Applied Wind Load Design for Rooftop, Wall-mounted, and Freestanding Structures - ASCE 7-22

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Introduction

This course guides practicing structural engineers through wind load determination for building appurtenances and other structures using the detailed provisions of ASCE 7-22.

Examples include direct references to chapters, sections, tables, and figures, with step-by-step calculations illustrating limit state design pressures for strength design applications. Where allowable stress design is used, adjustments are noted according to ASCE 7-22 load factors.

Designed as a practical, desk-side reference, this course helps engineers apply code provisions confidently, avoid common mistakes, and handle rooftop, wall-mounted, and freestanding elements that often govern project decisions.

Rooftop Appurtenances

Part 1: Wind Load Evaluation for Rooftop Elements in Accordance with ASCE 7-22

Rooftop-mounted appurtenances have been repeatedly identified as critical initiation points for wind-induced damage in low-rise and mid-rise buildings (Fig.1). Post-event reconnaissance reports issued by FEMA, ASCE, and state emergency management agencies following recent hurricanes and severe straight-line wind events consistently document a common failure sequence. Localized failure of rooftop equipment or attachments frequently precedes roof membrane uplift, progressive deck distress, loss of enclosure integrity, and, in some cases, secondary structural damage. These observations indicate that rooftop appurtenance failures are seldom governed by insufficient material strength of the component itself. Instead, they are most often attributed to unconservative wind pressure assumptions, incomplete consideration of internal pressure effects, or discontinuities in the load path between the appurtenance and the primary structural system.

Fig. 1: Typical Rooftop Appurtenances Subjected to Wind Loading per ASCE 7-22

ASCE 7-22 explicitly addresses this vulnerability by classifying rooftop appurtenances as components and cladding subject to localized wind pressures that commonly exceed those used for the main wind-force-resisting system. The code intent is clear: rooftop-mounted elements must be evaluated independently using pressure coefficients that reflect local flow behavior at roof edges and corners, rather than relying on global roof pressures or gravity-based attachment assumptions.

Minimum design wind pressures shall not be taken less than those specified in ASCE 7-22 §30.1.4. Risk Category selection directly influences basic wind speed per Figures 26.5-1A through 26.5-1C and must be verified at project initiation.
Where rooftop elements contribute to the Main Wind-Force-Resisting System (MWFRS), evaluation shall follow ASCE 7-22 Chapter 27 rather than Chapter 30. Misclassification between MWFRS and Components & Cladding remains a frequent source of review comments in permit submissions.

Enclosure Classification and Internal Pressure Effects

The wind load evaluation process for rooftop elements begins with enclosure classification, as defined in ASCE 7-22 Chapter 26. Enclosure classification directly influences internal pressure coefficients and therefore governs the net pressure acting normal to rooftop-mounted components. In practical design and forensic review, a significant proportion of commercial and industrial buildings qualify as partially enclosed due to the presence of large overhead doors, loading dock openings, glazing failures, or extensive louvered wall areas on windward elevations.

When a building behaves as partially enclosed, positive internal pressurization may develop under windward exposure. This internal pressure acts simultaneously with external suction on rooftop components, producing amplified net uplift demands. Field investigations have repeatedly shown that rooftop units located near roof edges or corners experience the most severe demand combinations when partial enclosure coincides with localized suction effects. Failure to properly account for internal pressure has been identified as a recurring deficiency in both original designs and post-construction evaluations.

Roof Zones, Exposure, and Velocity Pressure

Wind pressures acting on rooftop appurtenances are highly sensitive to roof zoning and exposure category. ASCE 7-22 Chapter 30 recognizes that flow separation and vortex formation near roof edges and corners generate substantially higher suction pressures than those acting on interior roof zones. These localized effects are particularly pronounced for low-rise structures, where the roof height is comparable to the atmospheric boundary layer thickness.

Exposure category further influences the magnitude of the velocity pressure evaluated at roof height. In accordance with ASCE 7-22 Chapter 26, the velocity pressure must be determined using the appropriate exposure coefficient, topographic factor, and directionality factor, and then carried consistently into the component and cladding provisions of Chapter 30. Inconsistent use of velocity pressure between Chapters 26 and 30 remains a common source of error in practice.

Worked Example: Uplift Demand on a Rooftop HVAC Unit

Consider a packaged rooftop HVAC unit installed on a low-rise commercial building with a mean roof height of 12 m. The structure is located in Exposure C and is classified as partially enclosed. The unit has a projected plan area of 7.2 m² and is installed within an edge roof zone.

The velocity pressure at roof height is evaluated in accordance with ASCE 7-22 §26.10 as

$q_h = 0.613\ Kz\ Kzt\ Kd\ V^2$

Using SI units throughout, and assuming a basic wind speed V = 42 m/s, the exposure coefficient at roof height is taken as Kz = 1.03. The topographic factor Kzt is taken as 1.0, and the directionality factor Kd is taken as 0.85. Substituting these values yields

$qz = 0.613 \times 1.03 \times 1.0 \times 0.85 \times 42^2 = 0.95$ kPa

When the basic wind speed V is expressed in meters per second, the coefficient 0.613 yields the velocity pressure directly in SI base units of N/m² (Pa). For practical structural design applications where pressures are carried forward in kN/m², the computed value may be converted by dividing by 1000, or alternatively the coefficient 0.000613 may be used to obtain velocity pressure directly in kN/m² without intermediate unit conversion.

For rooftop components and cladding located within an edge roof zone, ASCE 7-22 Chapter 30 specifies an external pressure coefficient of $GC_p = -2.8$. Given the partially enclosed classification, the applicable internal pressure coefficient is $GC_{pi} = +0.55$.

The resulting net design pressure acting normal to the rooftop unit is therefore:

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

$p = 1.00 \times (-2.8 - 0.55) = -3.35$ kPa

The corresponding uplift force acting on the unit is obtained by multiplying the net pressure by the projected plan area:

$F = |p|\ A$

$F = 3.35 \times 7.2 = 24.1$ kN

This uplift demand exceeds the self-weight of most packaged rooftop units of comparable size, indicating that anchorage design governs performance. Load combinations shall be applied in accordance with ASCE 7-22 Section 2.3 (LRFD) or Section 2.4 (ASD) [Corrected load combination reference] and Chapter 2. For allowable stress design, the governing combination is typically 1.0D + 1.0W.

The wind directionality factor Kd = 0.85 is applied in accordance with ASCE 7-22 §26.6. Zone-specific adjustments referenced in Chapter 30 modify pressure coefficients and do not replace the global directionality factor unless explicitly stated by the code.

Minimum Design Pressure Enforcement

ASCE 7-22 Section 30.1.4 establishes a mandatory lower bound for component and cladding wind pressures. The minimum design wind pressure for components and cladding (C&C) shall not be taken less than 16 psf (0.77 kPa). The calculated pressure obtained from the coefficient-based procedure shall not be taken less than this prescribed minimum value, regardless of exposure category, roof height, or zone location. In practice, this provision becomes critical in low-rise structures located in Exposure B, where computed pressures may fall below the minimum threshold, particularly within interior roof zones.

The professional obligation does not end at computing pressure coefficients and velocity pressure. The engineer shall explicitly verify that the resulting design pressure satisfies the minimum requirement of the code before converting it into anchor forces or connection demands. Failure to enforce minimum pressure provisions is frequently observed in peer reviews and post-event forensic evaluations, especially for rooftop mechanical units assumed to be located in sheltered regions.

Where the calculated pressure magnitude is less than the minimum required by ASCE 7-22 §30.1.4, the minimum pressure shall govern the design. This governing pressure shall then be carried forward through load combinations and anchorage verification in accordance with Chapter 2 of ASCE 7-22 and ACI 318-22 Chapter 17.

Professional Consideration:

Minimum pressure enforcement is not a conservatism adjustment; it is a mandatory code boundary condition intended to prevent underestimation of localized wind effects on rooftop components.

Equipment Screens and Vertical Rooftop Elements

Vertical rooftop elements, such as equipment screens and parapet-mounted barriers, differ fundamentally from rooftop units due to their orientation and exposure to lateral drag forces. When installed near roof edges, these elements are subjected to amplified pressures caused by accelerated flow and local vortex shedding. Post-event damage surveys frequently show that failures initiate at the screen-to-roof connection rather than within the screen framing itself.

This observed behavior highlights the importance of verifying the complete load path from the screen through its anchorage and into the supporting roof diaphragm or structural framing. Designs that focus exclusively on member capacity without explicit consideration of load transfer mechanisms have repeatedly been shown to be vulnerable under extreme wind events.

Worked Example — Flush-Mounted Solar Panels Adjacent to a Roof Edge

Consider a flush-mounted solar panel array installed adjacent to a parapet within an edge roof zone. Although the panels have a low profile, ASCE 7-22 Chapter 30 indicates that localized suction pressures may govern attachment design.

Using the previously evaluated velocity pressure of $q_h = 1.20$ kPa, the external pressure coefficient for low-profile rooftop components is taken as $GC_p = -1.9$. The internal pressure coefficient remains $GC_{pi} = +0.55$

The resulting net uplift pressure is

p = qh (GCp − GCpi) = 1.20 (−1.9 − 0.55) = −2.94 kPa

Implementation Note: This example uses GC_p and GC_{pi}, which already include the gust-effect factor G in accordance with ASCE 7-22. Therefore, no separate gust factor is applied in this load case. Where Cp and Cpi are used instead, G shall be applied explicitly. The two formulations shall not be mixed within a single load evaluation.

$$p = q_h\left(GC_p - GC_{pi}\right) = 1.20\,(-1.9 - 0.55) = -2.94 \text{ kPa}$$

For a tributary panel area of 1.6 m², the uplift force acting on an individual panel is

F = |p| A = 2.94 × 1.6 = 4.7 kN

In typical installations, this force governs fastener spacing, edge distance requirements, and embedment depth into the supporting roof structure rather than panel flexural capacity. Load combinations shall again be applied in accordance with ASCE 7-22 Section 2.3 (LRFD) or Section 2.4 (ASD) [Corrected load combination reference].

Peer reviews and post-event investigations consistently reveal recurring deficiencies in rooftop appurtenance design, including neglect of internal pressure effects, misapplication of interior roof pressure coefficients to edge and corner zones, and reliance on gravity-based attachment without verification of uplift resistance. These issues, while not always apparent under service-level conditions, can become critical under extreme wind events. Governing design failures are seldom material-related; they most often stem from inaccurate pressure selection, misclassification of enclosure types, and incomplete anchorage design. Consequently, evaluation of wind pressures and verification of load paths for rooftop-mounted elements should be treated as essential primary design tasks rather than secondary checks, ensuring both safety and compliance with ASCE 7-22 and related standards. Systematic application of these principles supports reliable structural performance and reflects practical engineering judgment in design, peer review, and post-construction verification.

Part 2: Wall-Mounted and Attached Appurtenances

Practical Wind Load Design in Accordance with ASCE 7-22

Wall-mounted appurtenances, including louvers, exterior ducts, architectural sunshades, mechanical supports, and attached canopies (Fig. 2) , are frequently treated as secondary elements relative to the primary structural frame. However, post-event investigations and plan review experience consistently demonstrate that these components often govern localized wind performance of the building envelope. Their failures rarely originate from global structural instability; rather, they stem from underestimated corner pressures, improper classification under ASCE 7-22, or inadequate anchorage design in accordance with ACI 318-22. This module presents a unified, code-compliant methodology for evaluating wind loads on wall-mounted and attached appurtenances using the International System of Units (SI), with direct reference to ASCE 7-22 Chapters 26 through 30 and ACI 318-22 Chapter 17.

Fig. 2 — Wall-Mounted and Attached Appurtenances Subjected to Components & Cladding Wind Pressures (ASCE 7-22)

Applicability Boundary — Component and Cladding versus MWFRS Behavior

Wall-mounted appurtenances are typically evaluated under Chapter 30 of ASCE 7-22 as components and cladding elements. However, classification is not determined solely by geometry or attachment method. The controlling criterion is structural behavior within the global load path.

If the failure of a wall-mounted element does not alter the stability of the primary structural system, the element may be treated as a component and cladding element under Chapter 30. Conversely, if the support assembly, bracket system, or framing configuration participates in the transfer of lateral forces to the main wind-force-resisting system, evaluation under Chapter 27 may become necessary.

This distinction frequently arises in canopy brackets, large mechanical support frames, and architectural projections rigidly connected to structural diaphragms. In such cases, the engineer shall examine the load path continuity from the element into the primary structural system. Where ambiguity exists, conservative evaluation under both frameworks may be warranted.

Design review experience shows that misclassification between Chapter 27 and Chapter 30 continues to represent a frequent procedural deficiency in wind design of attached elements.

Classification is a behavioral decision grounded in load path mechanics, not a labeling exercise based on appearance or architectural description.

Wind Pressure Determination Framework

Wind load determination begins with proper classification of the element as a **Component and Cladding (C&C)** element under **ASCE 7-22 Chapter 30**. The velocity pressure at height z is defined in Section 26.10 as:

$q_z = 0.613\ K_z \times K_{zt} \times K_d \times V^2$ N/m²

where

- ✓ q_z = velocity pressure at height z (N/m² or Pa)
- ✓ V= basic wind speed (m/s)
- ✓ K_z = exposure coefficient
- ✓ K_{zt} = topographic factor
- ✓ K_d = wind directionality factor

The constant **0.613** converts wind speed squared to pressure in SI units. When V is expressed in **m/s**, the equation produces q_z in **N/m² (Pa)**. For convenience in structural design calculations, the result may be divided by **1000** to express the pressure in **kN/m² (kPa)**. Alternatively, the coefficient **0.000613** may be used directly to compute q_z in **kN/m²**.

This formulation is applied once within the velocity pressure expression, and the directionality factor Kd shall not be reintroduced in subsequent pressure calculations to avoid unintended load amplification.

For components and cladding, the design wind pressure is computed using gust-effect adjusted pressure coefficients (GCp) as prescribed in ASCE 7-22 Section 30.3:
p = qz (GCp) - qi (GCpi)

In this formulation, the pressure coefficients GC_p and GC_{pi} are defined to include the gust-effect factor G as incorporated within the ASCE 7-22 provisions. Accordingly, application of an additional gust factor is neither required nor permitted, since it would artificially increase the resulting design pressure. When wind effects are instead evaluated using the C_p and C_{pi} coefficient format, the gust-effect factor must be introduced explicitly in accordance with the applicable code equations. Each coefficient format represents a self-contained analytical framework, and the two approaches shall remain separate within any single wind load evaluation to preserve methodological consistency.

Implementation Note: Internal pressure shall be evaluated using qh at mean roof height per ASCE 7-22 §26.11. In this course, qi is defined equal to qh; the symbols represent the same physical quantity and shall not differ.
The internal velocity pressure qi is determined at the mean roof height per Section 26.11. Internal pressure coefficients GCpi are selected based on enclosure classification in Section 26.2.
Technical Interpretation: This equation uses GCp and GCpi, which already include the gust-effect factor G. The gust factor shall not be applied separately in this load case. Where Cp and Cpi are used instead, G must be applied explicitly. The two formulations shall not be mixed within a single load evaluation.

Internal pressure is evaluated at the mean roof height in accordance with ASCE 7-22 §26.11, using qh as the reference velocity pressure. Within this course, the symbol qi is taken equal to qh, as both denote the same physical quantity at mean roof elevation; no distinction between the two is intended in the analytical development presented herein.

Both positive and negative internal pressure cases shall be evaluated, and the governing pressure magnitude shall control design.

The most unfavorable pressure effect shall govern both component strength and anchorage design.

Edge and corner zones shall be evaluated using the appropriate figures in Chapter 30, recognizing that corner regions (Zones 4 and 5) frequently control design.

Worked Design Example — Corner Louver

Code Correction: If the structure is partially enclosed, GCpi shall be ±0.55 per ASCE 7-22 §26.13. If GCpi = ±0.18 is used, the structure must be classified as enclosed. Internal pressure shall be evaluated as $p_i = q_h(GC_{pi})$.

Consider a wall-mounted aluminum louver installed within a corner zone of a building located in Exposure C terrain. The following parameters are assumed for illustration consistent with ASCE 7-22:
Given Parameters: Basic wind speed V = 40 m/s; Kz = 1.0; Kzt = 1.0; Kd = 0.85; GCp = 1.2 (corner zone); GCpi = ±0.55 (partially enclosed) [Corrected per ASCE 7-22 §26.13]; Projected area A = 2.0 m²; two anchor bolts.

The present formulation is expressed in terms of GC_p and GC_pi, which inherently incorporate the gust-effect factor G in accordance with ASCE 7-22. Consequently, no additional gust factor is introduced in this load case, as doing so would result in unintended amplification of the calculated wind effects. When the alternative formulation based on C_p and C_{pi} is adopted, the gust-effect factor shall instead be applied explicitly as required by the governing provisions. These two coefficient frameworks represent distinct analytical approaches and shall not be combined within a single load evaluation to maintain internal consistency of the wind load determination.

Velocity pressure is calculated as:
qz = 0.613 × 1.0 × 1.0 × 0.85 × (40)^2 = 0.834 kN/m²

Internal pressure shall be calculated using qh evaluated at mean roof height in accordance with ASCE 7-22 §26.11. The internal pressure term is qh (GCpi), where GCpi = ±0.55 for partially enclosed buildings per §26.13.

Technical Interpretation: This example follows the GCp/GCpi formulation introduced above; therefore, no separate gust factor is applied.

Net design pressure becomes:
p = 0.834(1.2) − 0.459(0.55) = 0.917 kN/m²
Resultant force on the louver:
F = pA = 0.917 × 2.0 = 1.83 kN
Assuming equal load distribution to two anchors:
T = F / 2 = 0.915 kN per anchor

The resulting anchor tension shall be checked against the design tensile strength ϕNn determined in accordance with ACI 318-22 Chapter 17.

For eccentric wall-mounted brackets, anchor forces shall be distributed using elastic anchor group analysis. Combined tension and shear interaction shall comply explicitly with ACI 318-22 §17.6. Failure to include eccentricity amplification is a recurring deficiency in forensic investigations.

Anchorage Design per ACI 318-22

Anchor design shall comply with ACI 318-22 Chapter 17. The nominal tensile strength shall be evaluated considering steel strength (Nsa), concrete breakout strength (Ncb), pullout strength (Np), and side-face blowout strength (Nsb) where applicable. Edge distance reductions and modification factors for cracked concrete shall be explicitly included. Wind load combinations shall follow ASCE 7-22 Chapter 2.

Typical governing LRFD wind load combinations include:

1.2D + 1.0W + L + 0.5S
0.9D + 1.0W

Applicable load combinations shall be selected in accordance with ASCE 7-22 Section 2.3 for LRFD or Section 2.4 for ASD, as applicable.

Design Integration and Field Application

In practice, failures of wall-mounted components frequently originate from corner-zone pressure amplification combined with inadequate edge distances in concrete anchorage. Free-body diagrams are recommended to verify eccentric load paths, especially for projecting canopies where uplift governs. Verification of tributary area assumptions and proper application of GCp values are critical during both design and plan review.
Professional Consideration: In design office reviews and forensic evaluations, underestimation of corner suction combined with omission of concrete breakout checks under ACI 318-22 remains a recurring deficiency. Conservative zoning assumptions and explicit anchorage verification significantly reduce retrofit risk and liability exposure.

Part 3: Freestanding and Other Structures (Non-Building)

Freestanding structures such as billboard signs on poles, ground-mounted solar arrays, and standalone canopies present unique challenges in wind load evaluation (Fig. 3). Unlike building-attached elements, these structures lack the stabilizing influence of a primary building frame, making them highly sensitive to localized wind forces, exposure conditions, and dynamic effects. Misapplication of force coefficients or improper evaluation of projected areas can result in underestimation of design loads, leading to structural failures or excessive serviceability deflections. A rigorous, step-by-step approach ensures compliance with ASCE 7-22 Chapter 29, providing both safety and efficiency in design and review.

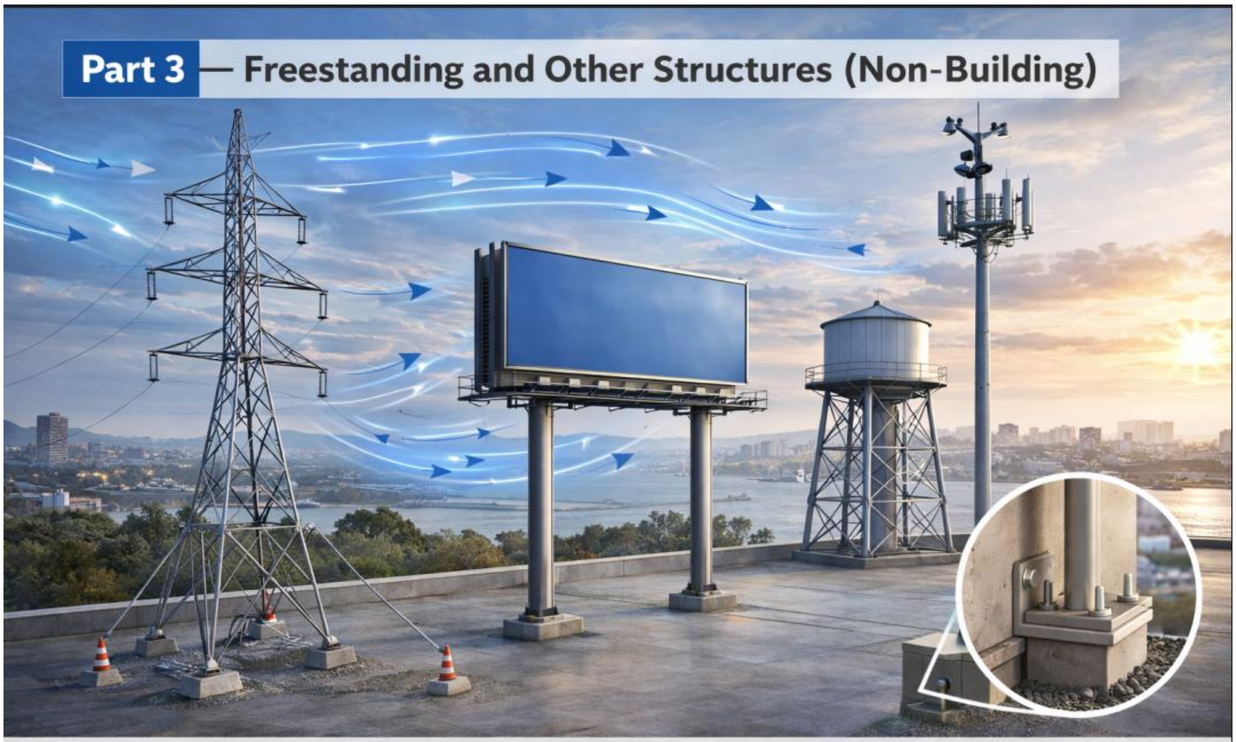

Fig. 3- Freestanding and Other Non-Building Structures Subjected to Wind Loading Per ASCE 7-22

The force coefficients provided in ASCE 7-22 Chapter 29 are formulated to include the effects of gust response within their definition. Accordingly, no additional gust-effect factor shall be applied when these coefficients are used in pressure or force calculations, as doing so would result in double counting of aerodynamic amplification.

Classification, Exposure, and Velocity Pressure

The initial step in assessing wind loads on freestanding structures is proper classification according to ASCE 7-22 Section 29.3, distinguishing between monopole-supported, frame-supported, and fully cantilevered configurations. The velocity pressure at the height of the structure is computed using:
$q_z = 0.613\, K_z\, K_{zt}\, K_d\, V^2$

When the basic wind speed V is expressed in meters per second, the constant 0.613 converts the squared wind speed directly into velocity pressure in SI base units of N/m² (Pa). For consistency with structural design calculations typically performed in kN/m², the resulting value may be divided by 1000, or alternatively the coefficient 0.000613 may be used to obtain velocity pressure directly in kN/m² without requiring an intermediate unit conversion step.

Where:

- q_z = velocity pressure at elevation [kN/m²]
- K_z= exposure coefficient at height zzz
- K_{zt}= topographic factor
- K_d= wind directionality factor
- V = basic wind speed [m/s]

Exposure categories (B, C, D) and topographic features such as ridges or escarpments significantly affect KzK and Kzt. Accurate classification ensures the velocity pressure reflects realistic site conditions and directional wind probabilities.

Force Coefficients and Projected Area

Non-building structures require careful selection of external force coefficients CfC_fCf based on element shape, orientation, and shielding. For monopole-supported billboards, ASCE 7-22 Table 29.3-1 provides guidance on CfC_fCf values for rectangular panels at mid-span, edges, and corners. The resultant wind force is then determined as:
$F = q_z \times C_f \times A$

Where:

- F = design wind force [kN]
- C_f = external force coefficient
- A = projected area normal to the wind [m²]

For structures with complex geometries, such as solar arrays with tilted panels, the net projected area must account for overlap, panel spacing, and aerodynamic shielding. Misestimation can lead to substantial under-design, particularly in edge and corner zones where pressure concentrations occur.

Important Code Note

Force coefficients in ASCE 7-22 Chapter 29 inherently include gust effects; therefore, a separate gust-effect factor G shall NOT be applied when using Chapter 29 force coefficients.

This is in contrast to wind pressure calculations for building components and cladding in ASCE 7-22 Chapters 26 and 30, where the gust-effect factor is explicitly included.

ASCE 7-22 Table 29.5-1 specifies force coefficients for circular members as a function of Reynolds number and surface roughness. Typical values range from $C_f \approx 0.6$ to 1.2. A value of $C_f = 0.7$ is representative for smooth circular steel poles with Reynolds numbers on the order of 10^5.

Worked Example: Wind Load on Billboard Sign Mounted on Flexible Pole (ASCE 7-22)

This example addresses the determination of design wind loads for a tall solid freestanding billboard sign mounted at an elevated height above grade, a configuration commonly encountered in highway and commercial corridor projects. The example demonstrates the application of ASCE 7-22 Chapter 29 provisions for freestanding signs mounted on flexible poles, where gust effects are inherently incorporated in the force coefficients.

Specifically, wind loads on the sign panel are determined in accordance with ASCE 7-22 Section 29.3, while wind loads on the supporting pipe column are evaluated per the requirements of Section 29.4. The geometric configuration of the billboard is illustrated in Figure 1, and project-specific parameters are summarized in Table 1.

1. Problem Description
A freestanding billboard sign is mounted on a single flexible steel pole. Wind loads are calculated according to ASCE 7-22, considering wind forces on both the billboard panel and the supporting pole. (Fig. 4)

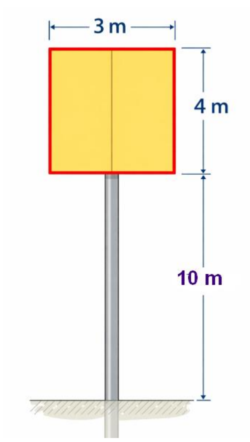

Fig. 4 – Geometry and dimensions of freestanding billboard sign

The freestanding billboard has a total height of 14 m measured from ground level to the top of the sign. The wind pressure is evaluated at a reference height z = 12 m, corresponding to the centroid of the exposed sign area, in accordance with ASCE 7-22.

2. Assumed Data:

Table 1- Data for Billboard Sign on Flexible Poles

Parameter	Value
Risk Category	II
Basic Wind Speed, V	42 m/s
Terrain Category	C
Directionality Factor, Kd	0.85
Topographic Factor, Kzt	1.0
Billboard Size	4.0 m × 3.0 m
Billboard Area	12 m²
Height to Center of Sign	12 m
Pole Height	14 m
Pole Diameter	0.6m

3. Velocity Pressure Calculation

According to ASCE 7-22 Eq. 26.10-1:
$q_z = 0.613\ K_z\ K_{zt}\ K_d\ V^2$
At $z = 12$ m, Exposure C → $K_z = 0.92$ (Table 26.10-1)
$q_z = 0.613 \times 0.92 \times 1.0 \times 0.85 \times 42^2 = 0.84$ kN/m²

4. Wind Force on Billboard Sign

Force coefficient for a solid freestanding rectangular sign (ASCE 7-22 Table 29.4-1): $C_f = 1.8$ (Note: Force coefficients in Chapter 29 inherently include gust effects; therefore, no additional gust factor is applied.)

Note that ASCE 7-22 Chapter 29 force coefficients include gust effects.

$F_{sign} = q_z \times C_f \times A = 0.84 \times 1.8 \times 12 = 18.1$ kN

5. Wind Force on Supporting Pole

Projected area of pole: $A = D \times H = 0.6 \times 14 = 8.4$ m²
Force coefficient for circular members (ASCE 7-22 Table 29.5-1): $C_f = 0.7$
For simplicity, velocity pressure evaluated at mid-height is used for pole loading as an engineering approximation. Where required, ASCE 7-22 permits evaluation of velocity pressure variation with height for increased precision.

Force coefficient for circular pole ($D = 0.6$ m):
$C_f = 0.7$ (smooth surface, $Re \approx 10^5$, ASCE 7-22 Table 29.5-1)

$F_{pole} = q_z \times C_f \times A = 0.84 \times 0.7 \times 8.4 = 5$ kN

6. Overturning Moment at Base

Moment due to sign load:
Msign = 18.1 × 12 = 217.2 kN·m

Moment due to pole wind load (uniform distribution):
Mpole = Fpole × H/2 = 5 × (14/2) = 35 kN·m

Total Base Moment = 252.2 kN·m

7. Structural Behavior Notes

The pole is considered a flexible cantilever structure. While ASCE 7 does not explicitly define flexibility criteria based on H/D ratio, a value of H/D > 20 is commonly used in industry practice to indicate flexible behavior for monopoles and lighting structures.

8. Slenderness Check

H/D = 23.3 > 20 → Pole classified as flexible.
For slender monopoles exceeding common H/D thresholds, designers shall consider susceptibility to vortex shedding and dynamic amplification effects in accordance with ASCE 7 Commentary and applicable FHWA technical advisories. While not always governing, omission of dynamic evaluation has contributed to fatigue-related failures in practice.

ASCE 7 does not prescribe a specific flexibility criterion based on H/D ratio. However, industry practice and ASCE 7 Commentary commonly classify monopoles with H/D > 20 as flexible for dynamic response considerations.

9. Deflection Due to Wind Load on Sign

The sign wind load is conservatively modeled as a concentrated horizontal force applied at the top of the cantilever.

The wind load acting on the sign panel is modeled as a concentrated horizontal load applied at the top of the cantilever for conservative estimation. The lateral deflection is calculated using the standard cantilever beam equation:

$\delta sign = Fsign\ H^3 / (3EI)$

$I = \pi \times D4/ 64 = 0.00636$ m4

Assuming solid circular steel pole for simplified illustration, E = 200 GPa, , the resulting deflection is:
$\delta sign \approx 15$ mm

10. Deflection Due to Wind Load on Pole

Wind pressure on the pole is modeled as a uniformly distributed horizontal load along the entire height of the cantilever. The equivalent load per unit height is:

Uniformly distributed load along pole height:
w = Fpole / H = 5 / 14 = 0.36 kN/m

δpole = w H4 / (8EI)
δpole ≈ 4 mm

11.Total Lateral Deflection

Assuming linear elastic behavior, total lateral deflection is obtained using the principle of superposition:

δtotal = δsign + δpole = 15 + 4 = 19 mm

12. Allowable Deflection Criterion

ASCE 7 does not explicitly define deflection limits for freestanding sign structures. Therefore, an allowable deflection limit of H/100 is adopted based on common industry practice for slender cantilever poles:
δallow = H / 100 = 14 / 100 = 0.14 m = 140 mm

Code & Practice Background

ASCE 7 does not explicitly specify deflection limits for billboard or sign structures.
Therefore, allowable deflection is selected based on:
Industry practice for slender cantilever poles
Serviceability and visual performance criteria

Commonly Adopted Limits

Structural Type	**Typical Limit**
Lighting /Sign Poles	H/100 to H/150
Monopoles	H/75 to H/100

13. Serviceability Check

$\delta_{total} = 19$ mm < $\delta_{allow} = 140$ mm → OK

The calculated total lateral deflection of 19 mm is significantly less than the allowable deflection of 140 mm. Therefore, the pole satisfies serviceability requirements under wind loading.

Dynamic Amplification and Vortex Shedding Considerations

Freestanding structures such as monopoles, sign supports, and slender cantilevered elements may experience wind-induced dynamic effects that exceed the demand predicted by static pressure calculations alone. Although ASCE 7-22 Chapter 29 provides force coefficients suitable for many non-building structures, the engineer shall evaluate whether dynamic amplification, along-wind response, or vortex shedding susceptibility is relevant to the structural configuration.

For slender elements with significant height-to-width ratios, oscillatory behavior may govern serviceability or fatigue performance even when static strength requirements are satisfied. ASCE 7 Commentary and FHWA guidance for sign structures and traffic support systems emphasize the importance of recognizing dynamic sensitivity, particularly for steel monopoles and tubular supports.

Where structural frequency, damping characteristics, or slenderness suggest potential amplification, supplemental evaluation may be required to ensure adequate performance under wind excitation. The absence of visible distress under static design conditions does not preclude dynamic vulnerability.

Field Observation:

post-storm investigations frequently attribute freestanding structure failures to overlooked dynamic behavior rather than insufficient static design capacity.

Wind loads were evaluated in accordance with ASCE 7-22 Chapters 26 and 29. Force coefficients from ASCE 7 inherently account for gust effects; therefore, no separate gust factor was applied. Structural deflections were calculated using linear elastic cantilever beam theory and evaluated against industry-accepted serviceability limits. The structure satisfies both strength and serviceability requirements under the governing wind load.

In practice, designers must account for interactions between projected area, orientation, and aerodynamic shielding. Arrays of billboards may experience amplified pressures due to channeling effects, while ground-mounted solar panels require careful evaluation of inter-row spacing to avoid underestimation of suction forces. For freestanding canopies, edge and corner uplift often governs anchor design more than mid-span forces. Dynamic effects, including vortex shedding and gust oscillations, introduce fatigue considerations for monopoles and cantilevered supports. Verification of anchor design using tributary area analysis, incorporation of soil-structure interaction for pole foundations, and consideration of simultaneous uplift and overturning moments are critical. Early-stage visualization of load paths and free-body diagrams enhances comprehension of complex force interactions, facilitating safer and more efficient structural solutions.

This integrated, narrative-based assessment ensures that engineers account for velocity pressure, force coefficients, projected areas, and anchorage simultaneously, guaranteeing ASCE 7-22 compliance, minimizing structural failures under extreme wind events, and aligning with professional judgment developed through field review and post-event analysis. Reference standards include ASCE 7-22, ACI 318-22, and ASTM E330 for wind load testing of structural panels, providing guidance for resilient and optimized design

Part 4: From Wind Pressure to Design Forces

This module establishes the direct link between calculated wind pressures and the design forces that govern anchorage, connections, and supporting members for appurtenances and non-building structures (Fig.5). In professional practice, errors rarely originate from pressure determination; they most often arise during translation of pressure into tributary forces, anchor demands, and load path resolution. ASCE 7-22 Section 26.12 formalizes this transition by defining how surface pressures are converted into applied loads, while Chapter 2 prescribes load combinations for both strength and allowable stress design. The objective of this section is to provide a consistent workflow that carries wind pressures through to anchor forces in a manner suitable for design offices, peer review, and field verification.

Fig. 5: Transition from Wind Pressure Analysis to Designing Structures for Code Conforming Wind Load Resistance (ASCE 7-22)

Pressure Formulation Framework

Once velocity pressure q_z has been established and appropriate gust-adjusted pressure coefficients selected, the governing design pressure acting on a component is expressed as:

p = qz (GCp) − qi (GCpi)

The present formulation is expressed in terms of GC_p and GC_{pi}, which by definition incorporate the gust-effect factor G in accordance with ASCE 7-22. No additional gust factor is therefore introduced in this load case. When the alternative representation based on C_p and C_{pi} is used, the gust-effect factor shall be applied explicitly as required by the governing provisions. These two coefficient formats are not interchangeable and shall not be combined within a single load evaluation, as doing so results in internal inconsistency in the calculated wind effects.

Internal pressure is evaluated at the mean roof height in accordance with ASCE 7-22 §26.11, using the velocity pressure qh as the governing reference quantity. Within the analytical framework adopted in this course, the internal velocity pressure qi is taken equal to qh, as both symbols represent the same physical parameter evaluated at mean roof elevation; no distinction between them is intended in subsequent calculations.

For elements governed by top and bottom surface interaction such as canopies and overhangs, the net pressure is expressed as:

pnet = qz (GCp,top − GCp,bottom) − qh (GCpi)

Implementation Note: This example follows the GCp/GCpi formulation; accordingly, the gust-effect factor is not applied separately.

Both positive and negative internal pressure coefficients shall be evaluated in accordance with ASCE 7-22 §26.13, and the governing pressure magnitude shall control design. When GC_p and GCpi are used, the gust-effect factor G is already included in these coefficients and shall not be applied separately.

From Pressure to Resultant Forces

Distributed surface pressures are converted into resultant forces by multiplying by the projected area normal to wind direction:

F = p · A

Service-level wind effects shall be distinguished from factored strength-level combinations to ensure deflection, rotation, and bearing performance are evaluated independently from ultimate resistance checks.

For anchor groups subjected to combined uplift and overturning, tension distribution shall be determined using elastic anchor group theory consistent with ACI 318-22 Chapter 17. Stability combinations such as 0.9D + 1.0W frequently govern foundation uplift and sliding resistance.

Load path verification shall extend from the applied surface pressure through anchors, supporting members, base plates, and into the foundation system. Each transfer interface shall be explicitly evaluated under governing load combinations.

Anchor-level forces are determined using tributary area concepts.

Once surface pressures have been translated into resultant forces, the engineering task extends beyond numerical conversion. The derived force must be traced through a mechanically consistent load path from the point of application to anchorage, supporting framing, and ultimately into the primary structural system. Wind design cannot be reduced to isolated force calculations; it requires verification that each interface within the structural assembly is capable of transferring the computed demand under the governing load combination of ASCE 7-22 Chapter 2.

Field investigations repeatedly demonstrate that wind-induced failures are rarely attributable to incorrect pressure coefficients. Instead, they arise from discontinuities in load transfer, insufficient anchorage detailing, overlooked eccentricities, or incomplete consideration of force accumulation where multiple components share common supports. Accordingly, the engineer shall confirm that tributary area assumptions, anchor force distribution, and connection detailing collectively reflect a continuous and reliable force transmission mechanism consistent with ACI 318-22 anchorage provisions.

Wind pressure calculation and structural load path verification form a single analytical sequence. One without the other does not constitute a complete design evaluation.

$T = |p| \cdot At$

At represents the tributary area assigned to each anchor. Tributary areas must reflect the actual load path and anchor spacing, not the gross component area. When multiple pressure components exist (uplift and lateral), each component shall be evaluated independently.

Worked Example — Rooftop Mechanical Unit (Partially Enclosed)

Consider a rooftop mechanical unit with projected area $A = 1.8\ m^2$ supported by four anchors. Basic wind speed $V = 42$ m/s, Exposure C, $Kz = 0.95$, $Kzt = 1.0$, $Kd = 0.85$. The gust-adjusted external pressure coefficient is $GCp = -1.30$. The structure is partially enclosed with $GCpi = +0.55$. For rooftop elevation, internal velocity pressure may conservatively be taken as $qi \approx qz$.

$qz = 0.613 \cdot 0.95 \cdot 1.0 \cdot 0.85 \cdot 42^2 = 0.873\ kN/m^2$

$p = qz\ (GCp - GCpi) = 0.873\ (-1.30 - 0.55) = -1.62\ kN/m^2$

This equation is formulated using GC_p and GC_{pi}, both of which inherently incorporate the gust-effect factor G within their definition. Accordingly, the gust factor shall not be applied separately in this load case, as doing so would artificially increase the calculated demand. Where the alternative formulation based on C_p and C_{pi} is adopted, the gust-effect factor must be introduced explicitly in accordance with the applicable provisions of ASCE 7-22. The two approaches represent distinct coefficient frameworks and shall not be intermingled within a single load evaluation, as such mixing compromises analytical consistency.

$T = |p| \cdot At = 1.62 \cdot 0.45 = 0.73$ kN per anchor

For strength design, the governing ASCE 7-22 load combination for uplift-dominated cases is 1.2D + 1.0W or 0.9D + 1.0W as applicable. Anchor design shall be verified in accordance with ACI 318-22 Chapter 17 including checks for steel strength, concrete breakout, pullout, pryout, and tension-shear interaction.

Load Combination Discipline — LRFD and ASD Application

Wind-induced pressures, once converted to discrete forces, shall be applied within the governing load combinations specified in ASCE 7-22 Chapter 2. For strength design, uplift-dominated conditions often govern under the combination:

$0.9D+1.0W$

where reduced dead load reflects destabilizing wind uplift. In contrast, allowable stress design applies wind effects under:

$D+W$

The selection of design methodology directly influences required anchor capacity and concrete breakout verification in accordance with ACI 318-22 Chapter 17. Identical wind pressures may produce materially different anchorage demands depending on whether LRFD or ASD is employed.

Engineering documentation shall clearly state the design basis used for anchorage and supporting members to avoid ambiguity during review, permitting, or construction.

Engineering Judgment:

Load combination discipline is as critical as pressure calculation accuracy; omission or misapplication of combinations can invalidate otherwise correct wind pressure derivations.

Worked Case Study — Freestanding Canopy Under Combined Actions

A single-column freestanding canopy with plan dimensions 6.0 m × 4.0 m and roof height 5.0 m is evaluated. The supporting column is eccentrically located 1.2 m from roof centroid. Basic wind speed V = 44 m/s, Exposure C, $K_z = 0.90$, $K_{zt} = 1.0$, $K_d = 0.85$. This example uses Cp–Cpi formulation with explicit gust factor G = 0.85 for clarity.

$q_z = 0.613 \cdot 0.90 \cdot 1.0 \cdot 0.85 \cdot 44^2 = 0.908$ kN/m²

$p = qz \cdot G \cdot (Cp - Cpi) = 0.908 \cdot 0.85\,(-1.30 - 0.55) = -1.43$ kN/m²

$F_{uplift} = |p| \cdot A = 1.43 \cdot 24 = 34.3$ kN

$\text{Torsion} = \text{Fuplift} \cdot e = 34.3 \cdot 1.2 = 41.2$ kN·m

$Tu = \text{Fuplift} / 4 = 8.6$ kN per anchor

Combined uplift, overturning, and torsion shall be evaluated simultaneously. Anchor groups shall be verified per ACI 318-22 §§17.4–17.6. Bearing stresses beneath base plates shall account for reduced compression area under net uplift conditions. Foundation stability shall include checks for overturning, sliding, uplift, and soil bearing in accordance with ACI and FHWA foundation guidance.

Engineering Judgment from Practice

Post-event structural evaluations repeatedly indicate that wind-related failures seldom originate from inadequate material strength; deficiencies arise from incomplete load paths, neglected torsion, underestimated tributary areas, or improper application of load combinations. Treating velocity pressure, pressure coefficients, projected area, anchorage, and foundation response as a continuous integrated system ensures compliance with ASCE 7-22 and ACI 318-22 while improving constructability and review efficiency.

Common Wind Design Deficiencies Observed in Plan Review and Forensic Investigations

Design review records and post-event forensic investigations across the United States consistently indicate that wind-related deficiencies in rooftop, wall-mounted, and freestanding structures are seldom the result of inadequate material strength or insufficient member sizing. Instead, these deficiencies typically originate from procedural shortcomings in wind load determination, element classification, and load path resolution. In many reviewed projects, the governing errors are not immediately apparent within isolated calculations, but emerge when wind pressures are traced through connections, anchors, and supporting systems under governing load combinations.

A recurring deficiency observed in plan review involves improper classification of appurtenances under the wind provisions of ASCE 7-22. Rooftop equipment frames, projecting canopies, and wall-mounted support assemblies are frequently treated as components and cladding without adequate evaluation of their structural behavior within the overall load path. Where these systems are rigidly connected to diaphragms or primary framing, their response may align more closely with main wind-force-resisting system behavior. Designs that default to Chapter 30 pressures in such cases often underestimate overturning effects and fail to apply appropriate load combinations, resulting in nonconservative demand on supporting members and foundations.

Incorrect enclosure classification represents another consistent source of underestimation in wind design. Buildings with large overhead doors, loading dock openings, or extensive louvered wall areas are frequently classified as enclosed based on nominal architectural intent rather than actual envelope behavior. Forensic investigations following wind events documented by agencies such as FEMA repeatedly demonstrate that partial enclosure conditions can develop rapidly once localized failures occur. When internal pressurization is not adequately considered, net uplift demands on rooftop components and attached elements may be significantly understated, particularly within edge and corner roof zones.

Misapplication of roof zoning and pressure coefficients further contributes to review rejections and observed damage. Components located near roof edges and corners are often designed using interior zone coefficients based on assumptions of shielding or parapet protection. Post-event reconnaissance and plan review commentary consistently show that these assumptions are unreliable. Localized flow separation and vortex effects dominate wind behavior in these regions, and failure to apply zone-specific coefficients in accordance with Chapter 30 frequently governs anchorage demand and connection performance.

Another deficiency observed in reviewed designs is inconsistent use of velocity pressure between code chapters. In several cases, velocity pressure is correctly evaluated at roof height per Chapter 26, yet internal pressure or component pressure calculations implicitly rely on a different reference height. This inconsistency, while subtle, can materially affect net design pressure when combined with partial enclosure effects. Review findings indicate that such discrepancies often stem from misunderstanding the relationship between qz, qh, and internal pressure terms, rather than from numerical error.

Anchorage-related deficiencies remain among the most critical findings in both plan review and forensic evaluation. Failures of rooftop units, equipment screens, and wall-mounted elements are frequently traced to incomplete verification of anchor behavior in accordance with ACI 318-22 Chapter 17. Designs commonly verify anchor steel strength while overlooking concrete breakout, edge distance reductions, or combined tension and shear interaction. In post-event evaluations, anchor failure modes are repeatedly found to govern performance even where the supported component remains structurally intact.

Eccentricity and torsional effects in anchor group behavior are also frequently neglected. Projecting canopies, offset equipment supports, and single-column freestanding roofs inherently introduce torsional demand under wind loading. Designs that distribute uplift forces uniformly among anchors without accounting for eccentricity consistently underestimate peak anchor forces. These errors may not be immediately evident during construction but become apparent during detailed review or after service-level distress is observed.

For freestanding structures, particularly slender monopoles and sign supports, dynamic sensitivity is often insufficiently addressed. While the static force coefficients provided in Chapter 29 of ASCE 7-22 are appropriate for many configurations, forensic evaluations of failed sign and support structures documented in FHWA guidance frequently reveal fatigue cracking and progressive deterioration associated with wind-induced vibration. Structures may satisfy static strength requirements while remaining vulnerable to serviceability or fatigue-related failure if dynamic behavior is not qualitatively assessed.

Across these investigations, a consistent pattern emerges. Wind-related failures and review rejections rarely result from a single incorrect coefficient or equation. They arise from a sequence of small procedural oversights that collectively compromise load path continuity. Effective wind design requires that pressure determination, classification, velocity pressure application, anchorage verification, and foundation response be treated as a single, continuous analytical process. When addressed systematically, these considerations improve both structural reliability and the efficiency of plan review and post-construction verification.

Engineering Judgment:

In professional practice, wind design deficiencies are predominantly procedural rather than analytical. Engineers who approach wind evaluation as a load-path-driven process—from pressure determination through anchorage and foundation response—consistently reduce review comments, retrofit exposure, and post-event performance issues.

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