Prestandard for Performance-Based **Wind** Design

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Executive Summary

Prestandard for Performance-Based Wind Design developed by the Structural Engineering Institute (SEI) of the American Society of Civil Engineers (ASCE) presents a recommended alternative to the prescriptive procedures for wind design of buildings contained in the nationally adopted standard *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7) and in the *International Building Code* (IBC). The intended audience for this document includes structural engineers, architects, building component and cladding specifiers/designers, and building officials engaged in the wind design and review of buildings. Properly implemented, this prestandard results in buildings that are capable of achieving the wind performance objectives specified by ASCE 7, and in many instances, superior performance to such objectives. Designers, peer reviewers, or AHJ who possess an understanding of wind engineering may adapt and modify these provisions to achieve higher wind performance objectives other than those specifically required by this prestandard.

SEI has published the first edition of this prestandard in response to the increasing interest in using performance-based approaches for the design of buildings. In addition, this prestandard aims to help resolve conflicts in performance objectives that exist when using prescriptive procedures for the wind design and performance-based procedures for the seismic design of individual buildings. Major innovations introduced here include nonlinear dynamic analysis for wind design, limited inelasticity in the Main Wind Force Resisting System elements, system-based performance criteria, and enhanced design criteria for the building envelope.

Participants

The Structural Engineering Institute of ASCE acknowledges the work of the participants in developing this prestandard. The group of participants comprises individuals from many backgrounds, including consulting engineering, research, wind tunnel laboratories and consultants, education, government, design, and private practice.

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Disclaimer

The Structural Engineering Institute and ASCE, the Charles Pankow Foundation, sponsors, participants and their firms or employees, and contributors offer no warranty, either expressed or implied, as to the suitability of the provisions of this prestandard for application to individual buildings or projects.

This prestandard was prepared through careful deliberations and review using current state of practice and decades of standards development experience. Although further research is needed, and methodologies and criteria will evolve as more knowledge is gained in this area, this prestandard represents the best knowledge available at the time of publication.

Abbreviations

AHJ	Authority Having Jurisdiction
BRB	Buckling-restrained braces
C&C	Components and cladding
CQC	Complete quadratic combination
EOR	Engineer of Record
ESWL	Equivalent static wind load
GFRS	Gravity-Force-Resisting System
IBC	International Building Code
LRFD	Load and resistance factor design
MWFRS	Main Wind Force Resisting System
MRI	Mean recurrence interval
NLTHA	Nonlinear time history analysis
PBD	Performance-based design
PBSD	Performance-based seismic design
PBWD	Performance-based wind design
SSI	Soil–structure interaction

Notation

D_L = Dead load (Chapter 6)

$E[\cdot]$ = Expectation (Chapter 6)

g = Peak factor (Chapter 6)

h = Height of story under consideration (Chapter 7)

H = Height of the building (Chapter 7)

L = Live load (Chapter 6)

L_r = Roof live load (Chapter 6)

W_{MRI} = Wind effect with specified mean recurrence interval (MRI) (Chapter 6)

μ = Mean response (Chapter 6)

σ = Standard deviation (Chapter 6)

Glossary

Acceptance criteria: A quantifiable condition that is judged to express acceptable response of a component or system within the building. Acceptance criteria are most commonly measured by an engineering parameter such as force, stress, strain, or deformation.

Authority Having Jurisdiction (AHJ): The city, county, state, or federal building official with responsibility for administration and enforcement of the building code.

Basis of design: A formal document prepared by the designer that expresses the performance objectives, acceptance criteria, methods of analysis, and methods of design to be used in the design of the building.

Brittle element: *See* Force-controlled element or system.

Deformation-controlled element or system: An element that exhibits predictable response until a specific displacement is reached. When an element exceeds its defined maximum permissible displacement, it is considered to have failed for design purposes. The element may respond linearly or nonlinearly up to the displacement limit, and damage may occur within the element prior to reaching the deformation limit. The element is modeled in the analysis model such that changes in stiffness and strength are accounted for and for which nonlinear response history analysis is required to compute the demand. Within seismic engineering deformation-controlled elements are referred to as ductile elements. Deformation-controlled elements and systems for wind are not referred to using seismic “Ductile Element” naming conventions in order to avoid implying mandating of seismic detailing requirements.

Deformational Velocity: The rate of deformation in a viscous or viscoelastic damping device.

Demand parameter: A quantity (e.g. displacement, velocity, acceleration, force, moment) determined by analysis of the structure.

Design strength: Strength provided by an element or connection, computed as the nominal strength multiplied by the appropriate strength reduction factor.

Drift Damage Index: A measure of the shear strain in a nonstructural component in a Drift Damage Zone.

Drift Damage Zone: The region of a nonstructural element for which the Drift Damage Index is computed.

Ductile element: *See* Deformation-controlled element or system.

Element or system damage: Demand response that causes the element, system, or building to exhibit cosmetic or structural changes after the wind event that may affect its value, usefulness, or function. Damage may not warrant replacement of the element if it is noncritical to the building function.

Element or system failure: Demand response that causes the element, system, or building to lose the ability to resist the demand permanently. Failure suggests the element, system, or building will require replacement to resume function safely or adequately after the wind event.

Expected strength: The mean value of resistance of an element or connection at the anticipated deformation level for a population of similar elements, including consideration of the variability in material strength, strain hardening, and plastic section development. Strength reduction factors are taken as 1.0.

Equivalent static wind load (ESWL): Wind load statically applied to the building representing the wind tunnel determined combination of the background and resonant wind components.

Force-controlled element or system: An element that exhibits predictable (mostly linear) response until a specific force response is reached. When an element exceeds its defined maximum permissible force, it is considered to have failed for design purposes. Force-controlled elements include those defined with force and length (e.g., pressure or moment). Damage may occur within the element prior to reaching the force limit. Within seismic engineering force-controlled elements are referred to as brittle elements. Force-controlled elements and systems for wind are not referred to using seismic “Brittle Element” naming conventions in order to avoid implying mandating of seismic detailing requirements.

Gravity Force Resisting System (GFRS): An assemblage of structural elements assigned to provide support and stability of the structure to gravity loads. The GFRS works together with the MWFRS in carrying gravity loads but is not considered to carry lateral wind loads.

Level: A horizontal plane where a horizontal floor or roof diaphragm exists.

Low Cycle Fatigue: A tensile material strength limit state associated with repeated plastic deformations. The low cycle fatigue limit is governed by the number and magnitude of plastic loading excursions. The number of cycles before low cycle fatigue failure is significantly lower than the number of cycles associated with standard fatigue.

Main Wind Force Resisting System (MWFRS): An assemblage of structural elements assigned to resist wind loads and provide support and stability for the overall building or other structure. The system generally receives wind loading from more than one surface.

Mean recurrence interval: The average expected period of time between occurrences of a specific wind intensity.

Nominal strength: Strength provided by an element or connection using specified material strength, without strength reduction factors.

Performance objective: A specific desired outcome for an element or system of a building during or following a wind event as chosen by the stakeholders and designers. Performance objectives are established at the onset of design and are measured according to their related acceptance criteria.

***Commentary**: Objectives may be tangible (e.g., continued use), intangible (e.g., increased comfort), economic (avoidance or delay of cost or loss), or environmental (reduction of material waste due to loss or reduction of material devoted to construction).*

Performance requirement: Project and/or design requirement that is stipulated by the Authority Having Jurisdiction. The design team may submit to the AHJ request for alternate conforming methods as permitted by ASCE 7-16 Section 1.3.1.3.

***Commentary**: Requirements are identified as the minimum fundamental levels of safety or socially expected building continuity expressed in building codes, ordinances, or similar legislation.*

Ratcheting: Progressive unidirectional accumulation of plastic deformations leading to eventual P-delta instability. Ratcheting can occur in the along-wind or across-wind direction with sufficient plastic demand excursions.

Residual drift: Permanent deformation that exists at the end of the wind event due to inelastic response.

Story: The vertical distance between two adjacent floor levels.

Story drift: Maximum difference in lateral displacement over a story at a common plan location.

REFERENCED STANDARDS

ASCE 7. 2017. *Minimum design loads and associated criteria for buildings and other structures*. ASCE/SEI 7-16. Reston, VA: ASCE.

Chapter 1. Introduction

1.1 PURPOSE

The purpose of *Prestandard for Performance-Based Wind Design (prestandard)* is to advance design for wind for buildings and to enable the performance-based design (PBD), review, acceptance, and construction of buildings using analyses, materials, structural and nonstructural systems, and devices that the prescriptive provisions of building codes and standards may not cover. A secondary purpose is to advance the performance of building envelopes. This prestandard includes the latest knowledge and practices related to design process, risk categorization, performance objectives, wind demand characterization, analysis, acceptance criteria (for both Main Wind Force Resisting System (MWFRS) and building envelope (envelope)), and project review.

The prestandard benefits building owners and developers by enabling design of more efficient buildings that meet the desired building functionality requirements and reduce property damage from wind events while meeting public safety and performance requirements. It benefits designers, reviewers, and building officials by clarifying design requirements for the design and review of buildings. It benefits the general public by enabling development of buildings that renew urban centers and enhance sustainable design and use of natural resources.

> ***Commentary:*** *PBD for wind enables the creation of much "smarter" designs and buildings. The design guidance applies to the MWFRS and the envelope.*

The procedures contained in this prestandard are alternatives to the prescribed procedures contained in ASCE/SEI 7, as well as other standards that are referenced into the 2018 edition of the *International Building Code* (IBC). Both ASCE 7 and the IBC allow for the use of performance-based procedures. Many of the building envelope provisions in Chapter 8 of this prestandard exceed or are in addition to the requirements in ASCE 7 and the IBC.

> ***Commentary:*** *Use of these prestandard procedures constitutes an alternative or nonprescriptive design approach that takes exception to one or more of the prescriptive requirements of the 2018 IBC by utilizing Section 104.11 of that code. Section 104.11 reads as follows:*
>
> *"***104.11 Alternate materials, design and methods of construction and equipment.*** The provisions of this code are not intended to prevent the installation of any material or to prohibit any design or method of construction not specifically prescribed by this code, provided that any such alternative has been approved. An alternative material, design or method of construction shall be approved where the building official finds that the proposed design is satisfactory and complies with the intent of the provisions of this code, and that the material, method or work offered is, for the purpose intended, not less than the equivalent of that prescribed in this code in quality, strength, effectiveness, fire resistance, durability and safety. Where the alternative material, design or method of construction is not approved, the building official shall respond in writing, stating the reasons why the alternative was not approved."*

ASCE 7-16 Section 1.3.1.3 also permits the use of alternate performance-based procedures. Section 1.3.1.3 states the following:

*"***1.3.1.3 Performance-Based Procedures.** *Structural and nonstructural components and their connections designed with performance-based procedures shall be demonstrated by analysis in accordance with Section 2.3.6 or by analysis procedures supplemented by testing to provide a reliability that is generally consistent with the target reliabilities stipulated in this section. The analysis procedures used shall account for uncertainties in loading and resistance."*

1.2 APPLICABILITY

This prestandard's recommendations for design and detailing apply to the wind-loading resistance design of engineered buildings, and the building envelope, and select internal systems that are desired to have enhanced performance. This prestandard shall be permitted to be used for MWFRS design, for the building envelope, or for both as determined by the project stakeholders. All projects using performance-based wind design (PBWD) for the building must use wind tunnel data for the MWFRS. In addition, the prestandard shall be permitted be applied to the envelope if the MWFRS is designed according to prescriptive standards.

Commentary: *At this time, PBD assumes the following characteristics of the building and wind force data:*

- *Wind forces and demands are determined by wind tunnel analysis.*
- *The structural system is well defined, with discrete elements possessing the ductility, toughness, and fatigue resistance necessary to resist the full spectrum of wind demands.*
- *The MWFRS and envelope have a well-defined response mechanism where the system behavior must be understood to predict its response to wind effects, or sufficient data-based performance metrics are available to substantiate the performance of manufactured components considered in the analysis or design (in cases where a manufactured MWFRS or envelope element is used).*

The prestandard is not intended at this time for use with nonbuilding structures and structures without well-defined and documented deformation-controlled elements and connections.

This prestandard considers the wind hazards associated with both extra-tropical and hurricane wind events and the building code provisions for the design of buildings subjected to these events. These structures are intended to resist design wind events and shall be permitted to experience inelastic response of their structural components. Tornado wind hazards are not considered in this prestandard because of the current lack of definition of tornado wind behavior and analysis tools to model building response. Climate change effects are not addressed.

This prestandard is consistent with the provisions and performance objectives intended by ASCE 7 for buildings designed for specified risk categories.

> ***Commentary:*** *Beginning in the early 1990s, performance-based seismic design (PBSD) has allowed construction of structures subject to seismic demand to meet specific performance objectives rather than rely on code prescriptive methods and systems. These efforts largely began with the Applied Technology Council project 33 development of FEMA 273, NEHRP Guidelines for the Seismic Rehabilitation of Buildings, and were undertaken to improve building response to seismic ground shaking, increase structural economy, and focus design efforts on selecting and proportioning those elements best and least suited to resist seismic demands.*
>
> *Application of lessons learned through PBSD such as the Peer TBI initiative (PEER 2017) coupled with broadly available material modeling and response datasets such as ATC-58 (ATC 2016), plus enhanced computational ability, now make similar advances to PBWD possible.*
>
> *The benefits of PBWD can be found in several areas including the following:*
>
> - *Ability to rationally evaluate wind demands and building responses to avoid undesirable outcomes and/or predict expected wind damage and losses.*
> - *Ability to evaluate the expected response of a structure to wind demands and, where appropriate, permit deformation-controlled response of appropriately engineered building elements.*
> - *Ability to apply enhanced detailing, relative to prescriptive code-based requirements, of the MWFRS and/or the building envelope system to rationally reduce damage and losses for design wind effects.*
> - *Ability to gain structural economy through enhanced analysis and design techniques.*
> - *Enhanced ability of the designer to utilize alternate detailing, energy dissipation, or construction techniques through demonstration of building performance.*
>
> *A building owner or design team may further desire to apply PBWD for buildings subject to multiple hazards such as wind plus seismic (Aswegan et al. 2016). In combined seismic and wind demand environments, the strength or stiffness of seismically ductile elements can be governed by wind demands that prescriptive codes require to remain elastic. These seismically ductile elements, if increased for capacity due to wind demands, provide reduced seismic energy dissipation, as well as generate increased demands on surrounding force-controlled structures such as connections and protected elements. The resulting decrease in ductile seismic response and increased demand on potentially brittle structural elements decreases the seismic performance of the structure. Introduction of rational and demonstrated ability of the structure to tolerate limited deformation-controlled response to wind demands helps improve both wind and seismic performance and overall building economy.*

Finally, the building owner or design team may consider wind demands beyond present ASCE 7 requirements. PBWD permits the engineering and understanding of special or signature buildings subject to special wind events such as downslope and/or thermally driven winds unique to a project, or winds at mean recurrence intervals greater than required by ASCE 7.

1.3 USE OF *PRESTANDARD FOR PERFORMANCE-BASED WIND DESIGN*

This prestandard's provisions are compatible with, but amplify and amend, ASCE 7 requirements. When using these provisions for design prior to the official adoption of the 2016 edition of ASCE 7 in the local building code, include the use of the 2016 edition of the standard as a project-specific exception to the building code. When adopting the modifications to ASCE 7 recommended here, include these modifications as exceptions as well, regardless of the adoption status of ASCE 7.

Application of the prestandard requires the user to conform to the following aspects:

1. Ensure that the design team has the requisite knowledge and experience in wind demand characterization, selection of structural and nonstructural systems for resistance to wind loading, nonlinear dynamic structural response and analysis, and structural proportioning and detailing necessary to achieve intended performance.

   Commentary: *Proper execution of these provisions requires extensive knowledge of wind demand characteristics, structural material behavior, and nonlinear dynamic structural response and analysis. Therefore, the design team may require the inclusion of wind consultants, cladding consultants, and other consultants required to provide a design compliant with these provisions. Designers not possessing the requisite knowledge and skills can produce designs that will not perform as intended.*

2. Specify sufficient construction quality assurance to ensure that construction conforms to the design requirements.

   Commentary: *Historically, many of the failures that have occurred during wind events resulted from construction that did not conform to the design intent. Structures designed using these provisions may require limited nonlinear straining of designated structural elements. If appropriate construction quality assurance is not provided, the building may not perform as intended.*

3. Peer-reviewed envelope design is required for all buildings design using Chapter 8 of this prestandard.

   Commentary: *To identify unique demands on the envelope, peer review of the envelope should accompany structural designs utilizing ductile response. The envelope design and the envelope peer review should specifically review envelope detailing with respect to the wind demands and global structural response.*

4. Prior to initiating a design using these provisions, confirm that this approach will be acceptable to the Authority Having Jurisdiction (AHJ).

Commentary: *Acceptance of designs conducted in accordance with these provisions is at the discretion of the building official, as outlined in IBC Section 104.11. Each building official can decline to accept such procedures.*

5. Inform the project developer of the risks associated with the use of alternative procedures for design.

Commentary: *The design and permitting process for buildings that will be constructed in accordance with this prestandard will generally require greater effort and take more time than those that strictly conform to the building code's prescriptive criteria. Further, even in communities where the AHJ is willing to accept alternative designs, the development team bears a certain risk that the AHJ ultimately will decide that the design is not acceptable without incorporation of structural features, which may make the project undesirable from a cost or other perspective.*

6. Provide peer review by qualified experts as part of the design process.

Commentary: *Most buildings designed in accordance with these provisions are expected to sustain damage when subjected to wind events greater than the design allows. Some stakeholders may deem that the damage exceeds reasonable levels and may attempt to hold the participants in the design and construction process responsible for this perceived poor performance. In this event, the Engineer of Record may be required to demonstrate that he or she has conformed to an appropriate standard of care. Doing this for buildings designed by alternative means may be more difficult than for buildings designed in strict conformance to the building code. Independent peer review by qualified experts, as described in Chapter 9, can help to establish that an appropriate standard of care was followed.*

7. When exception is taken to the recommendations contained within this prestandard, provide appropriate technical substantiation for these exceptions to the peer reviewers and AHJ and obtain their approval.

Commentary: *The authors have endeavored to ensure these provisions are broadly applicable to the wind-resistance design of most buildings, given present industry knowledge and practice limitations. However, no prestandard can anticipate every building to which it may be applied, nor can it anticipate advances in the state of knowledge and practice. The authors do not intend to preclude the application of alternative techniques or approaches when they are appropriately substantiated, justified, and approved.*

1.4 INTERPRETATION

This prestandard consists of chapters in which primary guidance takes the form of (a) statements of scope and applicability and (b) imperative text giving instructions on recommended procedures. The Commentary sections explain the basis for these recommendations, as well as how to implement the recommendations and provide alternative approaches.

1.5 LIMITATIONS

This prestandard is intended to provide an informed basis for the wind-resistance design of buildings based on the present state of knowledge, laboratory and analytical research, and the engineering judgment of persons with substantial knowledge in the design and response to wind loadings. When properly implemented, these provisions permit the design of buildings with equivalent, or superior, performance to that attainable by wind design in accordance with present prescriptive building code provisions. Wind engineering is a rapidly developing field in terms of nonlinear response and building envelope design, and knowledge gained in the future is likely to suggest modifications of some recommendations presented here. Individual engineers and building officials implementing these provisions must exercise independent judgment as to the suitability of these recommendations for that purpose.

REFERENCED STANDARDS

ASCE 7. *Minimum design loads and associated criteria for buildings and other structures*. ASCE/SEI. 2016.

ICC (International Code Council). *International building code.* International Code Council. 2018.

REFERENCES

ASCE. 2016. *Minimum design loads and associated criteria for buildings and other structures*. ASCE/SEI 7-16. Reston, VA: ASCE.

Aswegan, K., R. Larsen, R. Klemencic, J. Hooper, and J. Hasselbauer. 2017. "Performance-based wind and seismic engineering: Benefits of considering multiple hazards." In *Proc., 2017 ASCE/SEI Structures Congress*. Reston, VA: ASCE.

FEMA. 1997. *NEHRP guidelines for the seismic rehabilitation of buildings,* FEMA 273. Redwood, CA: Applied Technology Council.

Hamburger, R. O. 2006. "ATC 58: Development of next generation performance-based seismic design procedures for new and existing buildings." In *Proc., Structures Congress 2006: Structural Engineering and Public Safety*. Reston, VA: ASCE.

ICC. 2017. *International building code,* 2018 ed. Country Club Hills, IL: International Code Council.

PEER (Pacific Earthquake Engineering Research Center). 2017. *Guidelines for performance-based seismic design of tall buildings*. Berkeley, CA: Pacific Earthquake Engineering Research Center, Univ. of California.

Chapter 2. Design Process

2.1 SCOPE

This chapter presents the recommended design process for performance-based wind design (PBWD) of the structural system (MWFRS) and the building envelope.

2.2 DESIGN PROCESS OBJECTIVE

Prior to using these recommendations, the design team shall confirm that the building owner is aware of issues, benefits, and risks associated with the use of performance-based design (PBD) procedures, that the design team has the appropriate knowledge and resources, and that the construction quality will be adequate to ensure that the structural design is properly executed.

> ***Commentary:*** *Chapter 1 provides examples of situations where the building owner, developer, or design team may desire to employ PBWD procedures. The objective for PBD is to allow the design team to employ enhanced engineering principles considering the expected structural response of elements, including appropriate inelasticity in designated elements, to meet the design and construction requirements of a building. The engineering principles used should include structural elements that, when subject to time varying wind demands, demonstrate sufficient strength and stiffness throughout the duration of the structural system response to design wind events over the service life of the building.*
>
> *The Engineer of Record (EOR) may determine those elements most capable of providing required resistance, provided the resulting response of the structure is demonstrated to be acceptable using appropriate engineering principles and methods. Elements experiencing inelastic deformations may require evaluation by physical testing to determine their cyclic inelastic response characteristics subject to simulated wind time history loading. The design should be capable of resisting wind effects without unacceptable loss of strength, stiffness, or gravity resistance. It is the intent of this prestandard that inelasticity, if utilized, should be limited to well-detailed designated elements shown to have the necessary toughness to function throughout the required wind demand.*

2.3 DESIGN PROCESS CONFIRMATION

Prior to using these recommendations for design, the design team shall confirm that the Authority Having Jurisdiction (AHJ) approves use of PBD alternatives, including the peer review process described in Chapter 9.

> ***Commentary:*** *ASCE 7 accepts alternate methods of design and construction that are shown to provide equivalent levels of performance (ASCE 7-16, Section 1.3.1.3) subject to approval of the AHJ. Acceptance of alternate methods is predicated on rigorous demonstration of building performance. Although the methods in this prestandard provide a framework for demonstrating acceptable performance when using an alternate method*

of design, the design team should consider the ramifications of using an alternate method carefully before entering a PBWD approach.

2.4 DESIGN PROCESS DESCRIPTION

The following sections describe the design process for the PBWD approach. An acceptable design is complete when the performance objectives are satisfied. Each performance objective shall be evaluated independently, and a variety of analysis and assessment methods are permitted.

Linear elastic analysis procedures shall be permitted for evaluation of Occupant Comfort and Operational performance objectives. Continuous Occupancy performance objective evaluation shall be permitted using one of the three methods described in this section. Both linear elastic and nonlinear time history analysis (NLTHA) shall be permitted. Method 1 describes a linear elastic analysis procedure followed by NLTHA option, while Method 2 and Method 3 use NLTHA procedures with additional reliability assessments.

Commentary: *Acceptable Continuous Occupancy evaluation can be achieved using any of the three methods described. Method 1 provides a time history-based method to demonstrate linear elastic structural response and performance. If all the acceptance criteria are satisfied by a linear analysis, then no further analyses of performance or reliability are required. When the MWFRS remains linear elastic, the structural elements comply with the target reliabilities in ASCE 7 Table 1.3-1.*

If the acceptance criteria are not satisfied by a linear time history analysis, then the design team can either revise the design or conduct a NLTHA to see if the acceptance criteria are met. If the NLTHA successfully meets the acceptance criteria specified in Chapter 7, no further analysis is needed. Appendix A offers guidance on conducting time history-based analyses for Method 1.

Method 2 and Method 3 use a NLTHA to demonstrate inelastic structural response and performance, which must be limited to defined deformation-controlled elements. NLTHA procedures are substantially more complex and require careful attention to modeling of structural characteristics (e.g., strength, stiffness, and wind demands), as well as uncertainties.

If the NLTHA does not meet the acceptance criteria specified in Section 7.4.3, then a reliability analysis can be conducted to verify appropriate system reliabilities. Reliability analysis is used to demonstrate structural performance consistent with ASCE 7 wind design.

The reliability analysis as required in this prestandard should be performed using the method described in Appendix B or an alternative method as described in Appendix C. Both methods also check against the conditional system reliability target defined in Section 7.4.5

Appendix B presents a conditional reliability approach to evaluate structural performance for a given wind scenario that is based upon FEMA P-695 studies (FEMA 2009). This method requires that a minimum of 10 critical design wind scenarios are used in the

NLTHA, a lognormal distribution for strength parameters is utilized, and a probability of failure less than 0.0001 is provided for the design wind scenarios. Appendix B describes a conditional reliability analysis procedure for Method 2.

Appendix C offers additional guidance on alternative approaches for system reliability analysis for Method 3. System reliability analysis may use Monte Carlo simulations in which all significant parameters are taken as random variables with parameter distribution values consistent with laboratory testing, analytical data, and engineering judgment to evaluate the reliability requirements in this section.

When using Methods 2 or 3, the peer review team should include an individual well versed in reliability theory because the design incorporates a reliability investigation. The conditional system reliability goals were developed to be compatible with those contained in ASCE 7 Chapter 1. Elements that comply with the LRFD criteria of ASCE 7 and the companion industry design standards may be deemed to comply with the criterion.

2.4.1 Step 1: Identify Risk Category, Performance Objectives, and Performance Requirements, and Acceptance Criteria

The design team shall identify the risk category, performance objectives, and acceptance criteria, as well as the performance requirements stipulated, for the building design and functions that the PBWD will address. Separate performance objectives, performance requirements, and acceptance criteria need to be developed for the MWFRS and the building envelope (components and cladding, or C&C). Risk category shall be determined in accordance with Chapter 3, minimum performance objectives based on risk category and desired level of performance shall be determined in accordance with Chapter 4, and minimum acceptance criteria shall be determined in accordance with Chapters 7 and 8.

Commentary: *Performance objectives described in Chapter 4 include occupant comfort, operational, and continuous occupancy/limited interruption. All project performance objectives, including those in Table 4-1, should be satisfied to achieve an acceptable design. Acceptance criteria in Chapters 7 and 8 and performance requirements when stipulated (1) should quantitatively evaluate response mechanisms of the building system and elements therein, (2) may be defined for an element based on the response or capacity of another element to protect against force-controlled response or to enhance deformation-controlled response, and (3) may differ for a single element at differing wind mean recurrence intervals (MRI), depending on the severity of the element response and its role in supporting building performance objectives and desired building functionality.*

2.4.2 Step 2: Identify Wind Loads

The design team shall identify wind loads and effects specific to the building site, including representations of wind speed expressed in terms of MRI or as the likelihood that a storm system will generate that demand. The wind tunnel techniques and scientific methods in Chapter 5 shall be used to establish wind directions, velocities, and design loads. Wind tunnel investigation is required under the following circumstances:

- Where linear or nonlinear response history analysis is used to evaluate MWFRS response,
- For any design investigating MWFRS reliability,
- For envelope wind demands if a wind tunnel investigation was conducted for the MWFRS, and
- For buildings that are not generally prismatic in plan.

For linear elastic analyses used for Occupant Comfort, Operational, or Continuous Occupancy evaluation, the wind demand scenarios developed from Chapter 5 shall be used.

For the NLTHA used for Continuous Occupancy evaluation of the MWFRS response, there are two options permitted for the required wind demand input depending on the analysis method chosen, see Figure 2-1.

For NLTHA in Method 1, the two most critical wind demand time-histories developed in accordance with the provisions of Chapter 5 shall be used as outlined in Chapter 6.

For NLTHA in Method 2 or Method 3, a minimum of 10 of the most critical wind design scenarios developed in accordance with the provisions of Chapter 5, in terms of wind directions and speeds appropriate to the structure's risk category, shall be used.

ASCE 7 wind loads in Chapter 26 to Chapter 30 are permitted to be used for envelope evaluation when the MWFRS is not addressed in the PBWD building project.

> ***Commentary:*** *Building response to local wind climatology is highly dependent on building shape, height, dynamic properties, and the influence of natural terrain and built terrain (nearby buildings). The wind tunnel method is considered to be the only reliable technique for establishing specific wind effects on a structure. This prestandard does not address wind events including, but not limited to, tornado or climate change effects. The EOR may consider such wind events using available approved literature.*

2.4.3 Step 3: Conceptual Design

The design team shall select the MWFRS and materials, their approximate proportions, configuration, detailing, strengths, and desired mechanisms for inelastic behavior. The conceptual design selections shall be documented in a Basis of Design document for approval by the AHJ and peer reviewer.

2.4.4 Step 4: Develop a MWFRS Analysis Model

The design team shall develop an analysis model of the MWFRS that can express wind demands and structural responses in engineering terms at the system and element level. The analysis model shall be based on appropriate building code requirements and engineering principles, including the wind demands developed in Step 2. The development of mathematical and/or empirical analysis models and techniques, based on the methods in Chapter 6, shall be able to determine the wind demand on MWFRS elements, including element stresses, strains, and other appropriate parameters.

At the completion of Step 4, the design team shall document the risk category, performance objectives, performance requirements, wind loads, acceptance criteria, methods of analysis, and methods of design in a Basis of Design document for use by the peer review (see Chapter 9).

2.4.5 Step 5: Evaluate MWFRS and Building Envelope Acceptance Criteria

The design team shall demonstrate and document acceptable design in terms of satisfactory evaluation against the acceptance criteria, and performance requirements as stipulated, for occupant comfort, operational, and continuous occupancy performance objectives (Figure 2-1). The design team shall evaluate the building response results for the analysis model developed in Step 4, using the wind loads and effects developed in Step 2, and compare those results with the specified performance requirements and acceptance criteria established in Step 1.

NLTHA to evaluate the continuous occupancy performance objective shall be permitted. In addition, the load effects shall be not less than 80% of the mean recurrence interval (MRI) wind base overturning force or base shear of ASCE 7 prescribed load effect for continuous occupancy. Or, if the specific requirements for wind tunnel testing per ASCE 7-16 Section 31.4.4 are met in the wind tunnel testing required in Chapter 5, the load effects shall be not less than 50% of the MRI wind base overturning force or base shear of ASCE 7 prescribed load effect for continuous occupancy.

> ***Commentary:*** *ASCE 7 Chapter 31 limits wind tunnel effects to not less than 80% of the ASCE 7 Chapter 26 to 29 determined load effect. This limit may be reduced to 50% of the ASCE 7 determined load effects provided additional wind tunnel testing is performed for the building under consideration. A similar minimum wind load effect is adopted for PBWD. The load effect limits apply to all continuous occupancy methods.*

The design team shall conduct PBWD for continuous occupancy evaluation of the MWFRS following one of the three methods indicated in Figure 2-1.

> ***Commentary:*** *Figure 2-1 illustrates permissible PBWD methods of MWFRS analysis and acceptance criteria evaluation for each of the performance objectives. The continuous occupancy performance objective may include NLTHA.*
>
> *For evaluation of the continuous occupancy performance objectives, three methods have been developed and included in this prestandard. Other methods may also satisfy the performance objectives.*
>
> *Method 1 is a deemed-to-comply method based upon engineering experience and judgement; Appendix A provides more guidance.*
>
> *Method 2 is based upon nonlinear time history analysis of the structure followed by a conditional probability reliability assessment of the design; Appendix B provides additional guidance. This method requires the use of a minimum of ten sets of wind demand design*

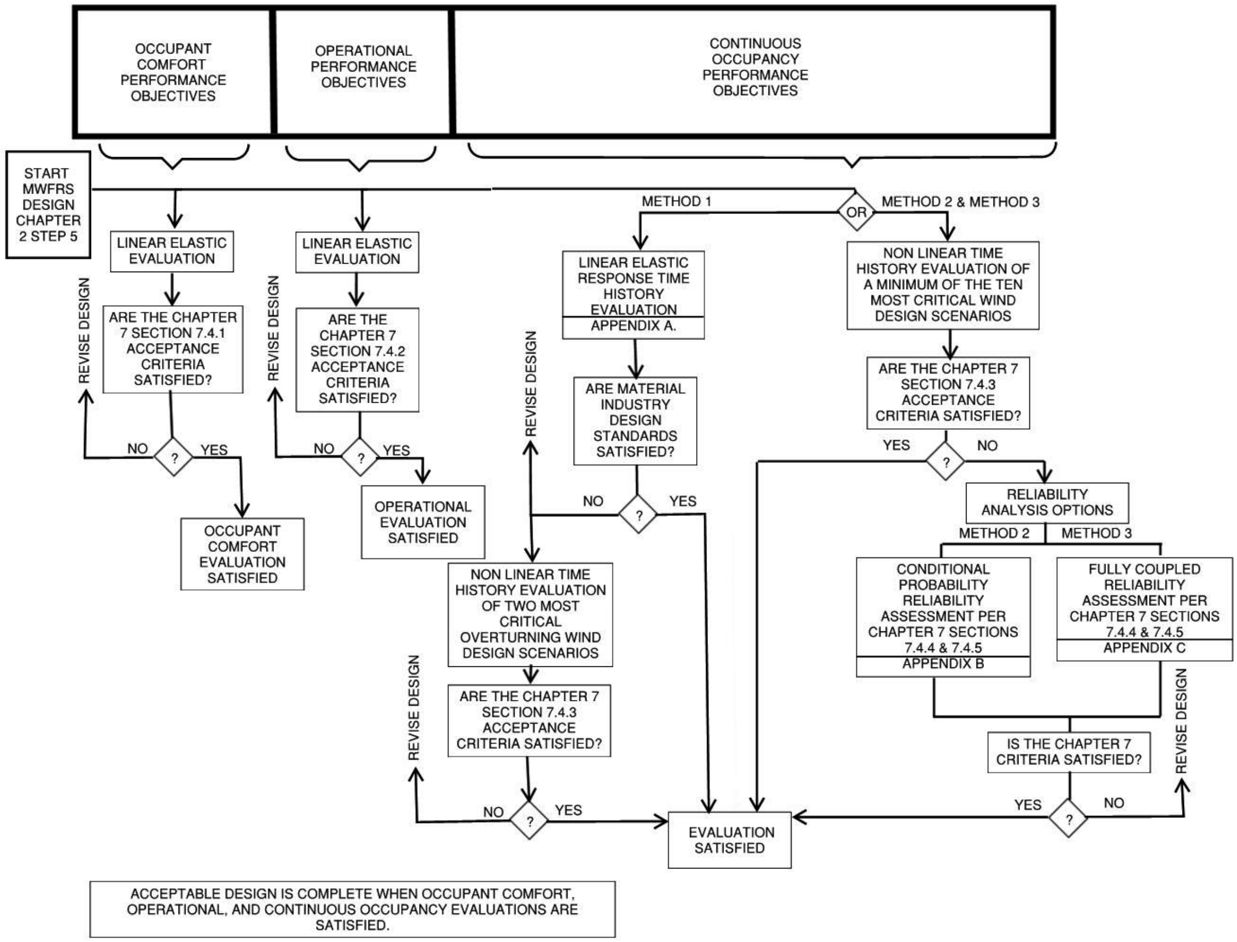


Figure 2-1. Outline of PBWD MWFRS analysis and acceptance methods for each performance objective.

scenarios for loading input for the analysis and an evaluation of the probability of failure as noted following.

Method 3 also is based on nonlinear time history analysis of the structure in conjunction with an alternative procedure to evaluate the reliability of the structure as described in Appendix C.

2.4.6 Step 6: Refine the Design

The design team shall review the building performance achieved in Step 5. Where necessary or desired, the design team and project stakeholders may alter the design to achieve the performance objectives, performance requirements, and acceptance criteria. At the completion of Step 6, the design team shall document the final analysis and design steps employed in a Basis of Design document for review by the peer review team and the AHJ.

Commentary: *The EOR should confirm at this point that the building analysis modeling assumptions, acceptance criteria, system and element response, and element detailing are compatible. For example, if the modeling of an element is only valid between specific calibrated limits, then the response of that element must exist within those calibrated limits. Similarly, if acceptable response of an element is based on specific detailing or number of inelastic cycles limits, then the design must include the necessary detailing and element performance requirements.*

2.4.7 Step 7: Gain Agreement of the Peer Review Team and the AHJ

The peer review team and the AHJ shall review the design steps, calculations, and project documents for agreement with the Basis of Design and for general completeness. The design team shall address requested clarifications and modifications of the project documents or calculations by the peer review team and the AHJ.

Commentary: *At resolution of the review comments by the peer review team and AHJ, the project design is deemed to satisfy the building code requirements for an alternate performance-based design method.*

2.4.8 Step 8: Implement Construction Observation and Supplemental Special Inspections

The design team shall be involved with construction contract administration and provide intermittent observation of construction progress, and the design team shall document construction progress and observations. If the design team intentionally alters the design during construction, and those deviations meaningfully alter the performance objectives or ability of the building to meet the performance requirements or acceptance criteria, the design team shall bring identified deviations of the design intent to the attention of the peer review team and the AHJ.

When required, the design team shall conduct supplemental special inspections to confirm correct installation of the building elements or systems.

Commentary: *International Building Code Sections 1704.3.1, 1704.3.3, 1704.6.3, 1705.11, 1709.4, and 1709.5 (ICC 2017) require special inspection for wind resistance. Supplemental special inspections are critical for achieving acceptable response of elements subjected to design wind effects. Correct installation becomes particularly critical for the building envelope and rooftop equipment where a local breach or failure may create progressive failure that results in widespread damage. Chapter 8 includes recommendations for construction inspection and testing to establish envelope performance against wind, wind-driven rain, and windborne debris.*

REFERENCED STANDARDS

ASCE 7. *Minimum design loads and associated criteria for buildings and other structures*. ASCE, 2017.

REFERENCES

ICC (International Code Council). 2017. *International building code,* 2018 ed. Country Club Hills, IL: International Code Council.

Chapter 3. Risk Category

3.1 SCOPE

This chapter presents the considerations and requirements necessary to establish the project's risk category. This chapter draws on building code requirements enforced by the AHJ and risk-based criteria that consider the project's use and importance to occupants and society.

3.2 RISK CATEGORY DETERMINATION

The design team shall establish the risk category for the building to meet or exceed the risk category requirements in the governing building code.

> ***Commentary:*** *IBC Section 1604.5 requires the determination of the building risk category, which is based on the consequences of building failure and/or nonperformance to the building occupants and users and potential impacts on society. Other performance-based design guides such as PEER TBI (PEER 2017) and FEMA P-424 (FEMA 2010) express general performance objectives specific to hazards and building risk category. With respect to wind, ASCE 7-16 (ASCE 2017) risk category criteria pertain only to the basic wind speed; the standard does not address other issues such as drift control or envelope toughness that are necessary to achieve a desired functional level of building performance. PBWD is intended to overcome this shortcoming.*

3.3 NONMANDATORY PERFORMANCE CONSIDERATION BASED ON COMMUNITY IMPACTS

If agreed to by the project stakeholders, the design team shall be permitted to identify a higher risk category and/or enhanced MWFRS or envelope detailing to meet community impact performance objectives, while maintaining or improving building function or performance.

> ***Commentary:*** *The methods and criteria described in this prestandard are not meant to preclude additional performance objectives, performance requirements, or enhanced design and construction methods to meet desired building functionality.*
>
> *Hurricanes and other design wind events have the potential to impact a large geographical area and cause widespread building damage in communities. Building owners may select to enhance wind performance through voluntary selection of enhanced design, structural detailing, or envelope detailing, where interruption to the building or facility function creates an unacceptable economic and/or community impact. Examples include high-value commercial facilities such as data centers, research laboratories, or manufacturing facilities; disaster response food or medical storage facilities; and select municipal facilities such as city halls beneficial for community disaster response. Publications such as FEMA 577 (FEMA 2007b) have provided enhancement techniques for facilities such as hospitals.*

A building will most likely depend on local services for utilities, transportation, and communication. Minor to severe service interruption may be expected for wind events of various mean recurrence intervals (MRI). Table 3-1 shows examples of possible service interruption by wind events to inform the likelihood of project impact for the wind MRI cited in ASCE 7. The design team may elect to provide enhanced building mechanical or utility infrastructure to increase building and community resilience against utility, transportation, or communication interruption. This is an important consideration for critical and essential buildings.

Table 3-1. Potential Local Service Interruptions and Community Impacts for Wind Events.

Wind MRI	Utility Service Continuity	General Community Impact
1-year	• Interruption to service not expected	• Little notable damage to trees or site work • Limited disruption to normal activities • Regular cleanup of leaves and small branches
10-years	• Short interruption to electrical service of minutes to hours	• Larger broken tree limbs • Common outside activities disrupted (transportation, shopping, exterior events)
50-years	• Interruption to electrical service of hours to days • Possible interruption to telecommunications • No interruption to water or sewer, unless accompanied by flood or prolonged power outage	• Many broken trees and limbs, some healthy trees uprooted
700-years	• Interruption of electrical and telecommunication service of days to several weeks • Interruption of water and sewer for days to weeks • Possible contamination of potable water supply if accompanied by flood	• Large-scale tree damage and widespread debris • Several days to weeks of disruption to economic activity • Casualties expected from wind debris • Evacuations ordered in flood-prone areas of hurricane-prone regions • Moderate population relocation expected

Wind MRI	Utility Service Continuity	General Community Impact
1,700-years 3,000-years	• Interruption of water, sewer, and electrical services for several weeks or months • Aboveground electrical distribution likely severely damaged, requiring rebuilding • Possible contamination of water supply system if accompanied by flood	• Extensive damage to civil infrastructure • Several weeks to months disruption of economic activity • Injuries and casualties expected • Widespread evacuations ordered in flood-prone areas of hurricane-prone regions • Long-term population relocation expected

Land-falling hurricanes typically interrupt municipal power, water, and sewer. Interruptions may range from a few days to several months. For critical and essential facilities (e.g., hospitals, fire and police stations, emergency operations centers, evacuation and recovery shelters) and other buildings that are intended to be operational during and/or soon after a wind events, their design should incorporate special measures to account for temporary loss of municipal utilities. The loss of power, water, and/or sewer has resulted in the forced evacuation of facilities that would otherwise have remained operational or the inability to resume operations.

The level of emergency power required by code for critical facilities, such as hospitals, provide minimum requirements for continued operations. An emergency generator can be beneficial for a building, even if the building does not need to be operational soon after a wind event to support repairs and maintain essential building operations. For example, if the building experiences water infiltration, a generator can facilitate drying of the building. See FEMA P-1019 (2014) for guidance on emergency power systems for critical facilities.

Access to potable water may be interrupted either by loss of power to pumping stations or by contamination of the water supply. FEMA P-543 (2007a) includes recommendations to enhance water and sewer systems from flooding and high winds.

Additional resilience references can be found in the FEMA Building Science Series: https://www.fema.gov/building-science-publications-flood-wind.

REFERENCED STANDARDS

ASCE 7. *Minimum design loads and associated criteria for buildings and other structures*. ASCE/SEI, 2017.

ICC. *International building code.* International Code Council, 2018.

REFERENCES

FEMA. 2007a. *Design guide for improving critical facility safety from flooding and high winds*. P-543. Accessed May 3, 2019. https://www.fema.gov/media-library/assets/documents/8811.

FEMA. 2007b. *Design guide for improving hospital safety in earthquakes, floods, and high winds: Providing protection to people and buildings.* 577. Washington, DC: FEMA, US Department of Homeland Security.

FEMA. 2010. *Design guide for improving school safety in earthquakes, floods, and high winds.* P-424. Washington, DC: FEMA, US Department of Homeland Security.

FEMA. 2014. *Emergency power systems for critical facilities: A best practices approach to improving reliability.* P-1019. Accessed May 3, 2019. https://www.fema.gov/media-library/assets /documents/101996

PEER (Pacific Earthquake Engineering Research Center). 2017. *Guidelines for performance-based seismic design of tall buildings*. Berkeley, CA: Pacific Earthquake Engineering Research Center, Univ. of California.

Chapter 4. Performance Objectives

4.1 SCOPE

This chapter provides minimum building performance objectives. The design team and stakeholders shall select additional or alternate levels of performance that are consistent with performance requirements mandated by the code or AHJ. When enhanced performance is desired, the basis of design shall explicitly state both the desired performance objectives and the means employed to achieve the performance.

> ***Commentary:*** *This prestandard addresses design considerations for the structural system, building envelope, and select building internal systems for design wind events. Other demand types (i.e., seismic, tsunami, and flood) are not addressed in this document, but may require special detailing or system design.*
>
> *Minimum project performance objectives for the MWFRS, building envelope, and nonstructural components are provided in Table 4-1. Additional performance objectives and acceptance criteria may be selected to meet specific project goals.*

4.2 MAIN WIND FORCE RESISTING SYSTEM AND ENVELOPE PERFORMANCE OBJECTIVES

The design team shall include the minimum performance objectives in Table 4-1 for PBWD in the project design and documentation.

> ***Commentary:*** *Table 4-1 provides an overview of the minimum performance objectives contained in this prestandard. The design team should identify pertinent building response and include measurement of the building demands and magnitude of response of each response.*

REFERENCED STANDARDS

ASCE 7. *Minimum design loads and associated criteria for buildings and other structures*. ASCE/SEI 7-16. ASCE, 2017.

REFERENCES

FEMA. 2009. *Quantification of building seismic performance factors*. P-695. Washington, DC: FEMA.

Mohammadi, A. 2016. “Wind performance design for high-rise buildings.” Ph.D. diss. Miami: Florida International Univ.

Table 4-1. Performance Objectives and Acceptance Criteria.

	Occupant Comfort	**Operational**	**Continuous Occupancy, Limited Interruption**
Risk Category II Risk Category III Risk Category IV	Risk category independent	10-years MRI 25-years MRI 50-years MRI	700-years MRI 1,700-years MRI 3,000-years MRI
MWFRS	Performance Objective: The structural system shall remain elastic. The building motions and vibrations shall minimize occupant discomfort at design wind 1-month, 1-year, and 10-years MRI. Acceptance Criteria: See Section 7.2	Performance Objective: The structural system shall remain elastic. The building systems shall remain operational during the wind event for the building risk category. Acceptance Criteria: See Section 7.3	Performance Objective: Specific elements or components of the structural system shall be permitted to become inelastic. The structural system shall withstand a design wind event for the building risk category with a low probability of partial or total collapse. Acceptance Criteria: See Section 7.4
Building Envelope		Performance Objective: The building envelope shall remain attached to the structure. The building envelope shall maintain wind-driven rain resistance. Acceptance Criteria: See Section 8.3	Performance Objective: The building envelope shall remain attached to the structure. The building envelope system shall be designed to maintain wind-driven rain resistance for 25-year MRI wind events for Risk Category II and 50-year MRI wind events for Risk Category III and IV. Acceptance Criteria: See Section 8.3
Nonstructural Components and Systems		Performance Objective: Nonstructural components and systems shall remain attached and maintain wind-driven rain resistance. Acceptance Criteria: See Sections 7.3.1 and 8.4.3	Performance Objective: Nonstructural components and systems shall remain attached. Acceptance Criteria: See Sections 7.4.1 and 8.4.3

Nowak, A. S., and K. R. Collins. 2013. *Reliability of structures*, 2nd ed. Boca Raton, FL: CRC Press.

Spence, S. M. J., and W.-C. Chuang. 2019. “Methods for the efficient estimation of high-rise wind-excited structures.” *Rep. to the MKA Foundation for Performance-Based Wind Design*. Seattle, WA: Magnusson Klemencic Associates Foundation.

Chapter 5. Wind Demand Characterization

5.1 SCOPE

Wind engineering design of buildings using this prestandard requires characterization of wind loads and/or responses at serviceability and ultimate strength levels. This chapter provides guidance on the following topics:

- Wind hazard analysis,
- Wind tunnel test methodologies, and
- Analysis of wind tunnel test data.

5.2 WIND HAZARD ANALYSIS

Probabilistic wind climate analysis shall be used to determine wind speeds and directionality for assessing loads and responses at varying return periods for the limit states of interest.

> ***Commentary:*** *The two approaches to determining site wind speeds are to use codified values (ASCE 7) or to conduct a site-specific wind climate analysis. In most cases, codified values are more conservative than site-specific analyses. Codified wind speed values also do not account, with a few exceptions, for directionality of the wind climate.*

5.2.1 Code-Specified Hazard Maps

If a site-specific hazard analysis is not conducted, basic wind speeds shall be obtained from ASCE 7 or as specified by the appropriate local AHJ.

> ***Commentary:*** *ASCE 7 provides nondirectional strength design wind speeds for a range of return periods, ranging from 300 to 3,000 yr. Serviceability wind speeds are given for return periods of 10, 25, 50 and 100 yr. These wind speeds can also be obtained for any U.S. location from the ASCE 7 Hazard Tool (https://asce7hazardtool.online/) and the Applied Technology Council (ATC) Hazards by Location website (https://hazards.atcouncil.org/). If performance objectives require consideration of return periods between these, intermediate values can be interpolated taking consideration of the logarithmic-linear relationship between wind speed and return period. For return periods outside of these ranges, a site-specific wind climate analysis should be conducted.*
>
> *In areas marked as Special Wind Regions on the ASCE 7 maps, the AHJ may specify design wind speeds or that a site-specific wind climate analysis be conducted in accordance with the requirements laid out in ASCE 7-16, Section 26.5.3 Estimation of Basic Wind Speeds from Regional Climatic Data.*

5.2.2 Site-Specific Hazard Analysis

For PBWD, if the code-specific hazard maps noted in Section 5.2.1 are not used, a site-specific hazard analysis shall be performed to determine the appropriate wind speeds and directionality associated with the MRI for each performance objective.

5.2.2.1 Local climatology and wind storm types

Site-specific wind hazard analyses shall account for all wind storm types relevant to the return period of interest.

> ***Commentary:*** *The storm type that governs each return period of interest may vary among geographical locations. In mixed wind climates, multiple storms types contribute to the extremes. Storm types include synoptic winds (straight-line winds associated with high- and low-pressure weather systems), thunderstorms, hurricanes, and tornadoes. Tornado effects are not considered by this prestandard. Special wind regions may also experience thermally driven winds as a result of local topography.*
>
> *Typically, synoptic winds govern low-return period wind speeds, with the influence of other storm types becoming more significant as return periods increase.*

5.2.2.2 Acceptable analysis methods and relevant factors affecting wind speed data quality

Site-specific hazard analysis shall be based on locally measured historical wind data and/or storm simulation, and where the impact of wind-driven rain on serviceability issues is required, precipitation data. Where historical data form the basis of the analyses, they shall be used to derive basic wind speeds consistent with ASCE 7 at a standard height of 10 m (33 ft) in Exposure Category C. The data shall be screened to ensure that only reliable data points are contained within the statistical analysis.

For longer return periods, where extrapolation of the data to MRIs beyond the length of the historical data set is required, extreme value analysis techniques shall be employed. Allowance shall be made for uncertainty in the extrapolated wind speeds based on the quality of the data set.

> ***Commentary:*** *It is common to find unreliable information within historical meteorological records. Where possible, data from multiple local meteorological stations should be compared, and the reliable data can then be combined into a superstation to increase the effective length of record and hence the reliability of the analyses.*
>
> *Extreme value analysis involves the fitting of a statistical distribution to the maxima. The Method of Independent Storms (Cook 1982, Harris 1999) is generally considered the most robust of current analysis techniques. Before fitting, extracted maxima should be classified according to storm type. Individual extreme value fits should be conducted for each storm type before recombining to determine the overall risk.*

Surface data sets are rarely perfect and often are of limited duration, and as a result a degree of uncertainty is present in the extreme value fits. A reliable statistical approach should be taken to the quantification of this uncertainty and account for it in the resultant wind speed recommendations.

In hurricane regions, there are insufficient quantities of surface data for analysis and storm simulation, typically based on Monte Carlo techniques (Georgiou et al. 1983), must be used.

In Special Wind Regions especially, the influence of local topography on both the anemometer and site locations should be recognized and appropriate adjustments made.

5.2.2.3 Wind profiles

Site wind profiles shall be determined from those provided in ASCE 7 or by alternative recognized methods. The profiles used shall be appropriate to the upwind terrain for the wind directions of interest (see Section 5.3.2.1 for further discussion).

Commentary: *ASCE 7 provides basic wind profiles of mean and gust wind speeds for three uniform ground roughnesses. A simplified technique for accounting for surface roughness changes is contained in ASCE 7 Commentary. These recommendations are based on the work of Deaves and Harris (1978), which was further developed by the Engineering Sciences Data Unit (ESDU 2006a, b). Most wind engineering practitioners use the ESDU approach to determine appropriate boundary layer characteristics for use in wind tunnel testing. This takes into account the upwind terrain and changes in terrain roughness for each wind direction for a sufficient distance to ensure that the assumption of equilibrium conditions is satisfied. A sufficient number of profiles and compass sectors should also be used to account for the directional variation of terrain roughness radially around the site.*

5.2.2.4 Wind directionality

If ASCE 7's wind hazard maps are the sole basis of wind hazard determination, then a uniform directionality shall be assumed. Where supported by site-specific wind hazard analyses, the probability of the occurrence of design wind speeds may vary by direction.

Commentary: *The site-specific wind hazard analyses allow statistical fits to the wind climate data to vary by wind direction. As the variation of wind speed with return period and the contribution of different storm types can change significantly between wind directions, the wind climate directionality may be quite different for serviceability and strength design. The wind climate models provided for use in design must reflect this variability.*

5.2.2.5 Relevant mean recurrence interval

Wind speeds appropriate for the determination of mean recurrence interval load effects shall be developed from the hazard analyses and provided for design. For the determination of pressures for the design of building envelope and façade components, the MRI shall be adjusted so that specified pressures are consistent with the design basis for product specifications and approvals while ensuring that the performance criteria and target reliabilities are maintained.

> ***Commentary:*** *For most buildings, a given mean recurrence interval wind speed can be used to calculate the load effect of interest for the same return period for that given wind direction. However, three important factors must be accounted for:*
>
> 1. *The overall mean recurrence interval load effect needs to consider the total probability of occurrence of loads from all wind directions. In strongly directional wind climates or for buildings with highly directional loading or response characteristics, it may be the case that one or two wind directions dominate the joint probability. For many buildings, however, multiple wind directions may contribute significantly to the probability of occurrence of a given load effect.*
>
> 2. *For tall buildings exhibiting across-wind response, peak load effects can occur at lower return period wind speeds. This is most often an issue for strength design where ensuring the full consideration of the wind speed that results in the largest load effect up to the return period of interest is important.*
>
> 3. *For building envelope and façade components, shorter MRIs are often used in product specifications and approvals, typically associated with ASD approaches to design. In these cases, analysis of wind tunnel data with appropriate MRIs to allow comparison with these data sheets should be conducted.*

5.3 WIND TUNNEL TEST METHODOLOGIES

Appropriate wind tunnel test and analysis methodologies shall be used in the determination of load and response effects of interest.

5.3.1 Review of Wind Tunnel Technique

Wind tunnel testing is the only approach consistent with reliable application of performance-based design principles for wind engineering and shall be used to determine local wind pressures and global wind-induced structural loads and responses.

Wind tunnel tests shall meet the requirements of ASCE 7, Chapter 31, and ASCE 49.

The only wind tunnels to be used for wind loading studies of buildings and structures need to be those capable of simulating the atmospheric boundary layer. Minimum boundary layer simulation requirements are

- Appropriate variation of mean wind speed with height,
- Appropriate variation of longitudinal turbulence intensity with height,
- Suitable turbulence integral length scales, and
- Minimal longitudinal pressure gradient.

The wind tunnel shall be large enough to allow a sufficient radius of surroundings to be included, so their influence on wind effects on the subject building can be assessed.

The wind tunnel shall be capable of generating sufficient wind speed to allow testing to ensure Reynolds number independence at typical test speeds on rigid sharp-edged models, although care must be taken with structures that may demonstrate Reynolds number dependence.

> ***Commentary:*** *Texts and guides to wind tunnel testing include ASCE 49-12 (ASCE 2012), ASCE Manual of Practice 67 (ASCE 1999), AWES Quality Assurance Manual QAM-1-2001 (AWES 2001), and CTBUH* Guide to Wind Tunnel Testing of Tall Buildings *(CTBUH 2013). These provide either a background to requirements, or minimum standards that must be achieved.*
>
> *Standard practice is to model an atmospheric boundary layer for design, regardless of the storm type that may cause the peak load effects of interest. Current research into thunderstorm and tornado loading effects may change this in the future, but there is not yet sufficient validated data to justify alternate approaches for design.*
>
> *A boundary layer shall normally be considered adequate if the variation of mean wind speed with height and the turbulence intensity are both within around 10% of target values, and the turbulence integral length scale is within a factor of 3. A minimal, or ideally zero, longitudinal pressure gradient in the wind tunnel ensures that the measured results are not affected by blockage effects, whether these are positive blockage in closed-circuit wind tunnels or negative blockage effects in open-section wind tunnels.*
>
> *The radius of surroundings required is dependent on the test site. In open country, few surroundings will be required, whereas in more urban environments, individual buildings up to 500 m (1,600 ft) or more away may have an influence on the measured wind effects.*
>
> *The minimum Reynolds number requirements are necessary to ensure similarity between force and/or pressure coefficients measured in the wind tunnel and those that would be expected in the field. A typical minimum value for sharp-edged buildings in turbulent flow is around 5×10^4 based on the mean wind speed at roof height and a representative minimum building width. For most buildings and structures, this means that measured coefficients on rigid models should be Reynolds number independent when minimum Reynolds number requirements are met; that is, they should be unchanging with increasing wind speed. Note that requirements may be more stringent for bodies with curved surfaces, and wind speed scaling is required for aeroelastic studies.*

Structural analysis capable of replicating nonlinear response characteristics may not be practical for structures exhibiting significant aerodynamic damping effects. For structures exhibiting such aeroelastic response, specific aeroelastic model tests with conventional linear–elastic strength design methods may provide a more reliable basis of design.

5.3.2 Additional Data Requirements for PBWD

For PBWD, records shall be of sufficient length to permit evaluation of the variability of nonlinear responses to different event records of the same intensity. Records shall also include an appropriate ramp-up period at their beginning to avoid impulse effects.

5.3.2.1 Minimum number of wind directions

At least 36 wind directions at equally spaced 10° increments of azimuth shall be tested.

Commentary: *Ten-degree azimuthal increments are common for most buildings and structures. In some rare cases, for example where very strong cross-wind responses are present, the peak response may occur at intermediate wind directions and be significantly larger than at adjacent directions, and care should be taken to capture these effects. For most cases, however, the “smoothing” of wind climate data between adjacent directions is sufficient to account for any larger load effects that may occur at smaller directional increments.*

5.3.2.2 Duration and number of records

Time histories measured in the wind tunnel normally approximate 1 hour at prototype (full) scale duration. Shorter records may be used where it has been demonstrated that the statistical analysis techniques used provide equally reliable results, and multiple records may be used as part of the analysis procedure. The peak response of the structure may often be associated with two or three critical wind directions, and it is recommended that records used in the analysis reflect the critical and adjacent wind directions.

Commentary: *The use of a sampling time prototype (full) scale of 1 hour (typically 30 seconds to 1 minute in the wind tunnel) has traditionally been used, as this allows confirmation of statistical stationarity of the wind tunnel data. More recent statistical approaches have shown that shorter periods can sometimes be used without degradation of data reliability. Care must be taken to ensure that data quantities and resolution are sufficient to be consistent with the integration of the wind tunnel data with the PBWD structural analysis framework being employed. It is recommended that a ramp-up to and ramp-down from the peak 1-hour storm event be included as part of the time series record for nonlinear PBWD. The duration of the ramp may be on the order of 1 hour at full-scale.*

5.3.2.3 Pressure tap distribution for façade design

Pressure taps shall be distributed over the areas of the building envelope where external pressures are required and also at locations of potential building openings that will influence internal pressures.

> ***Commentary:*** *Where PBWD is being used to determine design pressures for cladding and building envelope component specification, pressure taps need to cover the areas of interest with a sufficient density to ensure the peak external pressures are captured. Because many components of the building envelope are subjected to net pressures including a contribution from internal pressures, the internal pressures need to be determined through either code-based values or, more accurately, through analysis of measured pressures at potential areas of infiltration. For buildings with uniform or well distributed leakage, this will require pressure taps over the entire or at least large portions of the building envelope. For buildings with potentially dominant openings during the design-level storm, pressure taps should be concentrated at these areas. For buildings with large internal volumes, the adjustment procedure in ASCE 7 should be used to account for the effects of this volume on the internal pressures.*

5.4 EQUIVALENT STATIC LOAD METHOD

Where equivalent static wind loads are provided for design in the linear elastic domain, an adequate number of load cases that combine the measured wind tunnel data and the wind climate analysis to maximize the load effects of interest shall be provided. The load cases shall include suitable distributions of mean, background, and resonant components of response and shall be developed based on consideration of simultaneous building responses about the primary structural axes.

> ***Commentary:*** *The conventional design approach uses equivalent static wind loads to account for the combined effect of quasi-static and dynamic wind effects. Equivalent static wind loads are most commonly determined through high-frequency balance (HFB) or high-frequency pressure integration (HFPI) approaches. The HFB approach only measures applied loads at the base of the building, and hence assumptions must be made about the distribution of the mean and background components of the load, whereas the resonant components are distributed as a function of mass and mode shape. The HFPI approach has the advantages of providing measured distributions of the mean and background components and permits generalized forces to be integrated directly. Whereas an HFB model acts as a mechanical integrator of applied load, HFPI relies on integration of discrete pressure tap data, and hence care must be taken to ensure that sufficient pressure locations can be measured simultaneously to describe the overall pressure fields on the building. For particularly tall and/or slender buildings, the number of pressure tubes that can be extracted simultaneously from the pressure model may be limited, thus limiting the use of this technique.*
>
> *There are a number of different methods of integrating the wind climate analysis with the wind tunnel data to calculate the load effects of interest. These range from simple approaches using nondirectional wind speeds, as would be the case using wind hazard*

maps from ASCE 7-16, to more refined approaches that take into account directional wind climate data. These directional approaches include sector methods, multisector joint probability approaches, upcrossing analyses, and storm passage techniques. Discussion of the pros and cons of each of these approaches can be found in other publications (e.g., Isyumov et al. 2014), but for performance-based design, the method used must be consistent with the design reliability intent.

Typically, load combinations are developed based on maximizing base loads (moments or shears). Maximum and minimum values about each axis in general are used as a starting point, with load cases providing simultaneous companion loads about the other axes. These load cases are applied to the structural model using floor-by-floor distributions with height typically comprising orthogonal translational shears and a torsional moment.

5.5 WIND LOADING TIME HISTORIES METHOD

In a time domain analysis, measured applied loads from a wind tunnel study shall be applied to a structural model of the building. The time histories shall have a sufficiently finely resolved time step to allow the dynamic responses of interest to be determined. Sufficient directional time histories to allow determination of the load effects of interest shall be applied. Wind speeds relevant to generating the load effects of the required MRI must either be determined in advance or from extensive time history analysis.

5.5.1 Scaling Laws: Time, Force, and Pressure Scaling

Time histories of forces and/or pressures shall be converted to prototype (full) scale values for incorporation into structural analysis models.

Commentary: *Basic wind tunnel test data usually come in the form of loading or pressure coefficients that must be converted to prototype (full) scale time series using appropriate scaling factors. This may be done by the wind tunnel laboratory with the prototype (full) scale values provided directly to the design team, or the wind tunnel laboratory may provide the raw time histories with scaling factors for the design team use. The wind loads and/or pressures should be scaled to values consistent with the performance objectives for the design.*

5.5.2 Spatial Resolution of Loading Time Histories

Time histories shall be distributed appropriately taking account of the variation of mean and fluctuating components of wind loading. The method employed for doing this shall be compatible with the wind tunnel test technique that was employed.

5.5.2.1 High-frequency pressure data

Simultaneous pressure data shall be provided for a sufficient number of locations distributed over the building envelope to be able to accurately describe the fluctuating pressures fields over the building as a whole. Areas of influence of each of the pressure time histories shall be provided to the design team by the wind engineer. The time steps of the pressure data shall be sufficiently small to allow excitation of all of the important modes of resonant response.

> ***Commentary:*** *The pressure data can be provided as simultaneous individual pressure time histories at point locations with associated areas of application, or the time histories may be integrated over defined height segments of the building and provided as simultaneous time histories of wind loads on each height segment. Height segments should be selected to allow determination of mode generalized loads corresponding to building mode shapes of interest. Where pressure data are used, ensuring that the spatial resolution of the pressure taps is consistent with the architectural complexity of the building is important. For buildings with a high degree of modulation in the external envelope an increased number, and density, of pressure taps are required. For architecturally complex towers, model scale limitations may limit the ability to use this technique.*

5.5.2.2 High-frequency balance data

When HFB time histories are provided for performance-based design, guidance shall be provided on how to distribute the applied loads with height.

> ***Commentary:*** *While the HFB approach is very accurate in the provision of applied loads at the base of the building, it does not provide information on the distribution and correlation of wind pressures with height. An approximation of the distribution of mean loads can be provided by matching base moments and shears, but no information is available on the correlation of the background excitation with height. As such, the HFB approach may be more limited in its application to advanced performance-based design, especially if higher modes of vibration may be excited. Additional information can be obtained when multiple balances are distributed over the height of the building, although this is a relatively uncommon test technique. Model/balance response characteristics should be filtered from the time histories if that response is likely to affect the calculated protype (full) scale response for any modes of concern.*

5.5.3 Data Analysis

Appropriate data analysis techniques must be used to ensure quantifiable reliability of results.

5.5.3.1 Wind storm type and duration

For synoptic scale storms, the loading and/or pressure data to be used in the analysis shall be of sufficient duration to allow self-stationarity of linear-elastic responses to be demonstrated.

Commentary: *Wind tunnel data are, by their very nature, self-stationary as test records for each direction are obtained for invariant test wind speeds. Typically wind tunnel records are obtained for the equivalent of around 1 hour at full scale, consistent with the Van der Hoven spectral gap. Other wind storm types, such as thunderstorms or tornadoes, may have much shorter durations, but an approach to how the time histories of building loads and responses may vary as a result of the temporal and spatial variation of smaller-scale storm types has not yet been commonly agreed upon and validated.*

Where hurricane events cause the wind effects of interest, the general assumption is that the wind speeds from a given direction will last long enough for self-stationarity of response to be achieved, but that a number of wind directions are likely to be important for building performance within any given storm.

5.5.3.2 Transient effects

Current wind engineering approaches do not address transient effects. For strength design, the assumption shall be that the peak wind speeds generated by transient storms will generate wind loads in the same manner as a longer-duration storm, but alternative approaches may be taken to their inclusion in the assessment of serviceability performance.

Commentary: *In many parts of the United States, particularly inland, the peak design wind speeds may be governed by short-duration storms such as thunderstorms, or for longer return periods, tornadoes. These types of wind storms have fundamentally different temporal and spatial structures to the synoptic wind storms that form the basis of boundary layer wind tunnel testing. Extensive research is underway to determine how this type of storm will load buildings differently than boundary layer storms do, but there is not, as yet, sufficient peer-reviewed evidence to recommend changes to design procedure at present. For life safety design, it is assumed, particularly for taller buildings, that boundary layer assumptions will provide generally conservative results for structural loading. For serviceability wind effects, such as accelerations, pragmatic decisions may be taken to exclude these transient storm types from the statistical wind speed analysis if it can be rationalized that they will not govern the wind effect of concern.*

5.5.3.3 Statistical analysis: Significance of results

Wind loading time histories from a sufficient number of wind directions must be considered (see Section 5.3.2.1) to allow a statistically robust determination of the probability of exceedance of the wind effect of concern. Uncertainties in wind climate analysis, the derivation of dynamic properties, and inherent structural damping shall be considered.

Commentary: *For many buildings, multiple wind directions will contribute to given wind effects, and a sufficiently large number of wind directions must be considered in PBWD to allow accurate assessment of the load effect of interest. An early analysis of wind loads and responses using an equivalent static wind load (ESWL) approach can assist in determining the critical wind directions of interest and be a reference source in assessing the PBWD analyses' reliability.*

For dynamically sensitive buildings, the wind effects predicted from the analyses of wind tunnel data are highly dependent on the building's structural properties. For reliable design, considering a range of possible dynamic parameters, based on modeling uncertainties, may be prudent to ensure the critical wind effects are captured in design.

5.5.3.4 Equivalent mean recurrence interval

The time histories selected for analysis shall be provided for wind speeds consistent with generating the load effect of interest for the performance objective. The mean recurrence interval load effect shall be determined by examining response characteristics for all wind speeds up to those predicted for the equivalent mean recurrence interval wind event.

Commentary: *For wind effects with no contribution from resonant response, the equivalent mean recurrence interval wind speed can be directly correlated with the target mean recurrence interval load effect. Where resonant dynamic response is significant, particularly as a result of vortex shedding, peak load effects may occur at return periods lower than the target MRI for the performance objective. Thus, considering all wind speeds up to the MRI wind speed is necessary to ensure that the load effect of interest is not underestimated.*

Typically, this may be achieved by examination of frequency domain analyses to determine the reduced frequency at which the peak cross-wind responses will occur and calculating the associated critical wind speed.

5.6 PEER REVIEW

Independent peer review of the wind climate and mean recurrence interval wind load effects of interest for PBWD shall be undertaken by an experienced wind engineering practitioner, following the guidance provided in Chapter 9.

REFERENCED STANDARDS

ASCE 7. *Minimum design loads and associated criteria for buildings and other structures.* ASCE/SEI, 2017.

ASCE 49. *Wind tunnel testing for buildings and other structures.* ASCE/SEI, 2012.

REFERENCES

ASCE. 1999. *Wind tunnel studies of buildings and structures.* Manual of Practice 67. Reston, VA: ASCE.

AWES (Australasian Wind Engineering Society). 2001. *Quality assurance manual for wind-engineering studies of buildings.* AWES-QAM-1. Perth, AU: AWES.

Cook, N. J. 1982. “Towards better estimation of extreme winds.” *J. Wind Eng. Ind. Aerod. 9* (1982), 295−323.

CTBUH (Council on Tall Buildings and Urban Habitat). 2013. *Guide to wind tunnel testing of tall buildings*. Chicago: CTBUH in conjunction with IIT and Routledge / Taylor and Francis Group.

Deaves, D., and R. Harris. 1978. “A mathematical model of the structure of strong winds.”*Report 76*. London: Construction Industry Research and Information Association.

ESDU (Engineering Sciences Data Unit International). 2006a. “Computer program for wind speeds and turbulence properties: Flat or hilly sites in terrain with roughness change.” *ESDU Data Item 01008*. Englewood, CO: IHS ESDU International.

ESDU. 2006b. “Strong winds in the atmospheric boundary layer. Part 1: Mean-hourly wind speeds.” *ESDU Data Item 82026*. Englewood, CO: IHS ESDU International.

Georgiou, P. N., A. G. Davenport, and B. J. Vickery. 1983. “Design wind speeds in regions dominated by tropical cyclones.” *J. Wind Eng. Ind. Aerod*. *13* (1), 139−152.

Harris, R. I. 1999. “Improvements to the method of independent storms.” *J. Wind Eng. Ind. Aerod. 80* (1999), 1−30.

Isyumov, N., E. Ho, and P. Case. 2014. “Influence of wind directionality on wind loads and responses.” *J. Wind Eng. Ind. Aerod*. *133* (2014), 169−180.

Chapter 6. Modeling and Analysis

6.1 SCOPE

This chapter provides guidance for the creation of analytical models for determining the response of the main wind force resisting system (MWFRS) to the wind demands outlined in Chapter 5. The analytical models shall include all elements necessary to (1) ensure fidelity of the analytical models of the MWFRS, and (2) evaluate the demands used in the acceptance criteria of Chapter 7. The various response parameters described in this chapter shall be evaluated on the basis of acceptance criteria provided in Chapter 7.

> ***Commentary:*** *Prior to the development of this prestandard, the use of linear elastic analysis was to determine the distribution of wind forces for purposes of design, and strength based (LRFD) procedures were commonly used to ensure acceptably low probabilities of failure under design wind forces. These procedures implicitly admit to the possibility of some limited inelastic response occurring in response to design winds. In this prestandard, inelastic response is explicitly considered, as outlined in Chapter 6 and the procedure of Appendix A. Where linear analysis procedures are used, certain designated deformation-controlled elements and connections are allowed to resist forces up to 1.25 times their expected strength. Limited inelastic response may result in localized damage, residual deformations, loss of element or connection stiffness and/or capacity. Nonlinear analysis procedures are required to assess performance under these conditions.*
>
> *This chapter also provides guidance on the development of analytical models to estimate likely system response under wind loads. Although no predefined acceptance criteria are stipulated in Chapter 6, this chapter provides minimum requirements for performing nonlinear response history analysis for assessment of expected behavior that could be used in, for example, the first order reliability method or equivalent outlined in Appendix B.*
>
> *There is very little information in the literature on performing nonlinear analysis of structures subject to wind loads. However, there is a large amount of information related to performing nonlinear analysis under seismic loading. Four recommended publications are as follows:*
>
> - Guidelines for Performance-Based Seismic Design of Tall Buildings *(PEER 2017),*
> - Nonlinear Analysis for Seismic Design *(NIST 2010),*
> - ASCE 41-17 Seismic Evaluation and Retrofit of Existing Buildings *(ASCE 2017), and*
> - Recommended Modeling Parameters and Acceptance Criteria for Nonlinear Analysis in Support of Seismic Evaluation, Retrofit, and Design *(NIST 2017).*
>
> *It is important to point out, however, that there are considerable differences in seismic loads and wind loads, as well as in the expected nature of response under these loads. Key among these differences are the following:*

1. *The duration of strong wind loads can be on the order of 2 to 4 hours, whereas the duration of a strong earthquake rarely lasts more than 60 seconds. The long windstorm duration may significantly affect the practicality of nonlinear analysis, especially where multiple wind directions and intensity levels are considered.*

2. *For wind loading, the along-wind and the across-wind loading, and responses are significantly different. In general, the along-wind load has a significant mean value, and the loading can be considered as "force controlled." Across-wind loading is dominated by vortex shedding, which has the potential to generate significant dynamic response. Consequently, the across-wind load has a near-zero mean and can generally be considered as "displacement controlled." In addition, for oblique wind directions or irregular geometric profiles, wind loading, and response will generally present a mixture of the aforementioned characteristics. Seismic loading in a sense is similar to across-wind loading. The loading that induces vortex shedding and the associated maximum dynamic response may occur at wind speeds well below classic design wind speeds estimated solely from meteorological information.*

3. *Due to the long duration of wind loading, hundreds or thousands of inelastic excursions may occur, whereas for seismic the number of inelastic excursions is likely less than 25. Due to the larger number of cycles under wind load (assuming the system is allowed to deform inelastically), the magnitude of inelastic deformation that is permitted during the response is significantly less than for seismic due to low cycle fatigue.*

4. *In general, the inertial forces caused by seismic excitation will reduce in the nonlinear response range. In the case of wind excitation, this is not necessarily true, as the reduction in natural frequency of the structural system due to nonlinearity may cause greater resonance and therefore inertia loads unless damping increases sufficiently.*

5. *Cyclic degradation due to a few strong cycles of earthquake loading is expected to be significantly different from that produced by a large number of cycles of wind loading.*

6.2 LOADS

6.2.1 Gravity Loads

Gravity loads (e.g., dead, superimposed dead, live, snow, mechanical) shall be included in the analysis and applied in proportion to the likelihood of their meaningful effect during the wind event considered, for example, expected in-service gravity loads. Self-weight gravity loads shall include the known density of materials and quantity of finishes within the completed structure and the likelihood of appropriate "sustained live load" such as permanent furnishings and storage loads.

6.2.2 External Wind Loads

Wind loads shall be determined on the basis of the wind tunnel procedures provided in Chapter 5 of this prestandard.

Exception: *The Directional Procedure provided in Chapters 26 and 27 of ASCE 7 may be used for determining wind loads, including torsional wind loading, used for demonstrating compliance with Operational Performance Objectives.*

6.2.2.1 Equivalent static wind loads

For estimating demands to be used for assessing the operational performance objective, equivalent static wind loads (ESWLs) may be used. In this case, the ESWLs shall capture the three-dimensional nature of the wind loading and building response. Modeling approaches shall also capture both the background and resonant components of the equivalent load. All methods and assumptions used to determine the ESWLs shall be documented. ESWLs shall comply with the prescriptions of Section 5.4.

Commentary: *Due to the presence of eccentricities between the stiffness and mass centers of each floor, as well as irregularities in the vertical alignment of the mass and stiffness centers, buildings can exhibit meaningful coupling in their principal translational and rotational responses. This mechanical coupling generally can lead to lateral torsionally coupled fundamental modes. This effect must be fully accounted for in developing the ESWLs. Moreover, wind loads acting in the principal translational and rotational directions of the building generally present significant statistical correlation that must be fully modeled when developing the ESWLs.*

6.2.2.2 Wind load histories

Wind load histories for performing linear and nonlinear response history analysis shall be determined by use of wind tunnel testing procedures in accordance with Chapter 5.

Commentary: *To capture the along-wind, across-wind, and torsional components, the wind load histories must be three-dimensional in nature. The loads should be provided at intensities consistent with the performance objectives for which the analytical model is developed. The wind load histories must have a duration that is not less than the expected duration of the windstorm for the intensity level, that is, MRI wind effect, being considered. A full range of wind directions needs to be considered. For use with nonlinear analysis, it may be necessary to develop suites of wind events to account for possible record-to-record variability.*

1. *Approximate: Use of empirical formulas to assess human-comfort-based performance criteria.*

2. *Elastic Static: Analysis of a mathematical model of the building system where the system is modeled with linear elastic properties, and P-Delta effects are included. Linear elastic properties need to reflect the effective stiffness at the expected load level (e.g., concrete effective stiffness, moment frame panel zone stiffness effects, etc.). Wind loads are applied statically, with dynamic response effects included in a rational manner.*

3. *Linear Response History: Analysis of a mathematical model of the building system where linear elastic properties are used, and P-Delta effects are included. Linear elastic properties shall reflect the effective stiffness at the expected load level. Along-wind, across-wind, and torsional wind loads are applied by use of wind tunnel determined load histories, and dynamic response is computed step-by-step in the time domain or by use of frequency domain procedures.*

4. *Nonlinear Response History: Analysis of a mathematical model of the building system where the change in element and connection stiffness and strength due to cyclic inelastic response is explicitly accounted for, and P-Delta effects are included. Along-wind, across-wind, and torsional wind loads are applied simultaneously through the application of wind tunnel determined load histories variability or to carry out incremental dynamic analysis in the case of collapse modeling.*

6.3 ANALYSIS PROCEDURES AND REQUIREMENTS

Table 6-1 provides the requirements for performing structural analysis. These procedures are associated with the basic performance objectives provided in Table 4-1 and are used to determine local (element and connection) or global (system-level) response demands. Acceptable performance is based on criteria established in Chapter 7. The methods of analysis are defined in the following:

1. Approximate: Use of empirical formulas to assess human-comfort-based performance criteria.

2. Elastic Static: Analysis of a mathematical model of the building system where the system is modeled with linear elastic properties, and P-Delta effects are included. Linear elastic properties need to reflect the effective stiffness at the expected load level (e.g., concrete effective stiffness, moment frame panel zone stiffness effects, etc.). Wind loads are applied statically, with dynamic response effects included in a rational manner.

3. Linear Response History: Analysis of a mathematical model of the building system where linear elastic properties are used, and P-Delta effects are included. Linear elastic properties shall reflect the effective stiffness at the expected load level. Along-wind, across-wind, and torsional wind loads are applied by use of wind tunnel determined load histories, and dynamic response is computed step-by-step in the time domain or by use of frequency domain procedures.

4. Nonlinear Response History: Analysis of a mathematical model of the building system where the change in element and connection stiffness and strength due to cyclic inelastic response is explicitly accounted for, and P-Delta effects are included. Along-wind, across-wind, and torsional wind loads are applied simultaneously through the application of wind tunnel determined load histories, and dynamic response is computed step-by-step in the time domain.

If all elements and connections of the MWFRS are designed to remain elastic at the performance objective of continuous occupancy, then linear response history analysis and/or elastic static analysis can be used to estimate demands.

Table 6-1. Analysis Procedures.

Performance Criteria	Analysis Procedure: Approximate 6.5.1	Elastic Static 6.5.2	Linear Response History 6.5.3	Nonlinear Response History 6.5.4
Occupant Comfort	P	NA	P	P
Operational	NA	P	P	P
Continuous Occupancy, Limited Interruption	NA	C	C	P

Note: P = Permitted, C= Conditional on Linear Response, NA = Not Applicable.

Commentary: *For systems including certain types of added damping systems (e.g., nonlinear viscous fluid dampers), nonlinear analysis may be the most accurate tool for determining horizontal floor accelerations.*

6.4 BASIC ANALYSIS REQUIREMENTS

The basic requirements for all analyses are provided in this section.

6.4.1 Model Extent

The mathematical model of the building shall be developed in three dimensions (3-D). Nonstructural elements, auxiliary energy systems, and foundation systems that significantly affect the computed response shall be considered as outlined as follows.

6.4.1.1 The structural system

All elements of the MWFRS and the gravity system sufficient to capture P-Delta effects shall be modeled. Axial, flexural, shear, and torsional deformations shall be included. Increased flexibility associated with beam-column joint (panel zone) deformations in moment resisting frames, cracking in reinforced concrete structures, and system-level second order effects shall be included where such effects are significant. Increased stiffness due to composite action in steel structures may be included and shall be included where additional stiffness creates elevated demands on force-controlled elements.

Commentary: *When considering cracking in reinforced concrete elements, reduction in stiffness due to cracking should be considered. This is of particular importance for beams (such as link beams in coupled wall systems) with span to depth ratios of less than 4.0.*

Note that cracked section property guidelines provided in ASCE 41 (ASCE 2017) do not consider shear cracking.

Including the gravity system is important to provide an accurate distribution of gravity forces throughout the system and is necessary for correct modeling of P-Delta effects.

6.4.1.2 Nonstructural load resisting elements

The elastic stiffness of architectural component and cladding elements shall not be included in the mathematical model of the system.

> **Exception:** *For assessing demands at the occupant comfort and operational performance levels, the effective stiffness of these elements may be included.*

> ***Commentary:*** *Common nonstructural load resisting elements are exterior cladding and partition walls. For assessing the deformation damage indexes, zero stiffness membrane elements can be included in the mathematical model (Aswegan et al. 2015).*

6.4.1.3 Diaphragms

Diaphragms shall be modeled with representative in-plane stiffness where deformation in the plane of the diaphragm influences the transfer of forces between separated portions of the main wind force resisting system. Diaphragm modeling is also required between the main wind force resisting system and basement walls and similar rigid elements (e.g., backstay effects). Diaphragms may be modeled as rigid in-plane where such influences do not occur.

> ***Commentary:*** *Unintended stiffening of the structural system due to out-of-place (bending) stiffness of diaphragms needs to be avoided. For more information on diaphragm modeling and backstay effect, see NIST (2012).*

6.4.1.4 Mass

The mass of the building system shall be represented in the mathematical model and shall accurately represent the spatial distribution of the mass throughout the building system. The mass shall include 100% of the dead load and the expected in-service live load. Where diaphragms are modeled as rigid, the mass moment of inertia related to torsional response shall be explicitly included.

> ***Commentary:*** *Expected live load is included in the calculation of total mass because accelerations experienced by the live load masses are expected to be equal to those experienced by the dead load mass.*

6.4.1.5 Soil/foundation system

Explicit consideration of soil-structure interaction (SSI) effects in the structural model is optional in this prestandard. For high-rise (flexural responding) structures, a sensitivity study for structures shall be included where the stiffness of the foundation alters the first mode response of the building in a meaningful way.

Below grade foundation structure stiffness shall be considered where podium and below grade diaphragms provide meaningful stiffness alterations through coupling with basement walls.

> ***Commentary:*** *Most commonly, the foundation influencing modal properties occurs with deep foundation (piles or caisson) supported buildings with few or no basement levels. If such conditions occur, consider the axial stiffness of the piles or caissons and what influence this stiffness may have on the dynamic characteristics of the superstructure. Base rotation of raft foundations can also influence modal properties. Additional radiation or kinematic damping may be considered. For details, see NIST (2012). Whereas the intent of this prestandard is to allow a level of nonlinear behavior in the MWFRS, nonlinear behavior of the foundations is not envisioned.*

6.4.1.6 Second-order effects

Second-order effects shall be included in the mathematical model in such a manner that translational (P-Delta) and rotation about the vertical axis (P-Theta) effects are captured. Localized second-order effects (p-delta) shall be included where they contribute to a reduction in the effective bending stiffness of slender axial-force resisting elements. Gravity loading for incorporation of second-order effects shall include 100% of the dead load and the expected in-service live load, each distributed realistically throughout the structural system.

> ***Commentary:*** *Local second-order effects occur where very slender elements are loaded in compression, thereby reducing their bending stiffness. Where such effects are considered important, it should be determined that the software used can model such effects.*
>
> *A realistic estimate of life load could be stated as 50% of the reduced and unreduced live load indicated in ASCE 7-16 Chapter 4, or another rational estimate of the expected in-service live load given the building usage and occupants.*

6.4.1.7 Inherent energy dissipation

Where linear or nonlinear dynamic analysis is used, energy dissipation in structural elements shall be included in the mathematical model by use of viscous damping, hysteretic damping, or friction damping. Energy dissipation contributed by elements that are not explicitly modeled shall be represented by linear viscous damping.

> ***Commentary:*** *Inherent energy dissipation is typically represented as linear viscous damping. Linear viscous damping is amplitude independent and frequency dependent. Actual energy dissipation in buildings is very complex and has been observed as being amplitude*

dependent, deformation history dependent, and frequency independent (Spence and Kareem 2014). Different levels of damping may be appropriate for evaluations at the various performance objectives. Selection of equivalent viscous damping ratios should consider this complexity. Where inelastic response history analysis is used, caution should be taken to avoid developing unintended added damping (Hall 2006, Charney 2008, Zareian and Media 2010, Chopra and McKenna 2016).

6.4.1.8 Added energy dissipation

Added energy dissipation devices (e.g., tuned mass damper, tuned liquid column damper, tuned liquid sloshing damper, viscoelastic damper, viscous fluid damper) if provided in the structure, shall be included in the analysis. Analysis methods shall be documented together with the assumptions on the response behavior of the of added energy dissipation devices.

> ***Commentary:*** *The modeling of the response of the structure and added energy dissipation device can be carried out in the frequency domain using classic approaches if it is demonstrated that the structure and added energy dissipation devices will behave linearly. If nonlinear behavior is expected, time domain procedures should be used to model the response of the structure with added energy dissipation devices. Energy dissipation devices used to reduce strength loads (at the Continued Occupancy, Limited Interruption Performance Objective) should be shown to have a reliability of function similar to that of the primary lateral force resisting system.*

The modeling of added energy dissipation systems that are expected to remain operational under continuous occupancy shall be carried out in the time domain with full consideration of any nonlinear behavior of the added energy dissipation devices. If added energy dissipation devices are used that are based on energy dissipation through material nonlinearity (e.g., bucking-restrained braces), modeling shall be based on the results of cyclic test data. Degradation of the device's properties over the expected duration of the windstorm must be modeled and documented. Acceptance criteria for added energy dissipations systems shall be provided by the manufacturer and approved by the Engineer of Record.

> ***Commentary:*** *In developing analytical models for viscous dampers, the reduction in damping capacity generally seen in these devices for an increase in temperature should be modeled.*

6.4.1.9 Gravity loads and nonlinear analysis

Gravity loads in accordance with Section 6.2.1 shall be applied in advance of any static or dynamic analysis that includes inelastic response or second-order effects.

> ***Commentary:*** *Traditional load combinations, as stated in Chapter 2 of ASCE 7, are not applicable for nonlinear analysis, because the principle of elastic superposition is not valid. Thus, for nonlinear analysis, the gravity load shall be applied statically, and then the system with such loads in place shall be analyzed for the applied wind loads.*

6.5 ANALYSIS DETAILS

6.5.1 Approximate Methods

Wind tunnel methods of analysis shall be used to determine horizontal floor accelerations for use in assessing occupant comfort acceptance criteria. System damping, frequencies, and mode shapes shall be determined on the basis of applicable requirements of Section 6.4.

> ***Commentary:*** *Approximate methods appropriate for conceptual design are available to predict horizontal accelerations due to wind effects. One such source is the commentary to the National Building Code of Canada (National Research Council of Canada 2005).*

6.5.2 Static Methods

For the assessment of acceptance criteria that are written in terms of elastic displacement-based measures (e.g., story drifts and deformation damage indexes), analysis methods based on ASCE 7 loads or on wind tunnel derived ESWLs shall be used. Where ESWLs are used, they shall comply with the prescriptions of Section 6.2.2.1.

6.5.3 Linear Response Analysis

Linear response analysis shall be performed in the time domain or in the frequency domain.

6.5.3.1 Analysis in the frequency domain

For the assessment of acceptance criteria that are written in terms of general elastic responses (e.g., displacements, velocities, accelerations), classic frequency domain analyses can be carried out.

> ***Commentary:*** *Peak responses should be estimated as*
>
> $\mu + g\sigma$ (6-1)
>
> *where*
>
> μ = *Mean response*
> g = Peak factor
> σ = Standard deviation
>
> *Unless clearly demonstrated to be unnecessary, at least the first six modal responses should be considered in estimating* σ. *In combining the modal responses, a complete quadratic combination (CQC) rule should be used that accounts for the partial correlation of the generalized wind loads. In estimating* σ *for displacement responses, the background and resonant components should be treated separately, then combined at the level of each modal response. For velocity and acceleration, the background component of* σ *can*

be neglected. In the case of estimating peak responses, the assumptions made in choosing g *should be reported. Input for the frequency domain analysis should be obtained from the spectral analysis of the modal forces estimated from specific wind tunnel tests or carefully selected and documented analytical models.*

The white noise assumption can be used in estimating the standard deviation of the resonant component of the modal responses. The choice of peak factor will depend on the response parameter under consideration.

6.5.3.2 Analysis in the time domain

6.5.3.2.1 Direct integration of fully coupled equations

Elastic response time histories of system response shall be established through direct integration of the fully coupled equations of motion of the system.

Commentary: *For linear response history analysis carried out through direct integration, time steps no greater than 0.1 s should be considered. The time step should be chosen to ensure stability of the integration scheme, with respect to high modes of vibration.*

6.5.3.2.2 Modal superposition

Elastic response time histories of system response shall be established through direct integration of the modal equations. Damping shall be assigned at the level of the modal damping ratios. All modes with a natural frequencies less than 2 Hz (period of vibration greater than 0.5 s) shall be included. Modal responses shall be combined directly in the time domain.

Commentary: *Mode shapes should not take on assumed forms except for preliminary analyses. Mode shapes should be estimated from carrying out an eigenvalue or Ritz analysis in terms of the mass and stiffness matrices of the system. Eigenvalue or Ritz analysis should include P-Delta effects.*

6.5.4 Nonlinear Response History Analysis

Nonlinear analysis shall be carried out for estimating inelastic wind effects when any element or connection in the system is expected to respond inelastically or where added damping systems have nonlinear force-deformation or nonlinear force-velocity relationships. Wind records for performing nonlinear analysis shall be selected in accordance with the requirements of Chapter 5.

Force-controlled elements shall be modeled with elastic properties.

Deformation-controlled elements shall be modeled using expected strength. Modeling of the nonlinear hysteretic behavior of deformation-controlled elements shall be consistent with ASCE 41 or equivalent, or applicable laboratory test data. Test data shall not be extrapolated beyond tested deformation levels. Degradation in element strength or stiffness shall be included in the hysteretic models unless it can be demonstrated that response is not sufficient to produce these effects.

Commentary: *For nonlinear response history analysis, appropriate time steps should be chosen to ensure stability of the integration scheme. Accuracy of the obtained results should be demonstrated through a time step independence study.*

In the case of structures where nonlinearity will occur only in a predefined and limited number of elements, fast nonlinear analysis as defined by Wilson (2002) can be beneficial. The models used for the nonlinear elements must be capable of reproducing all nonlinear phenomena affecting response and demand simulation at the response amplitudes of the hazard level of interest. The nonlinear mechanical behavior of the elements needs to be documented for excitation durations that are consistent with total expected storm duration.

If dynamic shakedown analysis (Chuang and Spence 2017, 2019) is used to demonstrate a safe nonlinear state, then the validity of considering an elastic-perfectly plastic material behavior for the nonlinear components must be demonstrated. The validity of small deformation theory also needs to be documented. In applying dynamic shakedown, the effects of possible wind direction change during the event must be considered. Dynamic shakedown results must be supplemented with analysis of the expected deformations at shakedown. The deformations at shakedown must meet the relevant acceptance criteria of Chapter 7.

6.6 DEMAND PARAMETERS

Demands shall be estimated for the basic performance objectives of occupant comfort and operational through elastic analysis. Inelastic analysis shall be carried out for estimating the demands associated with the performance objective of continuous occupancy. Demand parameters shall be taken as the value closest to the limiting acceptance criteria of the response process over the duration of the wind event and be estimated for both the MWFRS and nonstructural components and, where present, passive energy systems.

Exception: *Demands estimated from elastic analysis can be used for checking the acceptance criteria associated with continuous occupancy if the MWFRS is designed and shown to remain elastic under all wind load effects.*

Demands used in assessing the performance objectives associated with extreme loading conditions shall be estimated from inelastic analysis considering expected material properties.

Commentary: *Consistent with traditional design, the use of elastic demands for assessing continuous occupancy and therefore of an elastic MWFRS at continuous occupancy, implies the adoption of a force-controlled approach for all elements of the MWFRS. If inelastic demands are adopted for assessing continuous occupancy, deformation-controlled approaches are implied for the design. For analytical models developed for extreme loading conditions (e.g., collapse modeling), deformation-controlled approaches are mandatory.*

6.6.1 Elastic Demands

The analytical models developed for the assessment of elastic demands will provide estimates of the following response parameters:

1. Peak story drifts (and drift ratios), velocities and accelerations for the assessment of acceptance criteria associated with C&C (components and cladding) and auxiliary systems. Responses shall be estimated at the location of each C&C or auxiliary system.

2. Deformation damage indexes for the assessment of acceptance criteria associated with C&C.

3. Peak and root mean square accelerations for the assessment of acceptance criteria associated with occupant comfort. Torsional effects shall be included as required by the acceptance criteria.

4. Element force demands.

> ***Commentary:*** *Story drift may be determined as the peak horizontal displacement differential between elements connecting from floor to floor (e.g., cladding, stairs, interior walls). Drifts may be normalized by the story height (drift ratio). Deformation Damage Index (Chapter 7) may be considered to evaluate the concurrent shear strain created by differential vertical movement created by shortening or elongation of members operating within the MWFRS.*

6.6.2 Inelastic Analysis

The analytical models developed for the assessment of inelastic demands shall be developed with the capability of estimating the effects of both geometric (e.g., global P-Delta/P-Theta effects) and material (e.g., material yielding) nonlinearity in the response. The analytical model shall include the capability to assess the potential for failure due to low-cycle fatigue, ratcheting, stiffness degradation, as well as the following:

1. Inelastic deformation demands for all elements,

2. Expected number of elastic excursions and associated maximum stress associated with the excursion, and

3. Expected number of inelastic excursions and the plastic strain demands associated with each excursion.

> ***Commentary:*** *Internal forces, as well as deformations occurring in structural members and elements, should be recorded. Values should be reported as required for checking the member and component acceptance criteria. The validity of the models should be checked and confirmed from the recorded demands. Additional demand parameters, for example, number of cycles of elastic response (high-cycle fatigue) cycles of alternating plasticity or accumulated plastic strains may need to be recorded. See Coffin (1954) and Manson (1953) for background on alternating plasticity (low-cycle fatigue).*

For structural members and elements that are force-controlled, the appropriate internal force demands should be reported. The methods and assumptions used to determine the force demands that will be used in the acceptance criteria should be documented.

For structural members and elements that are deformation controlled, appropriate strains, axial or shear deformations, and rotations should be reported. The methods and assumptions used to determine the deformation demands that will be used in the acceptance criteria need to be documented.

Monitoring of effects is only required where necessary to verify element compliance with acceptance criteria or assigned constitutive relationships or ranges of valid constitutive response.

6.6.3 Passive Energy Systems

When passive energy systems are included as part of the MWFRS, appropriate demands shall be estimated. These include, but are not limited to, the maximum deformation within the device and the maximum deformational velocity within the device. Demands associated with the effects of temperature shall also be estimated.

6.6.4 Global Demands

Peak drift, story drift, and residual story drifts (and drift ratios) shall be determined for acceptance criteria associated with the MWFRS. Demands shall be calculated in the plane of the system under consideration.

Commentary: *Peak drifts and residual drifts, as well as story drift ratios, should be recorded over the height of the building and along at least two orthogonal axes of the building plan. To estimate the torsional response of the building, drift responses should be recorded at multiple points of each floor. Peak dynamic drifts and story drift ratios should be reported at the floor plan locations where the largest values occur.*

6.7 LOAD COMBINATIONS

6.7.1 Occupant Comfort and Operational

For the basic performance objectives of occupant comfort and operational, the following load combinations shall be considered:

LC1: $1.0D_L + L_{ex} + 1.0W_{MRI}$

where

D_L = Dead load,
L_{ex} = Expected in-service live load, and
W_{MRI} = Wind effect with specified MRI.

Commentary: *The MRI is based on the selected risk category and selected performance objectives for the building. When other sustained lateral loads are expected (e.g., sloping sites), the effects of these other lateral load must be considered in conjunction with the effects of the wind loads. Expected in-service live load is the live load in place at the time of the wind event.*

6.7.2. Continuous Occupancy

Method 1 inelastic analysis shall follow these load sequences:

LC2: $1.2D_L + L + 0.5(L_r \text{ or } S \text{ or } R) + 1.0W_{MRI}$

LC3: $0.9D_L + 1.0W_{MRI}$

Method 2 and Method 3 analysis shall follow a load sequence of

LCA: $1.0D_L + L + 1.0W_{MRI}$

where

L = Live load, reduced according to ASCE 7,
L_r = Roof live load,
S = Snow load, and
R = Rain load.

Exception: In combination LC2, the companion load S shall be taken as either the flat roof snow load or the sloped roof snow load, as specified in ASCE 7.

Commentary: *For nonlinear time history analysis using load combinations 2, 3, and 4, the gravity loads should be applied initially followed by appliation of wind loads.*

REFERENCED STANDARDS

ASCE 7. *Minimum design loads and associated criteria for buildings and other structures*. ASCE/SEI, 2017.

ASCE 41. *Seismic rehabilitation of existing buildings*. ASCE/SEI, 2017.

REFERENCES

ASCE. 2017. *Seismic rehabilitation of existing buildings*. ASCE/SEI 41-17. Reston, VA: ASCE.

Aswegan, K., F. Charney, and J. Jarrett. 2015. “Recommended procedures for damage-based serviceability design of steel buildings under wind load.” *Eng. J. 52* (1), 1−25.

ATC (Applied Technology Council). 2019. *ATC design guide 3: Serviceability design of tall buildings under wind loads*. Redwood City, CA: Applied Technology Council.

Charney, F. A. 2008. “Unintended consequences of modeling damping in structures.” *J. Struct. Eng. 134* (4), 581−592.

Chopra, A. K., and F. McKenna. 2016. “Modeling viscous damping in nonlinear response history analysis of buildings for earthquake excitation.” *Earthq. Eng. Struct. Dyn. 45*(2),193−211.

Chuang, W.C., and S. M. J. Spence. 2017. “A performance-based design framework for the integrated collapse and non-collapse assessment of wind excited buildings.” *Eng. Struct. 150* (Nov.), 746−758.

Chuang, W. C., and S. M. J. Spence. 2019. “An efficient framework for the inelastic performance assessment of structural systems subject to stochastic wind loads.” *Eng. Struct. 179* (Jan.), 92−105.

Coffin, L. F. 1954. “A study of the effects of cyclic thermal stresses on a ductile metal.” *Trans. ASME, 76*, 931−950.

Griffis, L.G. 1993. “Serviceability limit states under wind loads.” *Am. Inst. Steel Constr. Eng. J. 30* (1), 1−16.

Hall, J. F. 2006. “Problems encountered from the use (or misuse) of Rayleigh damping.” *Earthq. Eng. Struct. Dyn. 35* (5), 525−545.

Manson, S. S. 1953. “Behavior of materials under conditions of thermal stress.” *Heat Transfer Symp.* Ann Arbor, MI: Univ. of Michigan, Engineering Research Institute, 9−75.

National Research Council of Canada. 2005. *User’s guide—NBC 2005 structural commentaries.* Ottawa, ON: Canadian Commision on Building and Fire Codes.

NIST (National Institute of Standards and Technology). 2010. *Nonlinear structural analysis for seismic design: A guide for practicing engineers.* NIST GCR 10-917-5. Gaithersburg, MD: NIST.

NIST. 2012. *Soil-structure interaction for building structures.* NIST GCR 12-917-21. Gaithersburg, MD: NIST.

NIST. 2017. *Recommended modeling parameters and acceptance criteria for nonlinear analysis in support of seismic evaluation, retrofit, and design.* NIST GCR 17-917-45. Gaithersburg, MD: NIST.

NRCC (National Research Council of Canada). 2015. *National building code of Canada.* Ottawa, ON: NRCC.

PEER (Pacific Earthquake Engineering Center). 2017. *Tall buildings initiative: Guidelines for performance-based seismic design of tall buildings.* Berkeley, CA: Univ. of California, PEER.

Spence, S.M.J., Kareem, A. 2014. "Tall buildings and damping: A concept-based data-driven model." *J. Struct. Eng. 140* (5), 04014005.

Wilson, E. L. 2002. *Three dimensional static and dynamic analysis of structures: A physical approach with emphasis on earthquake engineering,* 3rd ed. Berkeley, CA: Computers and Structures.

Zareian, D., and R. A. Medina. 2010. "A practical method for proper modeling of structural damping in elastic plane structural systems." *Comp. Struct. 88* (1), 45−53.

Chapter 7. Acceptance Criteria: Main Wind Force Resisting System (MWFRS)

7.1 MWFRS ACCEPTANCE CRITERIA FRAMEWORK

7.1.1 General

This chapter establishes acceptance criteria to verify that the response of the Main Wind Force Resisting System (MWFRS) to wind effects—calculated in accordance with the provisions of Chapter 6—will meet the performance objectives defined in Table 4-1. The performance objectives pertain to Occupant Comfort, Operational, and Continuous Occupancy with corresponding mean recurrence intervals (MRI) according to the risk category of the structure. Wind tunnel testing as specified in Chapter 5 is a prerequisite for the PBWD approach to be adopted for design of a structure's MWFRS.

> ***Commentary:*** *Conducting a performance-based wind design (PBWD) enables a building to achieve performance in a wind event that exceeds current code requirement and meets performance objectives. In addition, the application of PBWD in a building should allow seismic performance to be fully realized in areas where there is both a seismic and wind risk. The following aspects of PBWD are essential for the characterization and evaluation of the structural performance:*
>
> - *Performance objectives and acceptance criteria for the MWFRS and envelope should be established simultaneously to ensure proper coordination and design.*
> - *When the MWFRS is subject to both wind and seismic effects, performance objectives and acceptance criteria for wind and seismic response should be established simultaneously to ensure proper coordination and design.*

7.1.2 Forms of Acceptance Criteria

Acceptance criteria is dependent on the performance objectives and the analysis procedure. The criteria for the three categories of performance objectives take the following forms:

- Occupant Comfort—acceleration limits
- Operational—elastic response, drift, and peak deformation damage index (DDI) limits
- Continued Occupancy—limited member inelasticity in deformation-controlled elements, limited member forces in force-controlled elements, peak drift limits, residual drift limits and building stability

Acceptance criteria includes evaluation of incipient collapse under the Continuous Occupancy performance objective for member inelastic response, permanent deformation limits, and building stability.

7.1.3 Categorization of Structural Members

Inelastic response is permitted in certain pre-defined deformation-controlled elements of the MWFRS under the Continuous Occupancy performance objective. It is necessary to categorize all load effects on members as either deformation-controlled or force-controlled.

7.1.3.1 Force-controlled elements and actions

Where linear and nonlinear acceptance criteria are not specified in this prestandard, actions shall be taken as force controlled, unless component testing is performed to determine acceptance criteria.

Commentary: *Force-controlled elements are defined as those elements having limited ductility and are deemed to have failed upon exceedance of an applied force level.*

7.1.3.2 Deformation-controlled elements and actions

For wind loading, the deformation level shall account for multiple cycles of inelastic deformation and the potential for low-cycle fatigue.

Commentary: *Deformation-controlled load effects are defined as those actions that induce some acceptable degree of inelastic response and are deemed to have failed upon exceedance of a predefined deformation level.*

The duration of windstorms is varied, from a fast-moving thunderstorm downburst to hurricanes that can last for days. Although the peak of hurricanes affecting a specific building is short-lived, high-intensity winds can have an impact on a region for several hours. Hysteretic behavior of building structural members has primarily been studied for seismic loads under specifically developed loading criteria. Consequently, current analytical modeling tools are predicated on existing hysteretic behavior of materials and structural members that has been developed for seismic loading. Although it is a good starting point, application of hysteretic performance data developed for seismic loads should be implemented judiciously because the post-peak strength and stiffness degradation characteristics of structural members during several cycles over a long duration is unknown. While research and testing on the performance of building elements under long-duration wind loads (including development of loading protocols) is undertaken and results analyzed to provide guidance for PBWD, design of anticipated hysteretic behavior of predefined deformation-controlled MWFRS members can be implemented by adequate consideration of seismic detailing for the members designed to respond inelastically.

To control nonlinear actions as part of the PBWD process, designers may comply with the acceptance criteria in this chapter by focusing nonlinear behavior in predefined deformation-controlled elements specifically designated for this purpose. These MWFRS elements can be specifically designated to dissipate energy generated during peak demand through nonlinear behavior. If these specifically designated elements are not part of the gravity *system, in extreme cases they could also be designed for replacement subsequent to large-scale wind load events.*

7.2 MWFRS ACCEPTANCE CRITERIA FOR OCCUPANT COMFORT PERFORMANCE OBJECTIVES

Project acceptance criteria for acceleration limits shall be met.

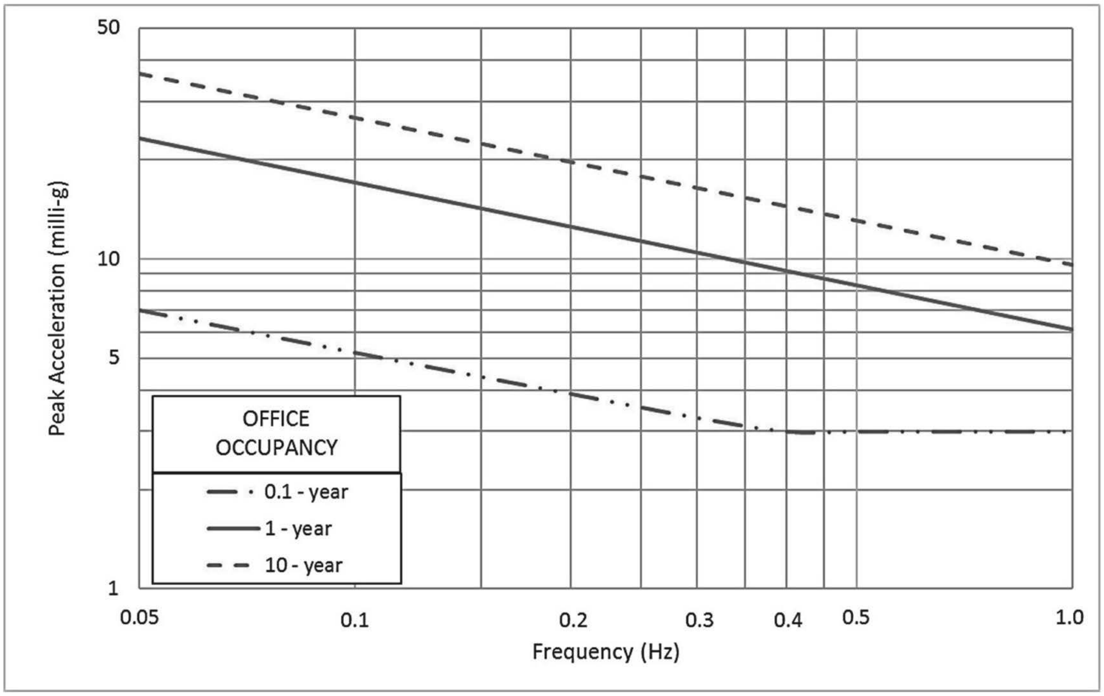
Commentary: *For wind-induced sway motions, Occupant Comfort criteria are expressed as frequency dependent peak acceleration limits. Residential buildings typically have accelerations limits that are 2/3 of the office limit (ISO 2007). The 1-yr criterion given is from ISO10137 (ISO 2007), and the 10-yr criterion is 1.6 times the 1-yr criterion, where the 1.6 factor is derived from information provided in the ISO6897 (ISO 1984) standard. The 0.1-yr criterion is the same for residential and office building, and it is based on a combination of the Architectural Institute of Japan (AIJ 2004) guidelines for 10% perception rate, ISO (2007), Denoon et al. (2000), and Burton et al. (2015) and is given in the ATC-DG3 (2018) guideline. Figure 7-1 (a) and (b) present these criteria graphically.*

Dynamic sway of buildings in wind can be perceptible to people, and occupant comfort may be one of the governing factors of design. Depending on the height and slenderness of the building, the dynamic response, often caused by vortex shedding, can cause perceptible accelerations at relatively low return period winds. In some cases, occupant comfort issues may occur more frequently than once a month. This drives the requirement to assess occupant comfort conditions at multiple return periods: 0.1 years, 1 year, and 10 years.

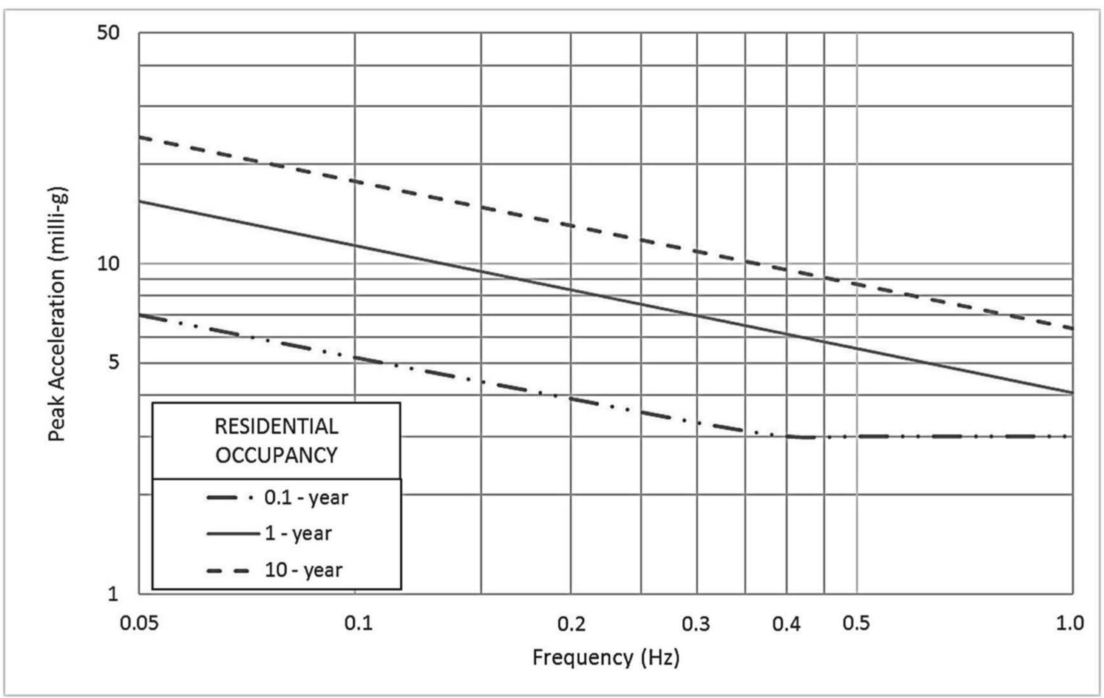
Dynamic sway can also introduce long-term concerns for elevators. Elevator speeds will slow down once accelerations above a certain threshold are observed, and these slowdown conditions are elevator and building specific. The acceleration motion of the building has the potential to create harmonic sway of the elevator cables themselves, and when accelerations become large, the elevators will park at predetermined floors in the building. Elevators will also temporarily shut down in the event of excessive overall building deflections and associated story drifts. These shutdowns are deemed acceptable for continuous occupancy wind events.

Visual and auditory cues can be equally important to consider. It is advisable to remove the potential for these sources of occupant distraction. Avoidance of chandeliers, hanging draperies, and pendants can reduce visual cues. Avoidance of piping in contact with partition tracks can reduce auditory cues.

In the case of hospitals or other critical buildings, accelerations should be checked for essential equipment needed to maintain the functionality of the building. If accelerations are deemed to potentially cause failure to essential equipment, retaining systems for the equipment should be designed.



(a) Comfort criteria for office occupancy buildings.



(b) Comfort criteria for residential occupancy buildings.

Figure 7-1. Frequency dependent acceleration limits for occupant comfort criteria.

7.3 MWFRS ACCEPTANCE CRITERIA FOR OPERATIONAL PERFORMANCE OBJECTIVE

No yielding of the MWFRS is permitted.

7.3.1 Peak Drift

Specific peak drift limits under the Operational performance objective are not required unless judged necessary by the Engineer of Record.

> ***Commentary:*** *Peak drift considers the overall roof deflection of the building relative to the base. The intent of providing peak drift criteria is to limit objectionable response under semi-frequent wind events. If alternate methods of dealing with envelope, interior partition, and elevator performance issues are developed, then the recommended peak drift limit may be relaxed.*
>
> *The appropriate peak drift is a matter of engineering judgment and building designers should discuss the limits with the appropriate project stakeholders prior to commencement of the design. A single peak drift limit at the Operational performance objective may not be suitable or telling for all types of projects. Furthermore, peak drift does not guarantee adequate performance of the envelope system. Consideration of peak drift can be meaningful in consideration of construction in close proximity to where building sway moves portions of the structure beyond the property boundary.*
>
> *In general, peak drift ratio of H/400 to H/500 is recommended at Operational performance objective.*

7.3.2 Residual Drift Ratio

No residual drift is permitted.

7.3.3 Deformation Damage Index

Structural deformation shall be determined by the deformation damage index (DDI) method and limited according to the composition of non-structural elements within the structure.

> ***Commentary:*** *In taller buildings where there is significant axial deflection of vertical elements, the drift ratio may not be an accurate measure of the "racking deformation" that internal and external panels experience and which is the primary source of damage to those elements. In this case, the DDI is preferable, which estimates the in-plane shear strain (Charney 1990, Griffis 1993, Aswegan et al. 2015).*
>
> *The DDI method determines shear strain in an element by measuring the displacement at the four nodes of a square or rectangular element. Mathematically, the strain in the panel ABCD, as illustrated in Figure 7-2 and can be defined as in Equation (7-1).*

$$DDI = 0.5\left[\frac{X_A - X_C}{H} + \frac{X_B - X_D}{H} + \frac{Y_D - Y_C}{L} + \frac{Y_B - Y_A}{L}\right] \tag{7-1}$$

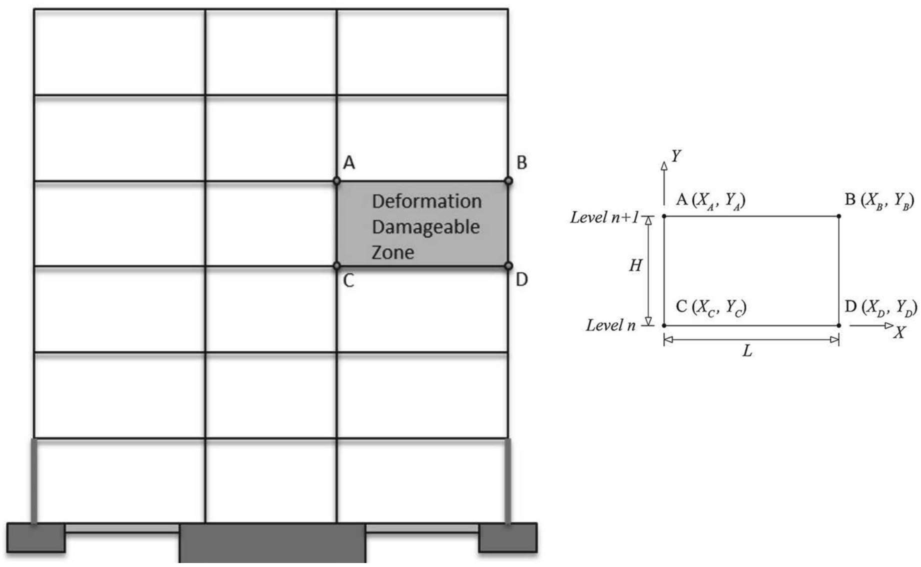


Figure 7-2. Terminology for computation of deformation damage index.

DDI limits for building design are provided in Table 7-1.

Commentary: *DDI is a measure of damage potential in envelope and partition systems whereas story drift in some cases may under or overestimate potential issues.*

DDI includes vertical racking, which is important in flexural type deformation that occurs in braced frames, shear walls, or tube systems with closely spaced columns. In some situations, floors must warp to accommodate the deformation required with stiff in-plane diaphragms and, in turn, this has the potential to cause damaging strains in interior spaces. This effect is not accounted for in the story drift limits, which can therefore underestimate damage potential.

The DDI filters out rigid body rotation which occurs in taller buildings, particularly on the upper floors, yet by itself does not cause damaging strain. Through the inclusion of this rigid body rotation in the story drift estimate, an over estimation of the damage potential can occur.

Soft joints at the head of interior partitions require a restraint to out-of-plane lateral movement while maintaining a gap to the floor above to avoid gravity load transfer to partitions. In-plane racking, which can lead to excessive cracking and/or noise in the building, needs to be controlled.

Alternate values for the DDI at the Operational performance objective may be acceptable where assembly testing or similar evidence shows acceptable element response to applied movements and strain.

Table 7-1. Deformation Damage Index Limits (ATC Design Guide 3).

Building Element		Suggested DDI Limit	Notes
Exterior Cladding	Brick veneer w/ metal studs	0.0025	1
	Brick veneer w/ unreinforced masonry	0.0025	1,2
	Plaster or stucco	0.0025	3
	Architectural precast	0.0025	4
	Stone clad precast	0.0025	4
	Architectural metal panel	0.0100	5
	Curtain wall or window wall	0.0025	6
Interior Partitions	Gypsum drywall, plaster	0.0025	7
	Concrete masonry, unreinforced	0.0015	8
	Tile or hollow clay brick	0.0005	9
Elevators	Drywall enclosure	0.0025	10

Notes:

1. Steel relief angles supporting the brick are provided at each floor with 3/8 in. soft joints and 3/8 in. control joints are provided in the brick at each column bay.
2. Control joints are provided in masonry walls and/or isolation joints (3/8 in. soft joints) are provided between CMU and structural frame.
3. Panelized wall with 3/8 in. control joints used at each floor line and between each column bay.
4. Assumes flexible and deformation-controlled connections of panels to floors or columns with ¾ in. joints between panels. Panel connections to floors or frame are simply supported or determinant.
5. Metal panels are designed with this limit or as defined by manufacturer. Other building elements generally demand stricter limits.
6. Applicable to most off-the-shelf systems. The manufacturer shall be consulted, and the limit defined in specifications. American Architectural Manufacturers Association (AAMA) wall testing recommended for most projects unless similar test results exist.
7. Soft joints recommended between floors as defined in ASTM C754 to allow for LL deflection and racking.
8. Applies if CMU is constructed hard against floors and structural frame. Soft joints recommended between floors between structural frame to accommodate building sway and to eliminate stiffness contribution to lateral load resisting system.
9. Assumes wall system constructed hard against floors and structural frame. Soft joints recommended between floors and to structural frame to accommodate building sway.
10. Proper performance of elevator system requires a knowledge of building mode shapes, frequencies, deflections and accelerations under design wind loads. Information shall be placed in contract documents for elevator manufacturer design.

7.3.4 Additional Acceptance Criteria at Operational Performance Objective

Component acceptance criteria not specified in this prestandard shall be determined by qualification testing. Peer review of the testing process shall be conducted by an independent engineer approved by the AHJ.

Commentary: *The test specimen for qualification testing should replicate, as much as practical, the geometry and boundary conditions as in the actual building. Consideration should be given to the possible influence of vertical gravity loads on the component lateral force resistance. The use of multiple test data allows some of the uncertainty with regard to actual behavior to be defined. At least two tests are required with the same loading protocol consistent with the customary practice of having multiple specimens when component testing. A specific loading protocol has not been recommended, as selection of a suitable loading protocol depends on the anticipated failure modes and sequences of the subassembly and the character of excitation it is expected to experience in the actual structure. The loading protocol has significant influence on the resulting envelope of the force–displacement relationship (back-bone curves).*

7.4 MWFRS ACCEPTANCE CRITERIA FOR CONTINUOUS OCCUPANCY PERFORMANCE OBJECTIVES

Minor localized yielding of deformation-controlled elements within the MWFRS is permitted.

The building shall be provided with at least one continuous load path to transfer wind forces in any direction, from the point of application of the wind load to the final point of resistance.

7.4.1 Peak Drift and DDI

Peak drift ratio of H/200 to H/300 is permitted at the Continuous Occupancy Performance Objective.

Commentary: *Similar to the Operational performance objective, the appropriate peak drift for the Continuous Occupancy limit state is a matter of engineering judgment to satisfy performance objectives. Building designers shall discuss the limits with the appropriate project stakeholders prior to commencement of the design. The intent of providing these criteria is to limit issues under this event with elevator and wall cladding damage. If alternate methods of dealing with these primary issues are developed, then the recommended peak drift ratio limit may be relaxed.*

In some cases, the DDI limits may be met at the Operational performance objective, but a large dynamic response may be observed at a mean recurrence interval between the Operational performance objective and the Continued Occupancy performance objective. If this is the case, then these criteria would apply at the MRI of peak response. These criteria help to improve the performance of a building by enforcing the consideration of a "design space" that looks at a building's response across all MRIs. Minor excursions of the DDI limits are unlikely to result in failure; however, significant excursion resulting from dynamic building response could cause an undesirable response. The designer also needs to verify that there is no compromise to overall stability.

The peak drift limit is provided to limit issues with elevator operation and alignment. Peak drifts beyond these limits may result in issues with tolerance in the elevator shaft.

Oversizing the shaft to tolerate larger drifts is possible. Consultation with an elevator consultant is recommended regarding drift limits.

Wall cladding system elements shall not fall from height at DDI ratios of 0.02 unless the structural analysis of incipient collapse justifies a less onerous criterion.

7.4.2 Residual Drift Ratio

In the Continuous Occupancy state, the residual peak drift ratio shall not exceed H/1000 and h/1000 on a per story basis.

> ***Commentary:*** *Residual story drift may arise when the predefined deformation-controlled elements of the MWFRS experience minor inelastic response. This can be assessed explicitly only by using nonlinear time history analysis.*

For incipient collapse assessment, the residual story drift ratio shall not exceed h/200.

> ***Commentary:*** *Residual story drift may arise when the MWFRS suffers significant inelastic response. The designer should consider if the residual drift is likely to induce creep related amplification of deformation, and if so, limit the residual drift to mitigate creep amplification.*

7.4.3 Strength Limits and Acceptance Criteria for Method 1

The following acceptance criteria represent a consensus view of the limiting structural demands permissible for this performance objective. These criteria may be adjusted with adequate experimental evidence to reduce consequence to structural function or cost.

For structural members having both deformation-controlled and force-controlled resistances (e.g., reinforced concrete shear wall), each action (flexure, shear, axial) should be evaluated independently. In general, maximum force means governing instantaneous combinations of axial, bending, and shear demands.

7.4.3.1 Force-controlled elements and actions

Calculated demand to capacity ratios for force-controlled elements shall not exceed 1.0, where demand is calculated per provisions in Chapter 6 and the capacity is calculated as follows:

1. For reinforced concrete elements and their connections, the capacity is the design strength in accordance with ACI 318, including appropriate phi-factors.

2. For structural steel and composite steel and concrete elements the capacity is the design strength in accordance with AISC 341 and AISC 360.

> ***Commentary:*** *Demand levels may be taken as the peak demand from the equivalent static wind load analysis in a linear elastic assessment or from the controlling wind direction observed in the nonlinear time history analyses.*

No yielding is permitted, and story stability and column force transfer shall be maintained. The following elements and associated actions shall be treated as force-controlled for this performance objective:

- MWFRS moment frame column axial compression and shear,
- MWFRS shear wall shear,
- MWFRS joint shear or panel shear in concrete,
- MWFRS column buckling,
- MWFRS column shear,
- Diaphragms providing stability to the MWFRS,
- Diaphragm or slab punching shear,
- Gravity beam shear,
- Gravity connections,
- Basement wall in-plane demands induced by the MWFRS, and
- Shear demands within deep or shallow foundations.

> ***Commentary:*** *For further details on force-controlled elements, refer to ASCE 41.*

7.4.3.2 Deformation-controlled elements and actions

Calculated demand to capacity ratios for deformation-controlled elements shall not exceed 1.25, where demand is calculated per provisions in Chapter 6, and the capacity is calculated as follows:

1. For reinforced concrete elements, the capacity is the expected strength in accordance with ACI 318, with the phi-factor taken as 1.0.
2. For structural steel, composite steel, and concrete elements, the capacity is the expected strength in accordance with AISC 341 and AISC 360, with the phi-factor taken as 1.0.

> ***Commentary:*** *Expected material properties should be based on mean values of tested material properties, with the phi-factor taken as 1.0.*

If demand to capacity ratios are found to be in excess of 1.0, a nonlinear time history analysis is required to quantify the true response of the deformation-controlled elements.

Calculated deformations shall be less than those that result in damage that

1. Exceeds deformation limits in predefined deformation-controlled structural elements,

2. Impairs the ability of the structure to maintain story stability or carry gravity loads, and

3. Results in permanent deformation that exceeds the stated residual drift criteria.

If the ultimate deformation capacity associated with any mode of deformation in an element is exceeded in any of the response history analyses, it is permitted to evaluate the stability of the structure assuming no strength contribution from these elements.

While maintaining the ability to resist gravity load, minor yielding is permitted within the following elements and associated actions:

- MWFRS link beams,
- MWFRS bracing,
- MWFRS shear wall flexure,
- MWFRS panel shear in structural steel joints,
- MWFRS column tension,
- MWFRS column flexure,
- Outrigger beams and trusses,
- Gravity beam flexure,
- Gravity system slab flexure,
- Moment frame beam hinge zone flexure, and
- Shear wall hinge zone axial and flexure (yielding in tension is permitted but no yielding in compression is permitted in absence of special detailing).

Deformation limits for deformation-controlled elements and actions shall be limited to prevent low cycle fatigue failure.

***Commentary:** Inelastic strain at 1.5 times section yield should be limited to approximately 10 cycles. Higher inelastic strains or number of inelastic cycles may be shown acceptable through testing. Higher number of lesser magnitude inelastic cycles are acceptable if shown through calculations or testing to result in stable behavior. Recognized methods* *such as the Coffin Manson relationship (Coffin 1954) may be one approach to justify adequate performance.*

It is recommended that deformation limits are agreed to between all project stakeholders and the project peer review team prior to conducting structural analysis.

7.4.3.3 Other deformation-controlled elements and actions

The list of deformation-controlled elements in Section 7.4.3.2 is not exhaustive; ASCE 41 shall be reviewed for additional deformation-controlled elements and actions. The following elements and associated actions are permitted to exhibit nonlinear response in a response history analysis:

- Foundation uplift,
- Deep foundation axial and flexure, and
- Supplementary damping systems.

Commentary: *Further research is required to establish deformation limits for elements beyond those listed here. The limits given have been formed from engineering experience and judgment. Departure from these limits is acceptable if the limits are consensually agreed upon by all project stakeholders and the peer review committee or they are established by experimentally obtained response characteristics of a subassembly.*

7.4.3.4 Minimum strength for Method 1 design

The MWFRS shall be designed so that the calculated demand to capacity ratio for deformation controlled elements shall not exceed 1.25, where demand is calculated per the static wind loads prescribed in ASCE7-16 Directional Procedure, and the capacity is calculated as follows:

1. For reinforced concrete elements, the capacity is the expected strength in accordance with ACI 318 with the phi-factor according to ACI 318.
2. For structural steel, composite steel, and concrete elements, the capacity is the expected strength in accordance with AISC 341 and AISC 360, with the phi-factor according to AISC 341 and AISC 360.

Commentary: *Wind demand for minimum strength evaluation is based upon the MWFRS loads established by the procedures in Chapter 27 of ASCE 7.*

7.4.4 Strength Limits and Acceptance Criteria for Method 2 and Method 3

7.4.4.1 Acceptance Criteria for MWFRS Capacity

Method 2 and Method 3 analysis shall utilize MWFRS elements with sufficient capacity under the number of peak loading cycles to provide acceptable structural response.

> ***Commentary:*** *The objective of incipient collapse assessment is to achieve a post-yield state that does not result in structural failure. It is anticipated that incipient collapse will be evaluated by means of a nonlinear time history analysis of the structure. The structural design and element detailing provisions must be chosen such that the analysis captures the change in building period, and therefore wind load effect, as a result of nonlinearity in the system. Further research is necessary, but as a guide, the highest excursion for a deformation-controlled element should not exceed 50% of the permissible acceptance criterion given in the life safety performance objective of ASCE 41.*

7.4.4.2 Minimum strength for Method 2 and Method 3 design

The MWFRS shall be designed so that the calculated demand to capacity ratio for deformation controlled elements shall not exceed 1.5, where demand is calculated per the static wind loads prescribed in ASCE7-16 Directional Procedure, and the capacity is calculated as follows:

1. For reinforced concrete elements, the capacity is the expected strength in accordance with ACI 318 with the phi-factor according to ACI 318.

2. For structural steel, composite steel, and concrete elements, the capacity is the expected strength in accordance with AISC 341 and AISC 360, with the phi-factor according to AISC 341 and AISC 360.

> ***Commentary:*** *Wind demand for minimum strength evaluation is based upon the MWFRS loads established by the procedures in Chapter 27 of ASCE 7.*

7.4.5 Acceptance Criteria for MWFRS System Reliability

Reliability analysis shall be employed to demonstrate satisfactory MWFRS element and system response for Methods 2 and 3 as follows:

- The design team shall demonstrate that MWFRS linear elastic elements and connections comply with the appropriate target reliability in ASCE 7 Table 1.3.1a.

- When nonlinear time history analysis methods are used, the design team shall demonstrate that the MWFRS system reliability for incipient collapse is acceptable for a target probability of failure of 0.01% that is caused by a design wind event for continuous occupancy in Table 4-1 and a lognormal distribution for strength parameters.

Commentary: *Appendix B presents a conditional reliability approach for Method 2 to evaluate structural performance for a given wind scenario and is based upon FEMA P-695 studies (FEMA 2009). Appendix C offers additional guidance on alternative approaches for system reliability analysis for Method 3.*

System reliability analysis may use Monte Carlo simulations in which all significant parameters are taken as random variables with parameter distribution values consistent with laboratory testing, analytical data, and engineering judgment to evaluate the reliability requirements in Section 7.4.5.

The peer review team should include an individual well versed in reliability theory where the design incorporates a reliability investigation. The conditional system reliability goals were developed to be compatible with those contained in ASCE 7 Chapter 1. Elements that comply with the LRFD criteria of ASCE 7 and the companion industry design standards may be deemed to comply with the criterion.

7.5 EXCEEDANCE OF ACCEPTANCE CRITERIA

When acceptance criteria for MWFRS response are exceeded, one or more of the following actions shall be implemented:

1. The building design shall be modified.

2. If the performance objectives and associated acceptance criteria are modified, any modifications shall be approved by the peer review team and AHJ.

Commentary: *Effective strategies to reduce the dynamic wind-induced response of a building include aerodynamic treatment (or optimization of the architectural form), structural refinement through alteration of the structural properties of the building (including mass and/or stiffness), or through the implementation of supplementary damping. A combination of all three control mechanisms is also effective.*

For guidance on effective strategies to reduce the dynamic wind-induced response of a building, see Appendix D.

REFERENCED STANDARDS

ACI 318. *Building code requirements for structural concrete and commentary*. ACI 318-14. American Concrete Institute, 2014.

AISC 341. *Seismic provisions for structural steel buildings*. ANSI/AISC 341-16. American Institute of Steel Construction, 2016.

AISC 360. *Specification for structural steel buildings.* ANSI/AISC 360-16. American Institute of Steel Construction, 2016.

ASCE 7. *Minimum design loads and associated criteria for buildings and other structures*. ASCE/SEI 7-16. ASCE, 2017.

ASCE 41. *Seismic rehabilitation of existing buildings*. ASCE/SEI 41-17. ASCE, 2017.

REFERENCES

ASCE. 2017. *Seismic rehabilitation of existing buildings.* ASCE/SEI 41-17. Reston, VA: ASCE.

Aswegan, K., F. Charney, and J. Jarrett. 2015. "Recommended procedures for damage based serviceability design of steel buildings under wind load." *Eng. J. 52*, 1−26.

ATC (Applied Technology Council). 2019. *ATC design guide 3, Serviceability design of tall buildings under wind load*. Redwood City, CA.

AIJ (Architectural Institute of Japan). 2004. *Guidelines for the evaluation of habitability to building vibration*. AIJ-GEH-2004. Tokyo: Architectural Institue of Japan.

Charney, F. A. 1990. "Wind drift serviceability limit state design of multistory buildings." *J. Wind Eng. Ind. Aerod. 36* (Part 1), 203−212.

Denoon, R. O., R. D. Roberts, C. W. Letchford, and K. C. Kwok. 2000. *Field experiments to investigate occupant perception and tolerance of wind-induced building motion*. Rep. No. R804. Sydney: Univ. of Sydney, Dept. of Civil Engineering.

Coffin, L. F. 1954. "A study of the effects of cyclic thermal stresses on a ductile metal." *Trans. ASME*, *76*, 931−950.

FEMA. 2005. *Improvement of nonlinear static seismic analysis procedures*. P-440. Washington, DC: FEMA.

Griffis, L. G. 1993. "Serviceability limit states under wind loads." *Eng. J. 30* (1), 1−16.

ISO (International Organization for Standardization). 1984. *Guidelines for the evaluation of the response of occupants of fixed structures, to low-frequency horizontal motion (0.063 to 1 Hz)*. ISO6897. Geneva: International Organization for Standardization.

ISO. 2007. *Bases for design of structures: Serviceability of buildings and walkways against vibrations*. ISO10137. Geneva: International Organization for Standardization.

Chapter 8. Acceptance Criteria: Building Envelope Systems Criteria

8.1 SCOPE

This chapter describes the performance acceptance criteria for the building envelope components and attached elements. This includes design parameters as well as construction processes to ensure the final assemblies meet the specified performance. This chapter includes the following building envelope systems:

- Roofs,
- Walls and fenestration, including doors, and
- Other architectural features such as but not limited to sun shades, signage, solar panels and exterior equipment.

8.2 CONSEQUENCE ESTIMATION

The requirements of this chapter include both standard and enhanced design criteria. The building types that shall be subject to the mandatory inclusion of the enhanced criteria described within are (1) Risk Category III or IV buildings and (2) buildings in hurricane-prone regions.

The requirements for mandatory enhanced criteria are minimum requirements. The owner and project designers shall weigh the benefits of the inclusion of the enhanced criteria for buildings that fall outside of the minimum criteria.

8.3 ACCEPTANCE CRITERIA

The requirements of this section define the acceptance criteria for and relating to enclosure system performance owing to wind pressure, wind-borne debris, and wind-driven rain. The requirements provided in this section do not include, nor are they intended to override other requirements, by the governing building code or the AHJ for each enclosure system or component.

8.3.1 Roof Systems

This section addresses low- and steep-slope roof systems. Section 8.3.1.2 addresses buildings located outside of hurricane-prone regions. Section 8.3.1.3 addresses buildings located in hurricane-prone regions.

> ***Commentary:*** *Risk Category: With respect to the roof system, a Risk Category II building could be designed using Risk Category III or IV wind load criteria, and a Risk Category III building could be designed using Risk Category IV wind load criteria. However, in lieu of spending money for the cost associated with the increased wind loads, more reliable roof*

system wind performance is likely to be achieved by allocating money for a professional roofing contractor to install the system and by allocating money for increased observation and testing during roof system application.

8.3.1.1 General industry standards, requirements, and guidelines

The wind performance requirements in Chapter 15 of the current edition of the IBC are considered baseline criteria. Sections 8.3.1.2 and 8.3.1.3 provide additional criteria to achieve more reliable wind performance.

Commentary: *IBC Chapter 15 provides roof system design criteria that pertain to issues unrelated to wind performance; these are also considered baseline criteria. The National Roofing Contractors Association (NRCA) publishes several technical documents that provide general roof design recommendations, available at* http://www.nrca.net/Technical/.

If the building will be insured by FM Global, also follow the applicable Property Loss Prevention Data Sheets, available at https://www.fmglobal.com/research-and-resources/fm-global-data-sheets.

8.3.1.2 Buildings located outside of hurricane-prone regions

This section pertains to buildings located outside of hurricane-prone regions (as defined in ASCE 7). To achieve enhanced wind performance of roof systems located outside of hurricane-prone regions, the following items shall be incorporated:

Commentary: *All the recommendations in this section exceed requirements in the 2018 edition of the IBC.*

- Specify roof systems that storm damage research has shown to offer reliable wind performance.

Commentary: *Asphalt shingles are commonly used on residences but are also used on low-rise critical facilities. Storm damage research has shown that asphalt shingles have low wind performance reliability and that other types of steep-slope roof coverings generally offer more reliable wind performance.*

- If the roof deck is concrete, specify a modified bitumen membrane (or other suitable membrane) that is torch-applied to a primed deck, followed by installation of roof insulation and the primary membrane.

Commentary: *The membrane that is applied to the concrete protects the roof insulation from moisture migrating from the deck, as well as, serves as a secondary membrane if the primary membrane is breached by wind-borne debris. Roof system puncture by wind-borne debris is uncommon outside of hurricane-prone regions.*

- If the roof deck is not concrete, a secondary membrane shall be specified if it is desired to have enhanced protection from water leakage in the event the roof covering is punctured by wind-borne debris.

  Commentary: *Recommendations for secondary membranes over other deck types are given in Section 6.3.3.7 in FEMA P-424,* Design Guide for Improving School Safety in Earthquakes, Floods, and High Winds *(FEMA 2010), available at* https://www.fema.gov/library/viewRecord.do?id=1986.

- For roof heights greater than 200 ft with an adhered roof membrane, specify that the roof insulation be attached with mechanical fasteners, in lieu of or in addition to adhesive. Also specify a perimeter roof membrane restraint system.

  Commentary: *Adhered systems sometimes have reduced uplift resistance due to workmanship deficiencies or strength degradation because of weathering or roof leakage. The use of mechanical fasteners is intended to provide more reliable wind uplift performance. The 200 ft roof height is based on judgment, which considers increased uplift pressures with increased height and considers the ramifications of wind-borne roof debris blown from great roof heights. For even greater avoidance of wind-borne roof debris, a hybrid approach is to adhere the membrane and also mechanically attach it as discussed in the Section 8.3.1.3 Commentary.*

  A perimeter restraint system such as a peel-stop (Figure 8-1), is intended to prevent progressive lifting and peeling of the roof membrane in the event the base flashing, coping, or edge flashing detaches. However, currently a test method to evaluate peel-stop effectiveness does not exist. Peel-stop design guidance is given in FEMA P-424 (FEMA 2010).

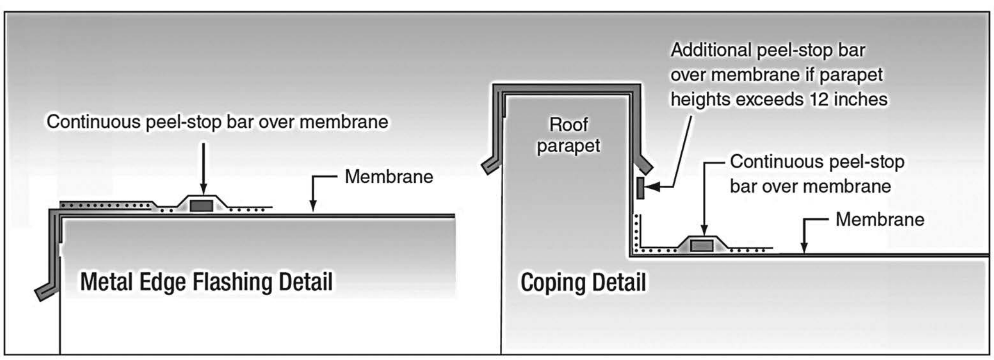


Figure 8-1. Continuous peel-stop bard details.
Source: FEMA (2010).

- If a gutter is specified, specify that uplift resistance be evaluated in accordance with laboratory test method ANSI/SPRI GT-1 (SPRI 2016), available at https://www.spri.org/wpfb-file/ansi-spri-gt-1-2016-test-standard-for-gutter-systems-pdf/.

  Commentary: *The IBC currently does not have gutter wind-resistance criteria.*

- If concrete roof pavers are specified, the wind load and resistance design shall be in accordance with the procedure given in "Concrete Roof Pavers: Wind Uplift Aerodynamic Mechanisms and Design Guidelines—A Proposed Addition to ANSI/SPRI RP-4" (Mooneghi et al. 2017), available at http://rci-online.org/wp-content/uploads/2017-cts-mooneghi-smith-irwin-chowdhury.pdf.

  Commentary: *ANSI/SPRI RP-4 is referenced in the IBC. It only provides paver wind design criteria for roof heights up to 150 ft. The paver design criteria given in the referenced paper is suitable for any roof height. The criteria in the paper are based on large scale wind tunnel testing; the criteria are judged to provide more reliable paver wind performance than the criteria in the 2013 edition of ANSI/SPRI RP-4.*

- Specify increased quality assurance observation during roof system application.

  Commentary: *Roof system blow-off is often caused by workmanship deficiencies, which are often difficult to detect after installation. Quality assurance observation during application is vital to achieve reliable wind performance.*

- If an adhered membrane system is specified, specify field uplift testing per Section 8.5.2.3.1.

  Commentary: *Field uplift test methods only currently exist for asphalt shingles, tile and adhered membrane systems.*

8.3.1.3 Buildings located in hurricane-prone regions

This section pertains to buildings located in hurricane-prone regions.

> ***Commentary:*** *Achieving reliable roof system wind performance in hurricane-prone regions is challenging because of the quantity and momentum of wind-borne debris that is often generated and because some hurricanes are long-duration events that may cause fatigue of roof system components. The 2018 edition of IBC does not address roof system puncture by wind-borne debris and subsequent interior water leakage, nor does it address roof system fatigue. Special criteria for achieving reliable roof system performance are presented in this section.*

The following performance objectives shall be considered:

> ***Commentary:*** *Standardized test methods and other criteria for demonstrating that the performance objectives can be met do not exist. Accordingly, guidance for meeting each of the performance objectives is given in the Commentary.*

- Rainwater does not leak into the building's interior.

  Commentary: *Interior water leakage is commonly caused by puncture of the roof covering by wind-borne debris. A variety of debris types and momentum commonly occurs. The following testing is recommended to evaluate the ability of a roof assembly to prevent water leakage into the building's interior:*

- *Test missile: A 15 lb sawn lumber 2 × 4 is judged to be a suitable proxy for light weight and some medium weight wind-borne debris. This is the test missile that is specified in ICC 500, "Standard for the Design and Construction of Storm Shelters" (ICC 2014). For hurricane shelters, ICC 500 requires the speed of the test missile impacting perpendicular to vertical surfaces to be a minimum of 0.50 times the basic wind speed. This speed is judged to be suitable for testing roof assemblies when the test missile is traveling at an angle of 20 degrees or less with respect to a horizontal roof surface (i.e., this angle represents debris that is primarily traveling horizontally).*

  ICC 500 requires the speed of the test missile impacting perpendicular to horizontal surfaces to be 0.10 times the basic wind speed. This speed is judged to be suitable for testing roof assemblies when the test missile is traveling perpendicular to a horizontal roof surface (i.e., this angle represents debris that is primarily traveling vertically).

  Roofs can be impacted by debris with much greater momentum than imparted by the aforementioned test missiles. Examples include large HVAC units and steel roof decking blown from upper-level roofs. In cases where the roof can be impacted by greater momentum debris, test missile speeds using ICC 500 criteria for tornado shelters (Table 305.1.1) should be considered.

- *Test specimens: Construct two test specimens, each 10 × 10 ft minimum. The specimens should include all components of the roof assembly, including the deck. The deck support spacing should be representative of design conditions. Each specimen should include at least one plumbing vent. One specimen should be impacted by one test missile that travels perpendicular to the specimen. This specimen is permitted to be placed vertically for the test missile to travel horizontally. The other specimen should be impacted by one test missile that travels at an angle of 20 degrees from the specimen. This specimen is permitted to be placed at an angle to allow the test missile to travel horizontally.*

- *Impact testing: Follow the procedure specified in ICC 500. The test missile should impact the test specimen near the center of the specimen. If the roof is surfaced with pavers and if the pavers are judged to be susceptible to blow-off after impact testing, the test is considered to have failed. See the following Commentary regarding brittle roof coverings.*

- *Water testing: After the two specimens are impacted, place the specimens in a horizontal position and flood the specimens with water. The water depth should be a minimum of 1 in. above the roof surface. Run the water test for a minimum of 72 h. After 24 and 48 h, check the water depth; add water to maintain 1 in. above the roof surface. After 24, 48 and 72 h, check the underside of the deck for water leakage. If leakage occurs, the testing is considered to have failed.*

- *Guidance for roof assembly design: Section 6.3.3.7 in FEMA P-424,* Design Guide for Improving School Safety in Earthquakes, Floods, and High Winds *(FEMA 2010), provides guidance on roof assembly designs that are intended to avoid leakage after being impacted by wind-borne debris. This guidance is available at https://www.fema.gov/library/viewRecord.do?id=1986.*

- *Hip and ridge closures at metal roof panels: In addition, interior water leakage is often caused by inadequate wind-driven rain resistance of closures at hips and ridges. Enhanced resistance can be achieved by installing two or three rows of closures (depending on rain demand). The inner closure is sealed all around the closure. The outer closure is sealed at the top and sides but is unsealed at the pan to allow drainage.*

- Wind-borne debris from the roof shall not be shed from the building.

Commentary: *Wind-borne debris shedding from the roof can damage the building's facade or lower-level roofs. Debris can either be from the roof itself or it can be items such as loose access panels on rooftop equipment, as well as various types of debris that were left on the roof before a hurricane. Debris shedding is particularly problematic at hospitals and hurricane shelters, wherein people approaching the building during a hurricane are vulnerable to injury from the debris. To avoid debris shedding, the following are recommended.*

- *Do not specify brittle roof coverings such as lightweight pavers, slate, or tile unless testing in accordance with the previously discussed Commentary indicates that the covering is not susceptible to blow-off after impact testing. These types of roof coverings can be easily broken by low-momentum wind-borne debris. Once broken, these types of roof coverings are susceptible to shedding a significant amount of debris.*

- *Specify a tall parapet. Storm damage research has shown that tall parapets often prevent debris from blowing off a roof. An appropriate parapet height likely depends on the basic wind speed, exposure, and roof height. Laboratory research has not been conducted to provide design criteria for the parapet height needed to retain debris on the roof. However, parapet heights in the range from 3 to 6 ft are likely to be effective, depending on the basic wind speed, exposure, and roof height.*

To protect the parapet base flashing from debris damage, it is recommended that the base flashing be armored. Armoring guidance is provided in FEMA P-424.

- Progressive lifting and peeling of the roof system shall be limited in the event of localized failure.

Commentary: *Roof system blow-off is often initiated by failure of edge flashings and copings. Typically, initial failure is typically due to inadequate attention to design or installation of the nailers or the edge flashing/coping, or due to nailer deterioration. Failure of a single length of nailer or edge flashing/coping can result in blow-off of a large section of the roof covering. Peel-stops can avoid progressive failure, as discussed in previous Commentary.*

Roof system blow-off can also be initiated by lifting of a small area of the roof, which can propagate. Limiting progressive lifting and peeling is difficult or impractical for many types of roof systems. However, adhered membrane systems can be designed to limit progressive failure if they become detached. A hybrid approach is to adhere the membrane and mechanically attach it with rows of mechanical fasteners as well. The fastener row spacing and spacing of fasteners along the rows is designed to accommodate the design wind uplift load. With this redundant approach, the membrane is not susceptible to fluttering. More important, If the adhered system becomes detached because of workmanship *deficiencies or strength reduction due to roof leakage, the uplift load will be transferred to the rows of mechanical fasteners.*

See Section 8.3.1.2 regarding specifying roof systems that storm damage research has shown to offer reliable wind performance, gutters, pavers, quality assurance observations, and field uplift testing.

8.3.2 Wall Cladding Systems

Wall cladding systems encompass components of different materials, each with design requirements outlined by standards and additional referenced standards pertaining to that material. Wall cladding systems shall meet the minimum design criteria outlined in ASCE Standard 7. Wall cladding systems shall be designed by a qualified registered professional.

8.3.2.1 Industry standards and requirements

Standards governing common materials used in wall cladding systems include but are not limited to the following:

Steel:

American Institute of Steel Construction (AISC): AISC 360—Specification for Structural Steel Buildings (AISC 2016b), AISC 303—Code of Standard Practice for Steel Buildings and Bridges (AISC 2016a)

Commentary: *Additional ASTM standards may apply depending on the type of system, how the steel is utilized in the system, and fabrication method of the steel.*

Aluminum:

Aluminum Association (AA): AA ADM-1 Aluminum Standards and Data: AA ADM-1 Aluminum Design Manual (AA 2015)

Commentary: *Additional ASTM standards may apply depending on the aluminum application and fabrication method. Coordinate the ADM edition with the project requirements as there are discrepancies between editions.*

Fasteners:

AAMA TIR-A9—Design Guide for Metal Cladding Fasteners (AAMA 2014b)

Commentary: *Refer to specific ASTM standards for bolts and nuts based on specific types and materials.*

Joint Sealants:

ASTM standards: ASTM C1193—Standard Guide for Use of Joint Sealants

Commentary: *Elements that connect wall systems to adjacent systems should be considered. Compatibility between the sealant and adjacent building materials is a main consideration. Sealant used as part of structural sealant glazing systems shall refer to ASTM C1087 and ASTM C1401.*

Brick Masonry:

American Concrete Institute (ACI): ACI 530.1—Building Code Requirements and Specification for Masonry Structures.

ASTM standards: ASTM C216—Standard Specification for Facing Brick (Solid Masonry Units Made from Clay or Shale); ASTM C1088—Standard Specification for Thin Veneer Brick Units Made from Clay or Shale.

Brick Industry Association (BIA): Technical Note 28B—Brick Veneer / Steel Stud Walls

Precast Concrete, Glass Fiber Reinforced Concrete:

American Concrete Institute (ACI): ACI SP-224—Thin Reinforced Cement-Based Products and Construction Systems

Precast/Prestressed Concrete Institute (PCI): MNL 128-01—Recommended Practice for Glass Fiber Reinforced Concrete Panels, MNL 120, PCI Design Handbook—Precast and Prestressed Concrete.

Concrete Masonry Units:

American Concrete Institute (ACI): ACI 530.1—Building Code Requirements and Specification for Masonry Structures.

ASTM standards: ASTM C90—Standard Specification for Loadbearing Concrete Masonry Units

Stone Panels:

ASTM standards: ASTM C1242—Standard Guide for Selection, Design, and Installation of Dimension Stone Attachment Systems, ASTM C1528—Standard Guide for Selection of Dimension Stone

Indiana Limestone Institute of America (ILI): ILI Indiana Limestone Handbook (ILI 2007)

Marble Institute of America (MIA): MIA Dimension Stone Design Manual (NSA 2016)

National Building Granite Quarries Association (NBGQA): NBGQA Specification for Architectural Granite (NBGQA v18-1)

Commentary: *Various ASTM standards exist for both the general design of stone panels, as well as those specific for different types of stone.*

Exterior Insulation and Finish Systems (EIFS):

ASTM E2568-17a *Standard Specification for PB Exterior Insulation and Finish Systems* (ASTM 2017b).

8.3.2.2 Structural performance

Wall cladding systems shall be designed per the applicable industry standards and requirements for that system and shall transmit design loads to the building structure via the points of attachment to the structure. There should be no deformation or damage under design loads that is detrimental to the intended performance of the system components, adjacent elements, or supporting structure under the Continuous Occupancy performance objective for the risk category of building. Equivalent ASD wind loads may be used for components where ASD design criteria is provided in industry standards.

> ***Commentary:*** *The designer of the wall systems should select systems that can accommodate the prescribed loading. Note that not all wall cladding system types will be capable of meeting these requirements.*

Enhanced criteria: Wall cladding assemblies for buildings shall meet the requirements of ASTM E1996-17—*Standard Specification for Performance of Exterior Windows, Curtain Walls, Doors, and Impact Protective Systems Impacted by Windborne Debris in Hurricanes* (ASTM 2017d)

> ***Commentary:*** *Note that this prestandard adopts a fenestration impact standard for opaque wall assemblies.*

8.3.2.3 Movement performance

Wall cladding systems shall be designed to accommodate the expected movement of their supporting framing members under the Continuous Occupancy performance objective for the risk category of building. Movements shall not structurally disengage, impose stresses, or degrade the waterproofing elements that seal the wall system or provide a seal to an adjacent building envelope system.

8.3.2.4 Weatherproofing performance

Wall cladding shall be designed to prevent water infiltration through the system at wind pressures assessed for the appropriate MRI for the selected risk category operational objective when tested to AAMA 520—*Voluntary Specification for Rating the Severe Wind-driven Rain Resistance of Windows, Doors and Unit Skylights* (AAMA 2012b). In lieu of the performance levels provided in the AAMA 520 standard, the following parameters shall be used for testing:

- Upper limit pressure: Inward acting wind pressures assessed for the appropriate MRI for the selected risk category Operational performance objective for the risk category of building.

Wind pressures shall be in ASD equivalent pressures and shall not include a tributary area reduction. The minimum allowed pressure shall be 766 Pa (16 psf).

- Lower limit pressure: 1/3 of the upper limit pressure
- Number of pulsating pressure cycles: 300
- Allowable water penetration: 15 ml

Walls breached by impact shall not be subject to these criteria.

Commentary: *Refer to Chapter 5 for the analysis procedure to determine wind pressures for a specific MRI. The criteria of the MRI for the operational objective may result in test pressures that exceed what wall cladding assemblies have historically been tested to. Over the previous decade, failures have been observed that have led to the development of this more onerous criterion. Note that this standard adopts a fenestration testing standard as a test method to be used for opaque wall assemblies.*

The objective is that the envelope retains weather resistance in most places and thus does not experience progressive tear off, separation, or other similar failure that exposes the internal spaces of the structure to wind pressures or rain. Operable units may lose water penetration resistance due to pressure driven rain at the continuous occupancy limited interruption objective. The intent is that clean up and temporary repair can be achieved quickly after the storm subsides.

The intent of these criteria is to limit water intrusion. Water intrusion would be unavoidable if the envelope is breached due to debris impact. This prestandard does not change the performance objective of impact testing or resistance. Hence, one must recognize that impact can puncture the envelope locally while the envelope system away from impacts continues to resist wind and water entry.

Some wall systems are designed as barrier systems and rely on face seals, not a rainscreen principle, to provide waterproofing performance. The use of these systems can both limit the waterproofing performance, as well as increase the reliance on installation quality. The designer of the wall system should evaluate the appropriateness of a barrier system based on the anticipated wind loads and the ability of the face seals to be maintained or replaced over the life of the wall system.

8.3.3 Fenestration Systems

Fenestration systems encompass components of different materials, each with design requirements outlined by standards, and additional referenced standards pertaining to that material. Fenestration systems shall meet the minimum design criteria outlined in ASCE 7. Fenestration systems shall be designed by a qualified registered professional.

8.3.3.1 Industry standards and requirements

Standards governing common materials used in fenestration systems include but are not limited to the following:

Steel:

American Iron and Steel Institute: AISC 360—*Specification for Structural Steel Buildings* (AISC 2016b), AISC 303—*Code of Standard Practice for Steel Buildings and Bridges* (AISC 2016a)

> ***Commentary:*** *Additional ASTM standards may apply depending on the type of system, how the steel is utilized in the system, and fabrication method of the steel.*

Aluminum:

Aluminum Association (AA) Aluminum Standards and Data: AA ADM-1 *Aluminum Design Manual* (AA 2015)

> ***Commentary:*** *Additional ASTM standards may apply depending on the aluminum application and fabrication method. Coordinate the ADM edition with the project requirements, as there are discrepancies between editions.*

Fasteners:

AAMA TIR-A9—*Design Guide for Metal Cladding Fasteners* (AAMA 2014b)

> ***Commentary:*** *Refer to specific ASTM standards for bolts and nuts based on specific types and materials.*

Glazing:

ASTM standards: ASTM E1300—*Standard Practice for Determining Load Resistance of Glass in Buildings* (ASTM 2016b)

Glass Association of North America (GANA): *Glazing Manual* (GANA 2008)

> ***Commentary:*** *Additional specific standards will apply depending on glazing type, application, safety requirements, and attachment method. For example, ASTM C1401 (ASTM 2014a) provides guidelines for structural sealant glazing specifically.*

Joint Sealants:

ASTM standards: ASTM C1193—*Standard Guide for Use of Joint Sealants* (ASTM 2016a)

> ***Commentary:*** *Elements that connect fenestration systems to adjacent systems shall be considered. Compatibility between the sealant and adjacent building materials is a main consideration. Sealant used as part of structural sealant glazing systems shall refer to ASTM C1087(ASTM 2016e) and ASTM C1401 (2014a).*

PVC:

UV-stabilized PVC complying with AAMA/WDMA/CSA 101/I.S.2/A440 (AAMA/WDMA/CSA 2017).

> ***Commentary:*** *Additional specific standards will apply. For example, ASTM D4216* Standard Specification for Rigid PVC and Related Plastic Building Product Compounds *(ASTM 2017d) provides minimum values and ranges for mechanical properties. ASTM D4726* Standard Specification for Rigid PVC Exterior Profiles Used for Assembled Windows and Doors *(ASTM 2018d) specifies extrusion quality.*

Fenestration systems as appropriate for the selected system:

- AAMA Vol. 1—*Windows and Doors*
- AAMA WSG—*Window and Door Selection Guide* (AAMA 2011)
- AAMA CW-DG-1—*Aluminum Curtain Wall Design Guide Manual* (AAMA 2005)
- AAMA MCW-1—*Metal Curtain Wall Manual* (AAMA 2003)
- AAMA SFM1—*Aluminum Storefront and Entrance Manual* (AAMA 2014a)
- AAMA IPCB—*Standard Practice for the Installation of Windows and Doors in Commercial Buildings* (AAMA 2008)

8.3.3.2 Glazing for fenestration

Glazing design for fenestration systems shall include appropriate structural, movement, deflection, safety, and other relevant performance requirements to meet overall fenestration system requirements.

Glazing structural design shall follow ASTM E1300 (ASTM 2016b) and referenced standards for glass.

> ***Commentary:*** *Structural properties of glazing products are governed by multiple ASTM standards including but not limited to ASTM C1036 (ASTM 2016c), C1048 (ASTM 2018c), and C1172 (ASTM 2019a) by reference to ASTM E1300.*

Glazing shall be designed per ASTM E1300. Glazing shall resist and transmit loads to the supporting framing of the system it is part of and accommodate their movements. These loads include thermal stress, distributed loads from wind, occupancy (live) loads, and impact loads from debris.

Glazing systems shall comply with the requirements of ASTM E1886-13a (ASTM 2013).

Enhanced criteria: Glazing systems shall comply with the requirements of ASTM E1886-13a for “enhanced protection.”

Commentary: *There may be additional project specific requirements such as blast resistance or forced entry / ballistic resistance that govern and need to be accounted for and will supersede wind load demands for glazing. Reference the appropriate ASTM standards for specific project requirements.*

This prestandard does not address potential aesthetic issues of glass deflections. Manufacturer recommended deflection limits for insulated glazing units will depend on the seal type and unit size. The deflection performance requirements will correspond to the requirements of the framing system of which it is part. Additional consideration may be required if a specific project has additional requirements such as safety glazing or blast resistance, which may have overriding failure criteria.

8.3.3.3 Structural performance

Fenestration shall be designed as per the applicable industry standards and requirements for that system and shall transmit design loads to the building structure via the points of attachment to the structure. There shall be no deformation or damage under design loads that is detrimental to the intended performance of the system components, adjacent elements, or supporting structure under the Continuous Occupancy performance objective for the risk category of building. Equivalent ASD wind loads may be used for components where ASD design criteria is provided in industry standards.

Commentary: *The designer of fenestration systems should select systems that can accommodate the prescribed loading. Note that not all fenestration system types will be capable of meeting these requirements.*

8.3.3.4 Movement performance

Glazing units shall be designed to accommodate the expected movement of their supporting framing members under the Continuous Occupancy performance objective for the risk category of building. Movements shall not structurally disengage, impose stresses, or degrade the waterproofing elements that seal to the glazing units or adjacent building envelope systems under the wind loads.

8.3.3.5 Weatherproofing performance

Fenestration shall be designed to prevent water infiltration through the system at wind pressures assessed for the appropriate MRI for the selected risk category Operational performance objective when tested to AAMA 520—*Voluntary Specification for Rating the Severe Wind-driven Rain Resistance of Windows, Doors and Unit Skylights* (2012b). In lieu of the performance levels provided in the AAMA 520 standard, the following parameters shall be used for testing:

- Upper limit pressure: Inward acting wind pressures assessed for the appropriate MRI for the selected risk category Operational performance objective for the risk category of building.

Wind pressures shall be in ASD equivalent pressures and shall not include a tributary area reduction. The minimum allowed pressure shall be 766 Pa (16 psf).

- Lower limit pressure: 1/3 of the upper limit pressure
- Number of pulsating pressure cycles: 300
- Allowable water penetration: 15 ml

Fenestration breached by impact shall not be subject to these criteria.

Commentary: *Refer to Chapter 5 for the analysis procedure to determine wind pressures for a specific MRI. The criteria of the MRI for the Operational performance objective may result in test pressures that exceed what wall assemblies have historically been tested to. Over the previous decade, failures have been observed that have led to the development of this more onerous criterion.*

The objective is that the envelope retains weather resistance in most places and thus does not experience seal failure, separation, or other similar failure that exposes the internal spaces of the structure to wind pressures or rain. Operable units may lose water penetration resistance because of pressure driven rain at the Continuous Occupancy performance objective. Intent is that clean up and temporary repair can be achieved quickly after the storm subsides.

The intent of these criteria is to limit water intrusion. Water intrusion would be unavoidable if the envelope is breached due to impact. This prestandard does not change the performance objective of impact testing or resistance. Hence, one must recognize that impact can puncture the envelope locally while the envelope away from impacts continues to resist wind and water entry.

Many manufactured products are tested to a maximum pressure differential of 15 psf per AAMA 501.1 (AAMA 2017b). The application of this standard may result in increased performance criteria. Note that not all fenestration system types will be capable of meeting these requirements.

All barrier or face sealed fenestration systems shall include fully sealed sill pan flashing with sealed end dams. In addition, the waterproofing system of the adjacent walls shall be terminated into the fenestration system and flashed, leaving no discontinuities.

Commentary: *Some fenestration systems are designed as barrier systems and rely on face seals, not a rainscreen principle to provide waterproofing performance. Any water that bypasses the seal(s) of the system needs to be drained by a sill pan flashing to prevent water infiltration into the wall assembly below the window. The designer of the fenestration system should evaluate the appropriateness of a barrier system based on the anticipated wind loads and the ability of the face seals to be maintained or replaced over the life of the fenestration system.*

Entrance door systems that cannot meet the water infiltration performance criteria listed above shall be used in conjunction with a vestibule. Vestibules shall include a drain in the floor that will accommodate any leakage from the doors. Interior doors and walls of vestibules shall meet the same performance requirements of the exterior doors of the vestibule.

> ***Commentary:*** *Most entrance door systems are not provided by manufacturers with water and or air infiltration test data, custom project testing may be required to meet the criteria provided herein.*

8.4 INTERFACES AND PENETRATIONS FOR BUILDING SERVICES IN BUILDING ENCLOSURES

8.4.1 General

The designer of the building envelope systems shall coordinate the expected movements of the roof, wall, and fenestration systems with the primary building structure and provide compatible detailing. All waterproofing elements between building envelope systems, as well as penetrations through the building envelopes, shall be designed to accommodate movements. These elements shall also be designed so that water infiltration is not allowed through the system interfaces at wind pressures assessed for the appropriate MRI for the selected risk category operational objective when tested to AAMA 520—*Voluntary Specification for Rating the Severe Wind-driven Rain Resistance of Windows, Doors and Unit Skylights* (AAMA 2012b). In lieu of the performance levels provided in the AAMA 520 standard, the parameters specified herein for the adjacent wall or fenestration system shall be used for testing.

> ***Commentary:*** *Penetrations through the building envelope should include those for the building structure, mechanical, electrical, plumbing, and other building services. It can also include the supports for building services equipment, as well architectural features.*

8.4.2 Openings in the Building Envelope

Louvers, Packaged Terminal Air Conditioning Units (PTACS), and other openings through the building envelope systems shall be sealed to meet the general requirements of this section. Where openings are provided to allow for airflow into and out of the building, the connecting assemblies to these openings shall actively allow for the drainage of water that enters beyond the outer screen, hood, or louver. Active water management can include but is not limited to the following: (1) drains in ducted systems and (2) drains in plenum spaces.

Where drainage is used as an active water management system, the envelope system shall be capable of draining water at a rate equal to that which bypasses the outer screen / hood / shield of the system at wind pressures assessed for the appropriate MRI for the selected risk category operational objective when tested to AAMA 520—*Voluntary Specification for Rating the Severe Wind-driven Rain Resistance of Windows, Doors and Unit Skylights* (AAMA 2012b). In lieu of the performance levels provided in the AAMA 520 standard, the parameters specified herein for the adjacent wall or fenestration system shall be used for testing.

8.4.3 Rooftop Equipment

This section addresses rooftop equipment, including lightning protection systems (LPS) and solar panel arrays.

> ***Commentary:*** *Storm damage research has shown that rooftop equipment is often damaged during high windstorms and that the poor wind performance often causes a significant amount of interior water damage. Types of damage include*
>
> - *Equipment that has inadequate wind resistance [e.g., fan cowlings inadequately attached to fans, air conditioner / heating units that have inadequately attached sheet metal unit enclosures (cabinets), or access panels],*
> - *Equipment that is inadequately attached to curbs or stands,*
> - *Equipment curbs that are inadequately attached to the roof deck/structure,*
> - *LPS inadequately attached to the roof covering, or*
> - *Roof covering puncture by wind-borne equipment debris.*

To achieve enhanced wind performance of rooftop equipment, the following performance objectives are recommended:

> ***Commentary:*** *Compliance with some of the performance objectives can be demonstrated by calculations. However, standardized test methods need to be developed to demonstrate that other performance objectives can be met. In the absence of required test methods, guidance for meeting the performance objectives is given in the Commentary. All the recommendations in this section meet or exceed requirements in the 2018 edition of the IBC.*

- Design wind loads: As a minimum, comply with ASCE 7. See C8.3.1 for risk category selection.
- Factory-fabricated equipment shall be designed and manufactured by the equipment manufacturer to resist the wind design criteria specified by the designer. Documentation shall be provided by the equipment manufacturer to the designer, showing that the equipment has sufficient strength to meet the wind design criteria.
- Equipment curbs and stands, including their attachment to the roof deck/structure, shall be designed to resist the design wind loads induced by the equipment.
- The attachment of equipment to curbs and stands, and the attachment of LPS shall be designed to resist the design wind loads induced by the equipment.

> ***Commentary:*** *For design guidance on attaching various types of rooftop equipment, including LPS, see Attachment of Rooftop Equipment in High-Wind Regions, Recovery Advisory 2, March 2018 (FEMA 2018a): https://www.fema.gov/media-library-data/1522347818123-9fa4b38a90dc40b91fdb82eeb0307e98/USVI-RA2AttachmentofRooftopEquipmentinHigh-Wind_Regions_V2_508.pdf.*

- Rooftop relief air hoods shall account for wind-driven rain.

> ***Commentary:*** *One option to achieve this performance objective is to specify wall louvers in lieu of rooftop air hoods (see Section 8.4.2). Another is to design a drain sump pan to intercept water that is driven past the relief air hood. With this second option, in the absence of design criteria, judgment is needed to size the sump and drain.*

- Solar panel arrays shall be designed and installed to account for dynamic loading.

> ***Commentary:*** *Current test methods for solar panel arrays do not explicitly evaluate the influence of dynamic loading. For design and construction guidance, see Rooftop Solar Panel Attachment: Design, Installation, and Maintenance, Recovery Advisory 5, August 2018 (FEMA 2018b): https://www.fema.gov/media-library-data/1535554011182-e061c2804fab7556ec848ffc091d6487/USVI-RA5RooftopSolarPanelAttachment_finalv3_508.pdf.*

8.5 QUALITY CONTROL IN CONSTRUCTION

These provisions shall apply to the building envelope systems included in the scope of this section, as well as to adjacent components and building services that interface with these systems. The scope that applies to the fabricating or installing contractor is referred to as the "contractor."

> ***Commentary:*** *In some cases, the construction team will be composed of several different entities depending on the complexity of the building envelope system supply chain. The responsibility of each entity needs to be reviewed on a project by project basis.*

8.5.1 Documentation

During construction the contractor shall be required to supply the following documentation demonstrating compliance with the contract documents to the designer of record for review and comment:

- Shop drawings including relevant details of the building enclosure system(s) including interfaces and penetrations through that/those systems,
- Structural calculations, including strength, deflection, and movement compatibility calculations,
- Product data including but not limited to structural and waterproofing testing,
- Testing plans for preconstruction and in situ testing, and
- Quality control processes and procedures.

> ***Commentary:*** *The detail of the shop drawings, calculations, and other submittals provided should be reviewed on a project by project basis. Complex and custom systems should include explicit requirements for what documentation the contractor needs to submit.*

During submittal review, the designer of record needs to ensure that all required documents are submitted and that they include the necessary information. The submittal information should be thoroughly checked to ensure its validity.

Calculation submittals should demonstrate the development of a load path through the building envelope system and into its supporting element including but not limited to attachment components such as fasteners, welds, rivets, embed anchors, and the like.

Further, the contractor shall provide documentation demonstrating movement compatibility with the main wind resistant system of the building.

8.5.2 Substitutions

Any contractor proposed substitutions shall be reviewed by the designer of record for compliance with the project performance requirements. Proposed substitutions shall not result in a decrease in the building envelope's performance.

8.5.3 Mockups and Physical Testing

Prior to construction, the contractor shall demonstrate performance compliance by building mockups and performing physical testing on mockups. All mockups shall meet the performance criteria of that system.

There shall be no special measures or techniques used in the mockup that will not be representative of those used in the building. The mockup tested shall be representative of the finished work to simulate final conditions.

During and after construction, there shall be in situ testing to demonstrate final as-built performance complies with design criteria.

Commentary: *Note different tests and quantities of test will apply to each type of building envelope system. More testing should be implemented for more complex and unproven systems and details. Laboratory testing is typically used in the development of façade systems; however, transitions to roof system can be included in these tests.*

In-situ mockups and testing can be performed for façade and roof systems. The type of testing desired can influence the schedule of construction and sequence of installation.

Testing shall be performed by an independent testing agency, not the installing contractor. This testing agency shall be specifically qualified. The results of all tests shall be documented by the independent testing agency.

Commentary: *The independent testing agency should not be the installing contractor to include a degree of independence to the testing. There is precedence to allowing designers of record or their consultants to perform the testing at the owner's consent.*

8.5.3.1 Industry standards and requirements

Industry testing standards include, but are not limited to, the following:

AAMA 520—*Voluntary Specification for Rating the Severe Wind-driven Rain Resistance of Windows, Doors and Unit Skylights* (AAMA 2012b).

> ***Commentary:*** *Note that this is the test standard through which the waterproofing requirements for this standard are based. Many products are tested by manufacturers under different standards, which may include less stringent criteria than provided in this chapter. Designers may include additional testing but should not replace the AAMA 520 testing with other substituted water infiltration testing.*

ASTM E330—*Standard Test Method for Structural Performance of Exterior Windows, Doors, Skylights and Curtain Walls by Uniform Static Air Pressure Difference* (ASTM 2014c)

> ***Commentary:*** *This test may be used to test for strength as well as deflections under wind loads. Other project loads such as blast requirements may require additional testing.*

ASTM E331-00—*Standard Test Method for Water Penetration of Exterior Windows, Skylights, Doors, and curtain Walls by Uniform Static Air Pressure Difference* (ASTM 2016f)

ASTM E1886-13a—*Standard Test Method for Performance of Exterior Windows, Curtain Walls, Doors, and Impact Protective Systems Impacted by Missile(s) and Exposed to Cyclic Pressure Differentials* (ASTM 2013)

AAMA 501.2—*Quality assurance and diagnostic water leakage field check of installed storefronts, curtain walls, and sloped glazing systems* (AAMA 2015)

AAMA 502—*Voluntary Specification for Field Testing of Newly Installed Fenestration Products* (AAMA 2012a)

AAMA 501.4—*Recommended Static Test Method for Evaluating Curtain Wall and Storefront Systems Subjected to Seismic and Wind Induced Interstory Drifts* (AAMA 2018)

AAMA 501.7—*Recommended Static Test Method for Evaluating Windows, Window Wall, Curtain Wall and Storefront Systems Subjected to Vertical Interstory Movements* (AAMA 2017a)

ASTM D7186—*Standard Practice for Quality Assurance Observation of Roof Construction and Repair* (ASTM 2014b)

ASTM E907—*Standard Test Method for Field Testing Uplift Resistance of Adhered Membrane Roofing Systems*

National Roofing Contractors Association—*Quality Control and Quality-assurance Guidelines for the Application of Membrane Roof Systems* (NRCA 2017)

National Roofing Contractors Association / Spray Polyurethane Foam Alliance—*Quality Control Guidelines for the Application of Sprayed Polyurethane Foam Roofing* (NRCA 2003)

8.5.3.2 Wall and fenestration laboratory testing

Laboratory testing is an optional requirement except for buildings that require enhanced criteria where testing is mandatory. The requirements of this section shall apply to the systems selected for a project that is implementing laboratory testing.

> ***Commentary:*** *Laboratory testing should be implemented for complex façade systems and building interfaces that require careful and unconventional detailing. Laboratory testing allows for structural and movement testing, which is often highly difficult to perform in an in situ condition.*

The testing sequence shall be determined on a project basis, taking into account the building envelope systems that are to be included in the laboratory testing. In the event of a failed test, the contractor shall work with the designer of record and independent testing agency to determine the cause of failure and to design a solution. After a failure, resume testing at least one test prior to the failed test. The designer of record may also determine to restart at an earlier test.

8.5.3.2.1 Structural testing

Structural testing shall include the following:

- *Preloading* (ASTM E330, 2014c),
- *Uniform Structural Design Load Test* (ASTM E330, 2014c),
- *Uniform Structural Over Load Test* (ASTM E330, 2014c), and
- *Impact Testing* (ASTM E1886-13a, 2013).

Structural testing shall demonstrate whether building envelope systems meet project structural performance requirements such as deflection criteria. Structural testing shall be used in conjunction with weatherproofing performance testing. Structural performance shall follow the design criteria specified.

> ***Commentary:*** *Designers should specify, in advance, locations on a mockup that require measurement. The composition of the mockup may determine the maximum testing pressure that can be utilized if the subcomponents of that mockup do not represent a project condition with uniform loading. As this testing can stress seals; weatherproofing performance should be tested after structural testing to ensure serviceability.*

8.5.3.2.2 Movement testing

Movement testing shall include (1) *Interstory Differential Horizontal Movement Test* (AAMA 501.4, AAMA 2018) and (2) *Interstory Differential Vertical Movement Test* (AAMA 501.7, AAMA 2017a)

Movement compatibility testing shall demonstrate whether building envelope systems can accommodate building structural movement criteria. Movement testing shall be used in conjunction with weatherproofing performance testing and should be tested at the building movements equivalent to the test criteria used for waterproofing performance testing.

> ***Commentary:*** *Movement of the mockup is tested via interstory differential movement testing. Typically, this is performed for façade systems but may include roofing interfaces. As this testing can stress seals; weatherproofing performance should be tested after structural testing to ensure serviceability.*

8.5.3.2.3 Weatherproofing testing

Weatherproofing testing shall include AAMA 520—*Voluntary Specification for Rating the Severe Wind-driven Rain Resistance of Windows, Doors and Unit Skylights* (AAMA 2012b).

Weatherproofing testing shall demonstrate whether building envelope systems can meet the performance criteria required in this standard. Weatherproofing performance testing shall be performed after structural and movement testing to demonstrate system serviceability. Weatherproofing performance shall follow the design criteria specified.

> ***Commentary:*** *Note that each project and system may require different degrees of weatherproofing intermediate testing throughout a test program. It is not the intent of this prestandard to specify the order and quantity of the test program; however, each of the weatherproofing tests should be performed at least once after structural and movement testing has been performed.*

8.5.3.3 Roof In Situ Testing

8.5.3.3.1 Uplift testing

Roof system field uplift testing: For adhered roof membrane systems, conduct field uplift resistance testing in accordance with ASTM E907. Conduct the majority of the tests in the corner and perimeter zones (i.e., ASCE 7 zones 2 and 3), and at least one test in the field of the roof (i.e., ASCE 7 zone 1 and zone 1' if it occurs).

> ***Commentary:*** *If the building will be insured by FM Global, use the Property Loss Prevention Data Sheet 1-52 in lieu of ASTM E907.*

If the building is not in a hurricane-prone region, test the corner(s) of the prevailing wind direction, plus the perimeter and field. If the building is in a hurricane-prone region, test all the corners, plus the perimeter and field.

> ***Commentary:*** *If the prevailing direction is not known, test the northwest and southwest corners, plus the perimeter and the field.*

8.5.3.4 Wall and fenestration in situ testing

In-situ testing shall be performed for wall and fenestration system to confirm installation quality relative to the performance specified herein.

> ***Commentary:*** *Note that structural and movement in situ testing for wall and fenestration elements can be difficult to implement when significant loading is required. The designer is advised to use laboratory testing to perform structural testing.*

8.5.3.4.1 Air and water infiltration testing

Field testing for installed fenestration, and wall systems shall follow AAMA 501.1 (ASTM 2017b), AAMA 502 (AAMA 2012a) and ASTM E1105 (ASTM 2015).

In-situ testing to shall be performed by a qualified independent testing and inspection agency, which will also prepare an inspection and test report. In-situ testing shall occur at various stages of façade installation, when sections of the complete façade system have been fully installed but before interior finishes are installed. Installation shall not proceed until test results for previously completed areas show compliance with requirements.

The in situ air and water infiltration tests will be subject to the same performance criteria as in the mockup test and performed on an agreed-on percentage of the façade. Locations shall be selected to provide the best sampling possible of the façade types, with each location covering a minimum area determined by the designer of record.

8.6 INSTALLATION INSPECTIONS

The owner shall engage a qualified inspector, independent of the contractor, to monitor the quality of the building envelope systems during installation.

8.6.1 Roof Systems

Quality Control—The roofing contractor shall have on-site and shall follow the applicable NRCA quality control guideline.

Quality Assurance—Quality assurance shall be performed in accordance with ASTM D7186 (ASTM 2014b). The quality assurance observer shall be a registered roof consultant or a registered roof observer. The designer of record shall determine the frequency of observation (part-time or full-time) based on the building owner's performance objective.

8.6.2 Wall and Fenestration Systems

Installation inspections shall follow the more stringent of the special inspection provisions in the local jurisdiction or the following:

Check the site conditions at the time the structure is prepared for component installation—and periodically during component installation. Verify that the following work is performed in compliance with the approved construction documents, including

- The supporting structure for components being installed is aligned and within specified tolerances required;
- Required inserts are installed;
- Framing components are installed and aligned as specified and without structural defects or weaknesses;
- Anchors are placed, welded, bolted, and finished as specified, as applicable;
- Weeps, flashings, and tubes are installed as specified and functioning;
- Joinery and end dams are sealed as specified;
- Sealing materials with specified adhesive and movement capabilities are installed;
- Gaskets, tapes, seals, insulation, flashing, and other materials that are barriers to air and water movement, vapor drive, and heat loss are installed as specified;
- Joint filler materials that accommodate specified horizontal and vertical movement are installed in accordance with the manufacturers' instructions; and
- Any other observations pertinent to safe installation of the wall system.

The inspections shall be performed by a qualified agency that is licensed by the local jurisdiction. A licensed professional from the qualified agency shall sign and stamp the local jurisdiction's document stating that the components inspected meet the provisions as outlined above or those in the local jurisdiction if more stringent.

8.7 POST-OCCUPANCY INSPECTIONS, MAINTENANCE, AND REPAIR

The design team shall advise the building owner of the importance of periodic inspections, maintenance, and timely repair of building envelope elements. The building envelope and exterior-mounted equipment should be inspected once per year by persons knowledgeable of the systems/materials they are inspecting. Items that require maintenance, repair, or replacement should be documented and scheduled for work.

__Commentary:__ The wind and wind-driven rain resistance of various elements of the building envelope will degrade over time due to weathering. To maintain the building owner's performance objective, periodic inspection, maintenance, repair, and replacement is necessary. The goal should be to repair or replace building envelope elements before they fail in a storm.

Unique inspections:

- At 2-yr intervals, perform a nondestructive evaluation (NDE) to check for moisture within the roof system, which can reduce a system's uplift resistance.
- An inspection is recommended following unusually high winds (such as a thunderstorm with wind speeds of 70 mph [112.65 kph] peak gust or greater). The purpose of the inspection is to assess whether the storm caused damage that needs to be repaired to maintain building envelope strength and integrity.
- For buildings located in hurricane-prone regions, an inspection is recommended before hurricane landfall. Remove roof debris and other items that are not anchored so that they do not become wind-borne debris. Also, clean roof drains/sumps, scuppers, and gutters so that their drainage capacity is not impaired.

__Commentary:__ See "Attachment of Rooftop Equipment in High-Wind Regions" (FEMA 2018) for securement of rooftop equipment before impending landfall: https://www.fema.gov/media-library-data/1522347818123-9fa4b38a90dc40b91fdb82eeb0307e98/USVI-RA2AttachmentofRooftopEquipmentinHigh-Wind_Regions_V2_508.pdf.

REFERENCED STANDARDS

ASCE 7. *Minimum design loads and associated criteria for buildings and other structures.* ASCE/SEI, 2016.

REFERENCES

AA (Aluminum Association). 2015. *Aluminum standards and data: Aluminum design manual*. AA ADM-1. Arlington, VA: Aluminum Association.

AAMA. 1989. *Metal curtain wall manual*. AAMA MCW-1. Schaumburg, IL: American Architectural Manufacturers Association.

AAMA (American Architectural Manufacturers Association). 2008. *Standard practice for the installation of windows and doors in commercial buildings*. AAMA IPCB-08. Schaumburg, IL: American Architectural Manufacturers Association.

AAMA. 2012a. *Voluntary specification for field testing of newly installed fenestration products.* AAMA 502. Schaumburg, IL: American Architectural Manufacturers Association.

AAMA. 2012b. *Voluntary specification for rating the severe wind-driven rain resistance of windows, doors and unit skylights*. AAMA 520. Schaumburg, IL: American Architectural Manufacturers Association.

AAMA. 2014a. *Aluminum storefront and entrance manual*. AAMA SFM1. Schaumburg, IL: American Architectural Manufacturers Association.

AAMA. 2014b. *Design guide for metal cladding fasteners*. AAMA TIR-A9. Schaumburg, IL: American Architectural Manufacturers Association.

AAMA. 2015. *Quality assurance and diagnostic water leakage field check of installed storefronts, curtain walls, and sloped glazing systems.* AAMA 501.2. Schaumburg, IL: American Architectural Manufacturers Association.

AAMA. 2017a. *Recommended static test method for evaluating windows, window wall, curtain wall and storefront systems subjected to vertical interstory movements.* Schaumburg, IL: American Architectural Manufacturers Association.

AAMA. 2017b. *Standard test method for water penetration of windows, curtain walls and doors using dynamic pressure.* AAMA 501.1. Schaumburg, IL: American Architectural Manufacturers Association.

AAMA. 2018. *Recommended static test method for evaluating curtain wall and storefront systems subjected to seismic and wind induced interstory drifts.* AAMA 501.4. Schaumburg, IL: American Architectural Manufacturers Association.

AAMA/WDMA/CSA. 2017. *North American fenestration standard.* 101/I.S.2/A440. Schaumburg, IL: American Architectural Manufacturers Association.

ACI (American Concrete Institute). 2004. *Thin reinforced cement-based products and construction systems.* ACI SP-224. Farmington Hills, MI: American Concrete Institute.

ACI. 2013. *Building code requirements and specification for masonry structures.* ACI 530.1. Farmington Hills, MI: American Concrete Institute.

AISC (American Institute of Steel Construction). 2016a. *Code of standard practice for steel buildings and bridges.* ANSI/AISC 303. Chicago: American Institute of Steel Construction.

AISC. 2016b. *Specification for structural steel buildings.* ANSI/AIS 360. Chicago: American Institute of Steel Construction.

ASTM International. 2013. *Standard test method for performance of exterior windows, curtain walls, doors, and impact protective systems impacted by missile(s) and exposed to cyclic pressure differentials.* ASTM E1886-13a. West Conshohoken, PA: ASTM.

ASTM. 2014a. *Standard guide for structural sealant glazing.* ASTM C1401. West Conshohoken, PA: ASTM.

ASTM, 2014b. *Standard practice for quality assurance observation of roof construction and repair.* ASTM D7186.

ASTM. 2014c. *Standard test method for structural performance of exterior windows, doors, skylights and curtain walls by uniform static air pressure difference.* ASTM E330. West Conshohoken, PA: ASTM.

ASTM. 2015. *Standard test method for field determination of water penetration of installed exterior windows, skylights, doors, and curtain wall by uniform or cyclic static air pressure.* ASTM E1105. West Conshohoken, PA: ASTM.

ASTM. 2016a. *Standard guide for use of joint sealants.* ASTM C1193. West Conshohoken, PA: ASTM.

ASTM. 2016b. E1300 *Standard practice for determining load resistance of glass in buildings.* ASTM E1300. West Conshohoken, PA: ASTM.

ASTM. 2016c. *Standard specification for flat glass.* ASTM C1036. West Conshohoken, PA: ASTM.

ASTM. 2016d. *Standard specification for loadbearing concrete masonry units.* ASTM C90. West Conshohoken, PA: ASTM.

ASTM. 2016e. *Standard test method for determining compatibility of liquid-applied sealants with accessories used in structural glazing systems.* ASTM C1087. West Conshohoken, PA: ASTM.

ASTM. 2016f. *Standard test method for water penetration of exterior windows, skylights, doors, and curtain walls by uniform static air pressure difference.* ASTM E331-00. West Conshohoken, PA: ASTM.

ASTM. 2017a. *Standard specification for facing brick (solid masonry units made from clay or shale).* ASTM C216. West Conshohoken, PA: ASTM.

ASTM. 2017b. *Standard specification for PB exterior insulation and finish systems.* ASTM E2568. West Conshohoken, PA: ASTM.

ASTM. 2017c. *Standard specification for performance of exterior windows, curtain walls, doors, and impact protective systems impacted by windborne debris in hurricanes.* ASTM E1996. West Conshohoken, PA: ASTM.

ASTM. 2017d. *Standard specification for rigid PVC and related plastic building product compounds.* ASTM D4216. West Conshohoken, PA: ASTM.

ASTM. 2018a. *Standard guide for selection, design, and installation of dimension stone attachment systems.* ASTM C1242. West Conshohoken, PA: ASTM.

ASTM. 2018b. *Standard guide for selection of dimension stone.* ASTM C1528. West Conshohoken, PA: ASTM.

ASTM. 2018c. *Standard specification for heat-strengthened and fully tempered flat glass.* ASTM C1048. West Conshohoken, PA: ASTM.

ASTM. 2018d. *Standard specification for rigid PVC exterior profiles used for assembled windows and doors.* ASTM D4726. West Conshohoken, PA: ASTM.

ASTM. 2019a. *Standard specification for laminated architectural flat glass.* ASTM C1172. West Conshohoken, PA: ASTM.

ASTM. 2019b. *Standard specification for thin veneer brick units made from clay or shale.* ASTM C1088. West Conshohoken, PA: ASTM.

BIA (Brick Industry Association). 2005. *Technical notes on brick construction 28B: Brick veneer / Steel stud walls.* Reston, VA: Brick Industry Association.

FEMA. 2010. *Design guide for improving school safety in earthquakes, floods, and high winds.* FEMA 424. Washington, DC: Federal Emergency Management Agency.

FEMA. 2018a. *Attachment of rooftop equipment in high-wind regions.* Recovery Advisory 2. Washington, DC: Federal Emergency Management Agency.

FEMA. 2018b. *Rooftop Solar Panel Attachment: Design, Installation, and Maintenance.* Recovery Advisory 5. Washington, DC: Federal Emergency Management Agency.

GANA (Glass Association of North America). 2008. *Glazing manual.* Topeka, KS: Glass Association of North America.

ICC (International Code Council) 2014. *Standard for the design and construction of storm shelters.* ICC/NSSA 500. Washington, DC: International Code Council.

ICC. 2018. *International building code*. Washington, DC: International Code Council.

ILI (Indiana Limestone Institute of America). 2007. *Indiana limestone handbook.* Bedford, IN: Indiana Limestone Institute of America.

NSI (Natural Stone Institute). 2016. *MIA Dimension stone design manual*. Oberlin, OH: Marble Institute of America.

Mooneghi, M. A., T. L. Smith, P. Irwin, and A. G. Chowdhury. 2017. “Concrete roof pavers: Wind uplift aerodynamic mechanisms and design guidelines–A proposed addition to ANSI/SPRI RP-4.” In *Proc., 32nd RCI International Convention and Trade Show, Anaheim, CA, March 16-21.* Raleigh, NC: International Institute of Building Enclosure Consultants.

NBGQA (National Building Granite Quarries Association). 2018. *Specification for architectural granite*. V18-1. Washington, DC: National Building Granite Quarries Association.

NRCA (National Roofing Contractors Association). 2017. *Quality control and quality-assurance guidelines for the application of membrane roof systems.* Washington, DC: National Roofing Contractors Association.

NRCA / Spray Polyurethane Foam Alliance (SPFA). 2003. *Quality control guidelines for the application of sprayed polyurethane foam roofing.* Washington, DC: National Roofing Contractors Association.

PCI (Precast/Prestressed Concrete Institute). 2001. *Recommended practice for glass fiber reinforced concrete panels.* MNL 128-01. Chicago: Precast/Prestressed Concrete Institute.

PCI. 2014. *PCI design handbook: Precast and prestressed concrete.* MNL 120. Chicago: Precast/Prestressed Concrete Institute.

SPRI (Single Ply Roofing Industry). 2016. *Test standards for gutter systems.* ANSI/SPRI GT-1. Waltham, MA: Single Ply Roofing Industry.

Chapter 9. Project Review

9.1 ENGAGING INDEPENDENT PEER REVIEW

An independent peer review shall be engaged when using the provisions of this prestandard.

Engage independent peer review by one or more individuals acceptable to the concerned parties and possessing experience and knowledge pertaining to the following items:

- Wind hazard definition including determination of design wind speeds and directions as a function or probability of exceedance;
- Wind performance of structural and cladding systems of similar type;
- Wind impacts on building occupancy and tolerance of the specific building occupancy to wind effects;
- Exposure determination;
- Establishment of wind loading by wind tunnel modeling;
- Behavior of structural systems including foundations and supporting soils, relevant to the building under consideration, when subjected to wind loading;
- Application of structural analysis software for use in wind response analysis of the type proposed for the project and interpretation of analysis results;
- Expertise in the use of physical tests to develop structural analysis models and associated acceptance criteria, if such development will be required for the project;
- Expertise in the use of physical tests to evaluate the ability of cladding systems and other nonstructural components to resist the effects of wind pressures in combination with rain and wind-induced building movements;
- Requirements of this prestandard as they pertain to design of the type of structure under consideration; and
- Structural reliability evaluation, if explicit evaluation of reliability is performed as part of the project analyses.

> ***Commentary:*** *Peer review may be undertaken to satisfy requirements imposed by the AHJ, or as a voluntary means of providing quality assurance on behalf of the owner, Engineer of Record (EOR), or both. When peer review is required by the AHJ, it is important to establish early in the process that the peer reviewer(s) will be acceptable to the AHJ. Regardless, peer reviewers should be acceptable to the owner, and be able to work in a collaborative manner with the EOR.*

It is the intent of this prestandard to require a minimum of two qualified design professionals to review the design of each building enclosure system. A delegated design by a contractor may serve as the second independent review of the design.

In addition to the technical expertise noted in this section, experience as a practicing design professional can help a reviewer or a review team understand the practical design conditions under which the designer is working. For this reason, the peer review should include at least one individual with experience as a practicing engineer engaged in the design of buildings of similar size and occupancy. Where performance-based design addresses the design of cladding, the expertise of an architect or subject matter expert with specialized knowledge in this area should be included.

9.2 SELECTION AND REPORTING REQUIREMENTS

When required by the AHJ, reviewers shall provide their professional opinion to and shall act under the instructions of the AHJ.

Commentary: *The composition of the peer review panel typically should be jointly determined by the owner/design team and, when performed on behalf of the AHJ, also the building department. Owner involvement is relevant because of the financial investment required for the project and in its peer review. Design team involvement is important because of its intimate knowledge of the building design, as well as knowledge of relevant expertise of individuals who might serve as peer reviewers.*

There is no recommendation as to whether an individual person or firm or a team of individuals and firms provides the peer review. However, the peer reviewer or reviewers should jointly possess expertise in the areas noted in Section 9.1. Reviewers should not bear a conflict of interest with respect to the project and should not be part of the project design team. In selecting peer reviewers, it is advisable to ascertain that the reviewer is able to commit the time required for the review such that the review can proceed in a timely manner.

On many projects, peer review is provided by a team. Typically, one member is a practicing structural engineer who has the expertise to review the proposed structural system, with experience in structural engineering, performance-based wind engineering, building response analysis, and design of structures of similar type. Where the cladding design is performance-based, the experience of an architect or specialty cladding designer may be also be required. The reviewing engineer, or the engineer's staff typically performs detailed reviews of structural analysis models implemented in computer software. Another member typically is an expert in wind hazard analysis and wind load determination by wind tunnel. A third member typically possesses specialized expertise related to the proposed structural system, possibly a structural engineering researcher, with additional expertise in wind engineering, performance-based engineering, wind response analysis, and building design, and/or cladding design. There is, however, no requirement that a panel comprise three members. The number of members may be expanded or contracted as appropriate, provided that the review team as a whole possesses expertise in all of the areas noted.

Commentary: *When review is performed by a team, one team member should serve as the review team chair. The chair's responsible to mediate disputes, if any, among the reviewers and responsible on behalf of the peer review team for maintaining the peer review record and for expressing the official positions and opinions of the review team. Some jurisdictions require that the chair of the review team be a design professional licensed to practice in the jurisdiction in which the structure is to be constructed, but that is not a general requirement of this prestandard.*

9.3 SCOPE OF WORK

Discuss the scope of the peer review among the owner, project design team, peer review team, and the AHJ as appropriate. Include the following items in the scope of work as appropriate:

1. Basis of design document, including the wind performance objectives, the overall wind design methodology, and acceptance criteria;

2. Proposed structural system and materials of construction, including damping system as applicable;

3. Proposed cladding systems, attachment to structure, and weatherproofing detailing;

4. Wind hazard determination, and wind tunnel testing technique to be used to determine loading;

5. Modeling approaches for structural materials and components;

6. Structural analysis model, including verification that the structural analysis model adequately represents the properties of the structural system within accepted norms for building designs;

7. Review of structural analysis results and determination of whether calculated response meets approved acceptance criteria;

8. Design and detailing of structural components;

9. Drawings, specifications, and quality control/quality assurance and inspection provisions in the design documents;

10. Laboratory testing of structural and/or cladding components; and,

11. Any other considerations that are identified as being important to meeting the established performance objectives.

Commentary: *It is necessary to have a clear definition of the peer review scope. When peer review is required by the AHJ, the building department should define the minimum acceptable scope. In most cases, the review is limited to the wind design, although design for seismic forces and deformations may control aspects of the design. The design of the building under gravity-only load combinations in general is excluded from the scope. However, consideration of gravity-load-resisting elements for forces and deformation* *compatibility issues, as the structure responds to wind loading should in general be included in the scope. Nonstructural elements that can create hazards to life safety or that are vulnerable to wind-induced damage are often included to ensure that proper anchorage and/or deformation accommodation has been provided. At the discretion of the building official, as well as other members of the development team, the scope of review may be expanded to include review of other building aspects, including wind design of other critical nonstructural elements.*

Based on the scope of review identified by the AHJ, peer reviewers—either individually or as a team—should develop a written scope of work in the contract to provide engineering services.

9.4 PEER REVIEW PROCESS

Convene a meeting among the Engineer of Record, the AHJ, and the peer reviewers to establish the scope of work, methods and lines of communication, frequency and timing of review milestones, and degree to which the Engineer of Record anticipates the design will be developed for each milestone.

Commentary: *The peer review process should initiate as early in the design process as is reasonable. Early agreement and discussion of the fundamental design decisions, assumptions, and approaches will help avoid rework later in the design process that will affect both the project cost and schedule. There may be differences of opinion on a number of issues during the process that need to be negotiated among parties. The earlier in the process that these issues can be identified and resolved, the less impact they will have on the building cost as well as the design and construction schedule. Early participation in the peer review should also help to establish a congenial working relationship with the design team.*

When involved, the AHJ, Engineer of Record, peer reviewers, and possibly owners should hold a kickoff meeting to establish expectations for the peer review. Normally, a kickoff meeting is held in person. The kickoff meeting should discuss scope of work, schedule, and any special communication or submittal requirements. It is effective at the kickoff meeting to establish a single point of contact for the building official, design team, and peer review team, and for all subsequent communications to be directed through those individuals, with copies to other individuals as appropriate. Written communications should have an agreed-on heading that identifies the project, such that it is easy to identify, and file communications related to the project.

Although the kickoff meeting is usually held in person, subsequent meetings may be conducted either in person or by electronic alternatives, as best suits the participants and the review.

The timing of reviews should be incorporated into the project design schedule to minimize any impact on the schedule. Periods of both review and response by the design team should be included into the project design schedule.

Provide design submittals for review by the peer review team, organized and documented in a manner that facilitates review by the review panel. Reviewers shall provide written comments, in a timely fashion, to the EOR and to the AHJ, when required, with requests for action as necessary. The design team is responsible for resolving all comments to the satisfaction of the reviewers.

> ***Commentary:*** *The review process is driven by submittals by the design team to the peer review team. Preferably, the submittals and their review should begin with the basis of design, which should resolve broad issues about the design approach, as well as detailed matters of acceptance criteria. Subsequent review is likely to progress to more detailed results of the design. In general, it is considered unfair to the EOR to bring up new issues related to the overall design process at later stages of the design, although such matters should be considered when critical to the design's performance capability.*
>
> *Most submittals for review are in electronic form. However, at certain phases of the design, it may be necessary to submit some materials such as structural drawings in paper form to facilitate the review. After each submittal, good practice is for the design team to convene a meeting with the reviewers in which the design team describes the nature of the submittals and explains important details. The review team is then given a reasonable time in which to review the submittals and develop comments in a comment log. A meeting to discuss the comments may be appropriate. The EOR should provide written responses to review comments, with multiple rounds of comment/response sometimes needed for key issues.*
>
> *Proper documentation of the peer review process is important for incorporation into the project records. It is best to develop a systematic process for establishing, tracking, and resolving comments generated by the peer review. In many cases, this takes on the form of a written spreadsheet that is used to log all comments and resolutions, with dates included. Comments that are discussed and/or any resolutions that are reached during project review meetings or conference calls should be formally written into the project review comment spreadsheet.*

At the conclusion of the review, and at other times requested by the owner or AHJ, the review team shall submit a written report to parties requiring review documenting the scope of the review, the comment log, and the reviewers' professional opinion regarding the general conformance of the design to the requirements of the design criteria document.

> ***Commentary:*** *Some projects may require interim reports from the review panel to facilitate phased permitting. Examples include the excavation permit or the foundation permit. In general, for such interim reports the design needs to have progressed sufficiently so that the review team is able to state that the permit can be justified on the basis of the work completed to date. The report should state clearly any caveats regarding the work not yet completed and should clarify that it is the responsibility of the EOR to provide, at a later date, any incomplete information necessary to support the requested permit.*

9.5 DESIGN RESPONSIBILITY

The architect and EOR are solely responsible for the construction contract documents.

Commentary: *It should be noted that the existence of peer review on a project does not relieve designers of record from any of their professional responsibility. Peer review participation is not intended to replace quality assurance measures ordinarily exercised by the designers of record. Design responsibility remains solely with the design professionals of record, as does the burden to demonstrate conformance of the design to the intent of the design criteria document and building code as applicable. The responsibility for conducting plan review resides with the building official. Third-party entities may be hired to assist with the plan review. It can be acceptable for one or more members of the peer review team to assist with plan review under separate contract.*

None of the reports or documents generated by the review are construction documents. Under no circumstances should letters, reports, or other documents from the review be included with the project drawings or reproduced in any other way that makes review documents appear to be part of the construction contract documents. The designers of record are solely responsible for the construction contract documents. Documents from the reviewers should be retained as part of the building department project files.

9.6 DISPUTE RESOLUTION

Where disputes between the designers of record and reviewers arise and cannot be resolved as part of the regular review process, resolution of the dispute shall be by the commissioning authority that is the owner or AHJ as appropriate. The commissioning authority can provide resolution based on personal knowledge of the situation or, alternatively, may retain other experts to review the material and generate a recommended course of action.

Commentary: *Given the complexity of the performance-based design process, disagreements may arise between the designers of record and the reviewers. In general, these disagreements fall into one of two categories. The first is regarding the level of complexity of analysis/evaluation/testing that has been performed to validate an aspect of the design. In most cases, this should be resolvable with additional analyses, confirming studies, and other means. The second case is related to differences of opinion in the interpretation of results, specifically as to whether elements of the design criteria have been met. Resolution of such issues may be obtained through sensitivity analyses, bounding analyses, or other means.*

For jurisdictions that have a large number of projects incorporating building department-mandated review of performance-based design procedures, establishment of an advisory board should be considered. An advisory board should consist of individuals who are widely respected and recognized for their expertise in relevant fields, including but not limited to structural engineering, cladding design, performance-based design, response analysis techniques, and wind engineering. The advisory board members may be elected to serve for a predetermined period on a staggered basis. The advisory board may oversee the design review process across multiple projects periodically, assist the AHJ in developing criteria and procedures spanning similar design conditions, and resolve disputes arising under peer review.

9.7 POST-REVIEW REVISION

When substantive changes to the building design occur during project phases subsequent to completion of the peer review, the EOR shall inform the AHJ, describing the changes to the structural design, detailing, or materials. At the discretion of the AHJ, such changes may be subject to additional review by the peer review team and approval by the AHJ.

> ***Commentary:*** *Because of the fast-track nature of many modern large building projects, it is not unusual for substantive changes to the design to occur during the final stages of the design or construction. It is the responsibility of the Engineer of Record to bring such changes to the attention of the AHJ wherever these changes may reasonably be suspected of affecting the building's performance. Substantive changes include changes in the wind-force-resisting system configuration, design, detailing, or materials.*

Appendix A
Method 1: Analytical Procedure for Continuous Occupancy Performance Objective

Structural Analysis for the Continued Occupancy performance has three possible methods, as shown in Figure 2-1. Method 1 uses traditional linear and nonlinear response history analysis. One approach for carrying out this analysis is provided in this appendix.

Step 1. For the chosen risk category, establish critical wind directions and associated wind load histories through the procedures outlined in Chapter 5.

1.1) Develop the wind design scenarios as outlined in Chapter 5 for all 360-degree wind directions at 10-degree intervals maximum.

1.2) Determine a minimum of two critical wind directions in terms of overturning moment and base shear for use in analysis noted following.

Step 2. Linear Response History Analysis:

2.1) Develop a preliminary design of the structure, where structural elements are divided into the following groups:

i. Deformation-controlled elements of the MWFRS

ii. Force-controlled elements of the MWFRS

iii. Force-controlled structural elements (e.g., gravity-only framing)

iv. Force-controlled nonstructural elements (e.g., partitions and cladding)

v. Passive energy devices incorporating friction, viscous, or viscoelastic damping

2.2) Develop a linear elastic mathematical model of the system. Passive energy devices are modeled using equivalent viscous damping.

2.3) Perform a gravity load analysis to initialize second order effects.

2.4) For each wind record to be considered, perform linear response history analysis as outlined in Chapter 6 for all critical wind directions.

2.5) Using the maximum response quantity determined among all wind records considered, validate the following criteria:

i. Demand-to-capacity ratios for deformation-controlled elements do not exceed 1.25, relative to the expected strength.

ii. Demand-to-capacity ratios for force-controlled elements do not exceed 1.0 relative to design strength, or deformation demands for these components does not indicate loss of strength under gravity loads.

iii. Deformations or deformational velocities in passive energy devices are within acceptable ranges.

2.6) If acceptance criteria are not met within the criteria outlined in Step 2.5, redesign the structure and go back to Step 2.1; or advance to Step 3 for Nonlinear Response History Analysis. If acceptance criteria are met, and all demands remain elastic through the linear response history analysis, no further analysis is required.

Step 3. Nonlinear Response History Analysis:

3.1) Develop a nonlinear mathematical model of the system.

i. Deformation controlled elements are modeled to respond inelastically, using expected strength.

ii. Force-controlled elements are modeled to remain elastic.

iii. Passive energy devices are modeled using appropriate force-deformation or force-velocity relationships.

3.2) Perform a gravity load analysis to initialize second order effects.

3.3) Perform nonlinear response history as outlined in Chapter 6 for all critical wind directions.

3.4) Using the maximum response quantity determined among the wind records considered in step 1.2, validate the following criteria:

i. None of the individual response history analyses fails to converge.

ii. Deformation demands in deformation-controlled elements to not exceed the acceptance criteria stated in Chapter 7.

iii. Demand-to-capacity ratios for force-controlled elements are less than 1.0 relative to design strength, or deformation demands for these components do not indicate loss of strength.

iv. Forces, deformations, or deformational velocities in passive energy devices are within acceptable ranges in accordance with manufactured specifications.

v. Transient drifts for all stories are within specified limits in Chapter 7.

vi. Residual drifts for all stories are within the limits specified in Chapter 7.

vii. The number of cycles of inelastic deformation beyond the specified yield strain limit does not exceed the criteria stipulated in Chapter 7.

3.5) If acceptance criteria are not met within the criteria of Step 3.4 limits, redesign the structure and go back to Step 2.1. If acceptance criteria are met, no further analysis is required.

Appendix B
Method 2: Conditional Probability Assessment Procedure to Validate Collapse-Resistance and System Reliability of Structures to Wind Loading

B.1 PURPOSE

This appendix provides an analysis procedure that can be used to verify that the design of a structure for wind resistance provides collapse-resistance reliability comparable to that which underlies the ASCE 7 requirements, even if the structure does not conform to those requirements in all respects. More rigorous means of verifying reliability in lieu of this procedure are acceptable. Use of either this procedure or more rigorous procedures requires peer review of the approach, assumptions, implementation, and findings.

Commentary: *Reliability assessment procedures provide a rational means of quantifying: (1) the safety inherent in a structures' design and construction and considering uncertainties, (2) the magnitude and character of loading the structure will experience (e.g., demand), (3) the response that the structure exhibits under the loading (e.g., modeling and analysis), and (4) the structural capacity to respond in a safe manner, that is, without failure or collapse.*

The LRFD structural design procedures embodied in Section 2.3 of ASCE 7, as well as the design standards produced by ACI, AISI, AISC, AWC, and TMS, are formulated to achieve the notional target reliabilities stipulated in Section 1.3 of ASCE 7. With the exception of seismic effects, ASCE 7 generally sets these notional reliability targets on an individual element or connection basis as opposed to a system basis, because in general, structures are designed on an element by element basis in which system behavior is considered only to predict the magnitude of demands on individual elements. In the ASCE Standard 7, reliability targets are expressed both as reliability indices (β) (for an assumed 50 yr design life) or as annual probabilities of failure.

In contrast, the target reliabilities for seismic design are expressed as a conditional probability of failure of the structural system, given that a reference design loading, termed the Maximum Considered Earthquake shaking, is experienced by the structure. This system definition of target reliability is adopted for seismic design as a result of economic considerations and historical development. Seismic design procedures have evolved around a philosophy of accepting failure of individual elements in a structure, as long as these individual failures did not compromise the safety of occupants, with structural collapse identified as the limit state most important to protection of life safety.

This prestandard adopts a similar approach as that employed for seismic design for performance-based wind design (PBWD). The prestandard recognizes that with sufficient analysis, care, and review in the design process, similar economy may be achieved in wind design without unduly compromising life safety, which is the primary goal of all structural design.

Rigorous reliability analysis procedures employ Monte Carlo analysis, in which significant uncertainties in prediction of system demands and capacity are identified and quantified in the form of random variables with defined probability distributions derived through analysis, testing, judgment, or a combination of these methods. The Monte Carlo procedures entail thousands of analyses, or realizations, of possible individual events over the design life. In each realization, the value of each random parameter (e.g., wind speed, wind pressure distribution given this speed, inherent structural damping, building stiffness and response frequency, and structural capacity) is assigned a unique value, based on sampling of its distribution. Ultimately, the structure's reliability is determined as the number of realizations in which unacceptable behavior (e.g., a limit state of collapse) is predicted divided by the total number of realizations evaluated.

The reliability analysis procedure in this appendix assumes that the probability of failure due to wind loads and effects can be determined by only considering the uncertainty of structural capacity (e.g., column shear or overturning moment), which is modeled with a lognormal distribution. The reliability analysis is conditioned on a specified design wind speed that is compatible with the ASCE 7 wind hazard maps. This approach reduces the complexity of the reliability analysis relative to that of a fully-coupled reliability analysis, which may be analyzed with Monte Carlo techniques or other appropriate methods.

For the reliability approach in this appendix, structural analyses are conducted for a limited suite of specified wind events determined by wind tunnel testing that are representative of the ASCE 7 wind hazard, using so-called best estimate models to identify the probable median value of the primary variate at which collapse occurs. Engineering judgment is then used to quantify the uncertainties, measured by either standard deviations (SD) or coefficients of variation (COV) that affect the determination of structural demand and capacity. These uncertainties are combined to determine whether the resulting fragility function indicates an acceptable conditional probability of failure, given the design wind loading.

We do not have extensive experience with characterizing wind hazards for the purpose of performance-based design. In particular, more experience is needed for predicting the history of building dynamic responses subject to a suite of wind records that capture the nature of the wind hazard over a long period of time (e.g., hours to days). Similarities do exist to performance based structural design for earthquakes, in which the seismic ground motion hazard is characterized by a site-specific hazard curve tied to the spectra response acceleration at the fundamental period of the building. Ground motion records are selected for response history analysis and scaled by various methods to match the response at the intended range of periods. For wind, however, there is no single parameter comparable to period of vibration that can be used to characterize the wind hazard curve for a single structure. Therefore, wind tunnel studies are needed to examine the planned structure in its future environment for a variety of wind speeds and windstorm histories, as well as for all possible orientations of the wind with respect to the building. In particular, cross-wind response of flexible structures and aeroelastic instability are strongly dependent on the building shape, which can produce very different levels of response for seemingly minor adjustments to the shape. This means that a hazard curve based on the peak velocity is inadequate to represent the wind hazard for a given building over its design life and that *the hazard curve must be described in terms of the expected wind tunnel response of the building.*

The proposed procedure is based on our state of knowledge at the present, and it would be prudent to expect changes as experience is gained.

B.2 GENERAL PROCEDURE

B.2.1 Wind Hazard Development

a. In consultation with the Engineer of Record, the wind consultant shall determine the behavior of key responses such as peak base overturning moments, M_x and M_y, base torsional moment, M_z, base shears S_x and S_y, accelerations, and so on as a function of mean recurrence interval (MRI), assuming linear behavior of the structure. This determination shall be accomplished by undertaking wind tunnel tests for the full range of wind directions at 10-degree intervals (or smaller), accounting for mean, background, and resonant responses in the along-wind, crosswind, and torsional directions, and combining the wind tunnel data with the joint probability of wind speed and direction. The necessary information on natural frequencies, mode shapes and mass distribution for the linear analysis shall be provided by the Engineer of Record and a representative damping ratio shall be chosen in joint consultation between the Engineer of Record and the wind consultant. The wind tunnel tests shall meet the requirements of ASCE 49-12.

   For exposed, lightweight, slender buildings susceptible to aeroelastic effects, the wind tunnel test program shall include aeroelastic model tests. If the aeroelastic model exhibits higher peak responses than predicted from the rigid model tests, then the negative aerodynamic damping causing this shall be quantified by the wind consultant as a function of wind velocity for each of the design scenarios and included in the nonlinear time history analysis.

b. The results of **B.2.1a** shall be used to identify the most critical wind directions and speeds contributing to the MRI curves, and to develop a minimum of 10 design scenarios in terms of wind directions and speeds appropriate to the structure's risk category. For each of these design scenarios, time histories of the relevant aerodynamic coefficients shall be provided by the wind consultant for durations corresponding to two or more hours at full scale. The time increments used in these histories shall be sufficiently short to resolve nonlinear structural response in the critical modes of vibration over the range of wind speeds relevant to design. When used in incremental nonlinear dynamic analysis in which full-scale wind speeds for each wind direction are successively increased, the time increments shall be appropriately adjusted for the design wind MRI to reflect the changed scaling from wind tunnel velocity to each full-scale wind velocity.

Commentary: *Wind effects for buildings are highly dependent on building shape and orientation, surroundings, and wind speed and direction. Selection of design scenarios should be based on inspection of the building's responses, not just the wind velocity statistics. Once the responses (*M_x, M_y, M_z, S_x, S_y, *etc.) versus MRI have been determined from the wind tunnel study (for the linear case), then the key wind directions and wind speeds* *contributing most to these responses can be identified and used to guide the conditions under which to run nonlinear time history analysis (NLTHA).*

The data from the wind tunnel used in the NLTHA are in the form of time histories of aerodynamic coefficients that are independent of wind speed. These can be combined with a reference mean speed (at some reference height) for the design wind MRI and scaled for use in incremental dynamic analysis. The requirement for a minimum of 10 design scenarios is based on engineering judgement and current wind tunnel practice.

B2.2 Collapse Initiation Mode Identification

a. Based on analysis of representative structural models to the representative design wind scenarios, the structural engineer shall identify critical collapse initiation modes for the structure. Examples of these include onset of P-delta instability, toe crushing of a primary wind force-resisting concrete wall, failure of a critical tie down element, and crushing and/or buckling of a critical load-bearing column in a braced frame or moment frame, among others.

b. For each critical collapse initiation mode, select a demand parameter (e.g., story drift, concrete compressive strain, column axial load, etc.) that can be used as a predictive parameter for collapse mode initiation. Estimate a value of this parameter that reasonably represents a high confidence lower bound value for the collapse mode (e.g., not greater than a 10% chance that the collapse mode would initiate, given the occurrence of that demand). Estimate an uncertainty (COV) associated with this behavior (β_f) and use this to directly derive a median value (θ_D) of the structural capacity, given the wind demand, for initiation of this failure mode. For this purpose, a lognormal distribution for structural capacity as a function of demand shall be assumed. Alternatively, it is permitted to estimate the median value directly.

B2.3 Incremental Dynamic Analysis

a. For each wind design scenario, perform a series of incremental nonlinear dynamic analyses consisting of application of the scenario load history at an index value of the wind demand parameter amplitude relative to the collapse initiation parameter (e.g., M_{OT} or V) for structural capacity. Repeat this analysis, with adjusted amplitudes, until one or more of the critical damage states reach their median value, as determined in B2.2(b).

b. For the set of wind design scenarios, determine the median value of the parameter (e.g., $\widehat{M_{OT}}, \hat{V}$) at which structural collapse initiates and the model uncertainty, β_m, which is computed as the COV of demand parameter at which the structural collapse initiates.

c. Determine an uncertainty associated with collapse, β_c, by combining uncertainties associated with modeling (β_m), failure uncertainty (β_f), and uncertainty associated with the wind tunnel testing's ability to predict actual loading on the structure, (β_T). The uncertainty, β_c, can be estimated by

$$\beta_c = \sqrt{\beta_m^2 + \beta_f^2 + \beta_T^2}$$

where

β_f = Uncertainty in collapse mode capacity, measured by an appropriate parameter (e.g., story drift, concrete compressive strain, column axial load) that can be used as a predictive parameter for collapse mode initiation. If the limit state associated with the collapse mode is relatively ductile (yielding, plastic hinging, and others, β_f should be no less than 0.12. If the limit state is nonductile (shear in RC, instability), β_f should be no less than 0.20.

β_T = Uncertainty associated with the demand on the structure reflected in the wind tunnel records used to identify the key responses for 10 design scenarios and the time history analyses that are used to identify one or more of the critical damage state demands. This uncertainty is determined by the quality and comprehensive nature of the professional services provided by the wind tunnel consultant. Typical values are expected to range between 0.20 and 0.30.

β_m = Uncertainty associated with fidelity in modeling the structural response to the key design scenarios, which are provided by the Engineer of Record. β_m shall not be less than 0.10.

Commentary: *Wind tunnel uncertainty, β_T, should be recommended by the wind consultant and generally should have a value on the order of 0.2. Modeling uncertainty, β_m, accounts for variation of response predictions from the analytical model relative to the actual structure, associated with estimates of damping, cracking assumptions in concrete stiffness, and similar uncertainties. This may range from about 0.1 for linear response to 0.2 or higher as nonlinearity in response increases. Uncertainty associated with the demand at which a damage state occurs, β_f, should be estimated considering available laboratory test data and judgment as to the effect of variability in materials quality, construction quality, and loading rate and pattern on damage state onset. Typically, β_c will have values on the order of 0.3 to 0.4. For example, if β_f = 0.15, β_T = 0.25 and β_m = 0.10, then β_c = 0.31.*

B.3 ACCEPTANCE CRITERION

For each wind design scenario, use the median value of the indicative capacity parameter at which collapse initiates as determined from B2.3b, and the uncertainty, β_c, as determined from B2.3c, to calculate the target reliability parameter, $X_{.01}$, at which there is a 0.01% conditional probability of failure ($P_F = 10^{-4}$ for a design wind event). The value of X determined by engineering analysis for each wind design scenario and associated collapse initiation mode shall equal or exceed the value, $X_{0.01}$, for the capacity parameter identified for that scenario, given the occurrence of a wind with the MRI stipulated for the risk category of the structure.

Commentary: *For lognormal distributions, the value of $X_{.01}$ at which there is a 0.01% conditional probability of collapse is the value that lies 3.72 standard deviations below the median value in log space. This can be easily calculated using the lognorm.inv function in Excel spreadsheets.*

The target reliability of 0.0001 (0.01%) has been judgmentally selected as a reasonable approximation of the system reliability that would be obtained from a structure that marginally meets the target reliability goals specified in ASCE Standard 7-16, Section 1.3 for critical elements, the failure of which could result in structural collapse. It is expected that some adjustment of this acceptance criteria will occur in the future, as data from actual buildings designed using performance-based procedures become available and can be evaluated.

Appenidx C
Method 3: Fully Coupled Assessment Method to Validate Collapse-Resistance and System Reliability of Structures to Wind Loading

C.1 PURPOSE

Alternative analysis approaches are allowed in Method 3 to evaluate structural collapse resistance and reliability for the Continued Occupancy Performance Objective. The approach described here is referred to as the Shakedown Method, which can provide a comprehensive procedure for evaluation member and system reliability while considering the inherent uncertainties affecting structural response. The Shakedown Method is based upon transforming the problem of solving non-linear dynamic equilibrium equations into the search for solutions to large-scale linear programing techniques. The use of this method permits direct evaluation of system and member reliability. The use of Method 3 requires peer review of this approach, assumptions, implementation, and findings.

Commentary: *Method 3 is also referred to the Dynamic Shakedown Method of analysis. Dynamic Shakedown is a state of force distribution under which the structure as well as its constituent members do not experience cyclic (reversing) plastic strain given application of a time history of load demands (Casciaro et al. 2002, Garcea et al. 2005, Chuang et al. 2017). A state of Dynamic Shakedown precludes reversing inelasticity (leading to low cycle fatigue) or instantaneous failure (plastic collapse). Determination of a Dynamic Shakedown solution also precludes unbounded accumulation of inelastic strains leading to ratcheting collapse.*

Dynamic Shakedown has the specific benefit of being highly computationally efficient thus permitting the consideration of hundreds or thousands of full storm non-linear response wind time histories in the order of hours.

Dynamic Shakedown Analysis differs from non-linear push over analysis in that push over analysis does not evaluate the possibility of reversing inelastic response. As such a push over analysis can not detect or rule out reversing plasticity leading to low cycle fatigue failure.

Alternatively, a structural system can be evaluated using traditional non-linear time history analysis techniques. With a sufficient number of analysis considering the variation in wind load histories as well as member material uncertainties, time history analysis can be used to demonstrate a level of reliability in agreement with the building code. Such an analysis can also demonstrate safety against collapse and conformance with the other acceptance criteria of this document. The computational resources necessary for such an evaluation are generally beyond those available to practitioners given the need for multiple suites of analysis demonstrating sound statistical reliability.

C.2 GENERAL PROCEDURE

Dynamic Shakedown analysis for reliability evaluation shall include a Monte Carlo procedure, or equivalent, for estimating the probabilistic performance metrics of a structure. The Monte Carlo analysis shall include consideration of relevant uncertainties that affect the structural response including member strength and stiffness uncertainty, wind, dead, and live loading uncertainty, and, modeling uncertainty.

Commentary: *Monte Carlo methods can efficiently evaluate complex problems of probability estimation through random sampling of uncertain parameters followed by test and evaluation of a modeled physical process. Uncertain parameters include the structural member properties as well as applied loading. Published resources describing material property uncertainty include but are not limited to the following:*

Reinforcing Bar and Concrete:

Nowak, A. S., and K. R. Collins. 2013. Reliability of Structures*. Boca Raton, FL: CRC Press.*

Structural Steel:

Bartlett, F., R. Dexter, M. Graeser, J. Jelinek, B. Schmidt, and T. Galambos. 2003. "Updating standard shape material properties database for design and reliability." Eng. J., 40*, 2−14.*

Zhang, H., B. R. Ellingwood, and K. J. Rasmussen. 2014. "System reliabilities in steel structural frame design by inelastic analysis." Eng. Struct. 81*, 341−348.*

Damping:

Celic, O. C., and B. R. Ellingwood. 2010. "Seismic fragilities for non-ductile reinforced concrete frames—Role of aleatoric and epistemic uncertainties." Struct. Safety, 32 *(1), 1−12.*

Healey, J., S. Wu, and M. Murga. 1980. "Structural building response review." NUREG/CR1423, Vol. I. US Nuclear Regulatory Commission, Washington DC.

Published resources describing loading uncertainty include but are not limited to the following:

Dead and Live Load:

Ellingwood, B., J. G. MacGregor, T. V. Galambos, and C. A. Cornell. 1982. "Probability based load criteria: Load factors and load combinations." J. Struct. Div. 108*, 978−997.*

Zhang, H., B. R. Ellingwood, and K. J. Rasmussen. 2014. "System reliabilities in steel structural frame design by inelastic analysis." Eng. Struct. 81*, 341−348.*

Wind Time History:

Bashor, R., T. Kijewski-Correa, and A. Kareem. 2005. "On the wind-induced response of tall buildings: The effect of uncertainties in dynamic properties and human comfort thresholds." 10th Americas Conference on Wind Engineering, Baton Rouge, Louisianna. Notre Dame, IN: American Association of Wind Engineering.

Diniz, S. M. C., F. Sadek, and E. Simiu. 2004. "Wind speed estimation uncertainties: Effects of climatological and micrometeorological parameters." Probabilistic Eng. Mech. 19*,361−371.*

Minciarelli, F., M. Gioffre, M. Grigoriu, and E. Simiu. 2001. "Estimates of extreme wind effects and wind load factors: Influence of knowledge uncertainties." Probabilistic Eng. Mech.16*, 331−340.*

Sadek, F., S. Diniz, M. Kasperski, M. Gioffre, and E. Simiu. 2004. "Sampling errors in the estimation of peak wind-induced internal forces in low-rise structures." J. Eng. Mech. 130 *(2), 235−239.*

C.3 ACCEPTANCE CRITERIA

Method 3 acceptance criteria shall be based upon established and accepted models for structural member response. System acceptance criteria shall be established according to Section 7.4.3 and 7.4.5, and as appropriate to the Performance Objectives.

> ***Commentary:*** *Method 3 is the most robust approach for building reliability determination within the prestandard. Use of Method 3 is specifically intended to permit the greatest design flexibility while demonstrating structure performance consistent with the reliability objectives.*

REFERENCES

Casciaro R., and G. Garcea. 2002. "An iterative method for shakedown analysis." *Comp. Meth. Appl. Mech. Eng. 191* (12), 5761−5792.

Chuang, W.-C., and S. M. J. Spence. 2017. "A performance-based design framework for the integrated collapse and non-collapse assessment of wind excited buildings. *Eng. Struct.150* (11), 746−758.

Garcea, G., G. Armentano, S. Petrolo, and R. Casciaro. 2005. "Finite elment shakedown analysis of two-dimensional structures." *Int. J. Num. Meth. Eng. 63* (3), 1174−1202.

Appendix D
Effective Strategies to Reduce Dynamic Wind-Induced Response

Effective strategies to reduce the dynamic wind-induced response of a building include structural refinement through alteration of the structural properties of the building (including mass and/or stiffness), or through the implementation of one or more of structural refinement, supplementary damping, or aerodynamic treatment (or optimization of the architectural form). A combination of all three control mechanisms is also effective.

D.1 STRUCTURAL REFINEMENT

To control wind-induced motion through structural refinement, often there is an increased structural cost through, for example, increasing the size of structural columns to improve the building stiffness.

For a building, increasing the stiffness to counter undesirable wind response can be difficult without increasing the mass. Dynamic wind acceleration in a tower is generally inversely related to the mass but increasing the mass without corresponding increase in lateral stiffness increases the natural period. In some cases, refinement (enhancement) of the structural system be found cost effective to manage wind effects. Substantial reduction in wind effects may not be practical or cost effective through structural modification alone.

In general, an increase in stiffness tends to lead to a reduction in dynamic wind motion. One exception to this rule is when a building is operating at or beyond its critical velocity for vortex shedding; in this case, an increase in stiffness may result in an increase in dynamic wind response.

D.2 SUPPLEMENTARY DAMPING

Auxiliary damping devices can be either passive, semi-active, or active and, depending on their degree of redundancy, can be aimed at mitigating dynamic wind action under continuous occupancy or operational events. Damping devices such as tuned mass or sloshing dampers require frequency tuning to the as-built frequency of the building for proper (and optimal) damping performance. Damping systems such as viscous dampers or viscoelastic dampers do not require frequency tuning and the overall damping achieved is stable with respect to changes in dynamic properties of the building. Dynamic wind motion varies roughly in inverse proportion to the square root of the damping ratio. Therefore, doubling or tripling the damping of a building achieves 30% to 40% reductions in the dynamic portion of the wind response, respectively.

Supplementary damping devices may be included in occupant comfort, operational, and/or continuous occupancy performance objective analysis provided the reliability of the device is commensurate with the hazard considered.

Continuous occupancy conditions should be evaluated with any passive or tuned device being out of operation, unless it can be demonstrated that the damping system offers a degree of reliability similar in performance to the MWFRS reliability.

D.3 AERODYNAMIC OPTIMIZATION

The architectural form of buildings heavily influences the impact of wind. Vortex-shedding characteristics are dependent on the basic tower shape and via the nondimensional quantity known as the Strouhal number. In some cases, vortex shedding, which occurs at a critical frequency, may occur at frequent wind recurrence intervals.

Aerodynamic modifications to the building form can help to control dynamic wind motion by altering the Strouhal number or by taking energy away from the vortex shedding action. Effective modifications include progressive recesses, slotted or chamfered corners, horizontal and vertical through-building openings or wind slots, porous tops, twisting or tapering of the structural form, and dropping off corners (e.g., Kwok 1998, Dutton and Isyumov 1990, Irwin and Baker 2005, Irwin 2007). Aerodynamic shape optimization can be a highly effective means of mitigating wind response but typically requires consideration early in the architectural design process.

In general, modifications to the building corners such as slotted or chamfered corners (Figure D-1) tend to need to be greater than approx. 5% to 10% of the building breadth to be beneficial. Twisting or tapering tends to need to be relatively dramatic to be substantially beneficial. If the dynamic wind action on the building is a result of interference with surrounding buildings, the dynamics can be difficult to control without making dramatic changes to the structural form.

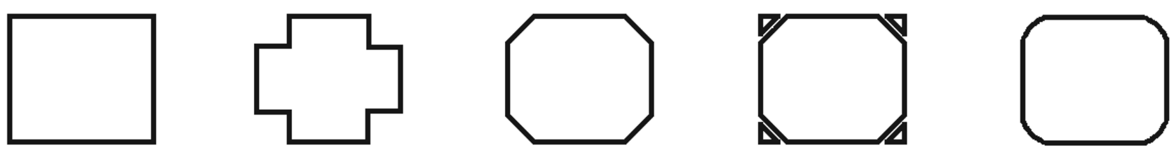


Figure D-1. Examples of corner modifications.

REFERENCES

Dutton, R., and N. Isyumov. 1990. “Reduction of tall building motion by aerodynamic treatments.” *J. Wind Eng. Ind. Aerod. 36* (1990), 739−747.

Irwin, P. A., and W. F. Baker. 2005. “The wind engineering of the Burj Dubai Tower.” In *Proc. of the Council on Tall Buildings and Urban Habitat, 7th World Congress*, Renewing the Urban Landscape, New York, October 16−19. Chicago: Council on Tall Buildings and Urban Habitat.

Irwin, P. 2009. “Wind engineering challenges of the new generation of super-tall buildings.” *J. Wind Eng. Ind. Aerod. 97* (7-8), 328−334.

Kwok, K. C. 1988. “Effect of building shape on wind-induced response of tall building.” *J. Wind Eng. Ind. Aerod. 28* (1-3), 381−390.