0.9344 | 0.0607 | 0.9712 |
| 9843 | 3000 | 0.0549 | 0.8784 | 0.0569 | 0.9104 | 0.0591 | 0.9456 |
| 10,000 | 3048 | 0.0547 | 0.8752 | 0.0567 | 0.9072 | 0.0588 | 0.9408 |

Probability of Exceeding Design Wind Speed During Reference Period

Table C6-2

| Annual Probability $P_a$ | Reference (Exposure) Period, $n$ (years) | | | | | |
|---|---|---|---|---|---|---|
| | 1 | 5 | 10 | 25 | 50 | 100 |
| 0.04 (1/25) | 0.04 | 0.18 | 0 34 | 0 64 | 0.87 | 0.98 |
| 0.02 (1/50) | 0.02 | 0 10 | 0 18 | 0.40 | 0 64 | 0.87 |
| 0.01 (1/100) | 0 01 | 0.05 | 0.10 | 0.22 | 0.40 | 0.64 |
| 0.005 (1/200) | 0 005 | 0.02 | 0.05 | 0 10 | 0.22 | 0.39 |

Conversion Factors for Other Mean Recurrence Intervals

Table C6-3

| MRI (years) | Peak gust wind speed, V (mph) m/s | | |
|---|---|---|---|
| | Continental U.S. | | Alaska |
| | V = 85-100 (38-45 m/s) | V > 100 (hurricane) (44.7 m/s) | |
| 500 | 1.23 | 1.23 | 1 18 |
| 200 | 1.14 | 1.14 | 1.12 |
| 100 | 1.07 | 1.07 | 1.06 |
| 50 | 1.00 | 1.00 | 1 00 |
| 25 | 0.93 | 0.88 | 0.94 |
| 10 | 0.84 | 0.74 (76 mph min.) (33.9 m/s) | 0.87 |
| 5 | 0.78 | 0.66 (70 mph min.) (31.3 m/s) | 0.81 |

Note: Conversion factors for the column "V > 100 (hurricane)" are approximate. For the MRI = 50 as shown, the actual return period, as represented by the design wind speed map in Fig. 6-1, varies from 50 to approximately 90 years. For an MRI = 500, the conversion factor is theoretically "exact" as shown.

## Gust Effect Factor - Example

### Table C6-4

| Values Obtained From Table 6-4 | |
|---|---|
| $z_{min}$ | 60 ft |
| $\bar{\epsilon}$ | 0.5 |
| $c$ | 0.45 |
| $\bar{b}$ | 0.3 |
| $\bar{\alpha}$ | 0.33 |
| $\hat{b}$ | 0.64 |
| $\hat{\alpha}$ | 0.2 |
| $\ell$ | 180 |
| $C_{fx}$ | 1.3 |
| $\xi$ | 1 |
| Height (h) | 600 ft |
| Base (B) | 100 ft |
| Depth (L) | 100 ft |