APPENDIX B—PERFORMANCE-BASED WIND DESIGN *(NEW APPENDIX)*

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B.1—Notation and Terminology

B.1.1 *Notation*

E_{ce} = expected modulus of elasticity of concrete, psi

f_{ce}' = expected compressive strength of concrete, psi

f_{ye} = expected yield strength for nonprestressed reinforcement, psi

f_{ue} = expected tensile strength for nonprestressed reinforcement, psi

h_{sx} = story height for story x

M_{ne} = flexural strength at section, determined using expected material strengths, in.-lb

P_{ne} = axial compressive strength of member, determined using expected material strengths, lb

P_{nte} = axial tensile strength of member, determined using expected material strengths, lb

R_n = nominal strength

$R_{ne,w}$ = expected strength of reinforced concrete member using f_{ce}' and f_{ye}

W_{MRI} = wind effect with specified mean recurrence interval (MRI)

Δ_L = deformation limit (strain, rotation, displacement)

$\delta_{x,w}$ = maximum story drift ratio expected in story x, according to analyses for wind demands. Drift ratio is calculated as relative difference of lateral displacement between the top and bottom of a story, divided by the story height.

ε_{ye} = expected yield strain of reinforcement

θ_{ye} = expected yield rotation of member determined using expected material strengths, radians

B.1.2 *Terminology*

action, deformation-controlled—action allowed to exceed the expected yield deformation of the element being evaluated.

action, force-controlled—action not allowed to exceed the design strength of the element being evaluated.

basis of design—formal document prepared by the licensed design professional expressing the performance objectives, acceptance criteria, analysis methods, and design methods to be used in the overall building design.

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.

expected strength, wind—strength of a member or cross section calculated in accordance with provisions and

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RB.1—Notation and Terminology

RB.1.1 *Notation*

E_{ce} = Expected modulus of elasticity of concrete is calculated using expected compressive strength of concrete

W_{MRI} = Wind effect with specified mean recurrence interval (MRI) depends on the risk category of the building and is provided in ASCE/SEI Prestandard for Performance-Based Wind Design.

RB.1.2 *Terminology*

action, deformation-controlled—Deformation-controlled actions are those under which elements exhibit acceptable degree of inelastic response and are deemed to have failed upon exceedance of a predefined deformation level or number of cycles.

action, force-controlled—Force-controlled actions are those under which elements exhibit limited ductility and are deemed to have failed upon exceedance of design strength.

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assumptions of this Code using expected material strengths provided in B.9.

performance-based wind design (PBWD)—alternative design procedure to the prescriptive provisions in the general building code and referenced standards, which considers direct evaluation of the wind demand on the structure and evaluates the building performance as it relates to occupant comfort, operational performance, and continuous occupancy, limited interruption performance objectives.

performance objective—specific desired outcome for an action, element, or system of a building during or following a wind event as chosen by the project stakeholders and licensed design professionals.

performance objective, continuous occupancy, limited interruption—specific desired outcome in which damage to the main wind-force-resisting system does not significantly disrupt or impair the continued operation and functionality of the structure.

performance objective, occupant comfort—specific desired outcome in which the accelerations from wind-induced sway motions remain within acceptable limits for occupant comfort and for equipment to maintain the functionality of the building.

performance objective, operational—specific desired outcome in which the main wind-force-resisting system remains essentially elastic and the building systems remain operational during the designated risk category-based event.

B.2—Scope

B.2.1 This appendix shall be in addition to the requirements of ASCE/SEI 7 and the general building code for performance-based wind design or evaluation of the main wind-force-resisting system of reinforced concrete structures.

B.2.2 The provisions of Appendix B shall be in addition to the provisions of Chapters 1 through 26, excluding Chapter 18 and Appendix A, unless specifically referenced.

B.3—General

B.3.1 Operational performance evaluation shall be permitted using (a), (b), or (c).

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performance objective, continuous occupancy, limited interruption—Continued operation and functionality of the structure is implicitly achieved when the main wind-force-resisting system is designed for a wind event corresponding to the designated building risk category to achieve the target reliability for structural stability that is consistent with the building code.

performance objective, occupant comfort—Although the occupant comfort performance objective is part of performance-based wind design, it is outside of the scope of Appendix B.

performance objective, operational—Members in reinforced concrete structures are considered cracked when concrete tensile stresses exceed the stress corresponding to the tensile strength. Even though the force-deformation relationship becomes nonlinear immediately after cracking, when performing the lateral analysis for the operational performance objective, it is assumed that the structure is essentially elastic, and its behavior is adequately represented using the secant stiffness for peak response for the risk category-based event.

RB.2—Scope

RB.2.1 Performance-based wind design is permitted by ASCE/SEI 7. This appendix is intended to supplement the ASCE/SEI Prestandard for Performance-Based Wind Design (2023). This appendix provides requirements specific to linear static, linear response history, and nonlinear response history analyses and design of reinforced concrete structures subjected to wind loading.

RB.2.2 Chapter 18 and Appendix A are not invoked using performance-based wind design unless specifically referenced. However, Chapter 18 is potentially applicable for seismic design depending on the Seismic Design Category and the seismic force-resisting system.

RB.3—General

RB.3.1 Acceptance criteria for the operational performance objective are provided in the ASCE/SEI Prestandard for Performance-Based Wind Design. Linear analyses

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(a) Linear static analysis
(b) Linear response history analysis
(c) Nonlinear response history analysis

B.3.2 Continuous occupancy, limited interruption performance evaluation shall use linear static analysis to evaluate minimum strength requirements followed by (a), (b), or (c):

(a) Method 1: Linear or nonlinear response history analysis meeting the requirements of B.11.
(b) Method 2: Nonlinear response history analysis with a reliability-based conditional probability assessment meeting the requirements of B.12.
(c) Method 3: Probabilistic nonlinear response history analysis with a fully coupled reliability assessment meeting the requirements of B.12.

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are typically used for models developed for the operational performance objective. Nonlinear analysis may be used for the operational performance objective.

B.3.2 Table RB.3.2 summarizes the applicability of the provisions of Appendix B as they are typically applied when using different methods to satisfy the continuous occupancy, limited interruption performance objective. Step 1 using linear static analysis is needed for all three methods. Method 1 may be conducted in two or three steps.

Table RB.3.2—Analysis procedure for evaluating Continuous Occupancy, Limited Interruption performance objective

Method	Step	Analysis type	Lateral loads	Sections of Appendix B to be satisfied	Notes
1	1	Linear static	Equivalent static lateral loads from building code[1]	B.11.2 Force-controlled actions and deformation-controlled actions	Minimum strength requirement for Method 1.
	2	Linear static or Linear response history	Equivalent static lateral loads from building code[1] or wind load histories in critical wind directions	B.11.3.1 Force-controlled actions B.11.3.2 Deformation-controlled actions	If acceptance criteria in B.11.3.1 are met and demands in members for deformation-controlled actions are less than $R_{ne,w}$, Step 3 is not needed. If acceptance criteria in B.11.3.1 or B.11.3.2 are not met, nonlinear analysis (Step 3) can be performed or the structure can be redesigned. Alternatively, it is permitted to implement Step 3 directly without performing Step 2.
	3	Nonlinear response history	Wind load histories in critical wind directions	B.11.4.1 Force-controlled actions B.11.4.2 Deformation-controlled actions	
2	1	Linear static	Equivalent static lateral loads from building code[1]	B.12.2 Force-controlled actions and deformation-controlled actions	Minimum strength requirement for Methods 2 and 3.
	2	Nonlinear response history	Wind load histories in critical wind directions	B.12.3 Target reliability of components and connections	Refer to ASCE/SEI Prestandard for Performance-Based Wind Design.
3	1	Linear static	Equivalent static lateral loads from building code[1]	B.12.2 Force-controlled actions and deformation-controlled actions	Minimum strength requirement for Methods 2 and 3.
	2	Probabilistic nonlinear response history	Wind load histories in all wind directions	B.12.3 Target reliability of lateral load resisting system	Refer to ASCE/SEI Prestandard for Performance-Based Wind Design.

[1]Wind tunnel equivalent static loads can be used instead of loads from the directional procedure of the building code if the wind tunnel loads are scaled such that the base overturning moment is not less than 80% of that using the general building code loads (or not less than 80% of the base shear if the first mode period is less than 1 second)

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B.3.3 Action Classifications in B.7 and Required Strength and Acceptance Criteria in B.10 through B.12 for the continuous occupancy, limited interruption performance objective shall take precedence over those of the ASCE/SEI Prestandard for Performance-Based Wind Design.

RB.3.3 Action Classifications in B.7 and Acceptance Criteria in B.10 through B.12 provide a comprehensive design approach following the intent of ASCE/SEI 7 and the general building code. Because of inconsistencies between ACI 318 and the ASCE/SEI Prestandard for Performance-Based Wind Design in the approach to Action Classifications and Acceptance Criteria for reinforced concrete members, the requirements in this Appendix take precedence over those of the ASCE/SEI Prestandard for Performance-Based Wind Design. For example, a moment frame column under axial load is considered a force-controlled element in the ASCE/SEI Prestandard for Performance-Based Wind Design. However, according to ACI CODE-318, a concrete column with axial load and moment may be force-controlled or deformation-controlled based on the level of axial load.

B.3.4 Independent structural design review consistent with B.14 shall be required when Appendix B provisions are used.

B.3.5 The licensed design professional shall provide justification for any interpretation required for the application of Appendix B and, if accepted by the independent structural design reviewer, justification shall be provided to the building official for acceptance.

B.4—Wind loading

B.4.1 Wind hazard analysis, wind tunnel testing, and wind tunnel data analysis shall satisfy the requirements of the general building code and the requirements of B.4.2 through B.4.4.

RB.4—Wind loading

RB.4.1 Design of concrete structures for wind requires characterization of the wind loading and responses.

B.4.2 Probabilistic wind climate analysis shall be used to determine wind speeds, directionality, and duration of wind loading.

RB.4.2 Chapter 6 of ASCE/SEI 49-21 (Wind Tunnel Testing for Buildings and Other Structures) and Chapter 26 of ASCE/SEI 7-22 (Minimum Design Loads and Associated Criteria for Buildings and Other Structures) provide guidance on wind hazard analysis. Code-specified basic wind speeds for a range of mean recurrence intervals (MRIs) can be obtained for any location in the United States using the ASCE/SEI 7 Hazard Tool.

B.4.3 Appropriate wind tunnel test methodologies shall be used in the determination of load and response effects of interest.

RB.4.3 Appropriate methodologies are discussed in Chapter 2 of ASCE/SEI 49-21 (Wind Tunnel Testing for Buildings and Other Structures) and Chapter 31 of ASCE/SEI 7-22 (Minimum Design Loads and Associated Criteria for Buildings and Other Structures).

B.4.4 The duration of wind load histories from wind tunnel testing shall be of sufficient length to enable the peak wind event duration to be modeled reliably, permit evaluation of nonlinear response within the Performance-Based Wind Design (PBWD) framework, and permit development of a loading protocol for testing deformation-controlled elements.

RB.4.4 The ASCE/SEI Prestandard for Performance-Based Wind Design, Chapters 3 through 5 of ASCE/SEI 49-21 (Wind Tunnel Testing for Buildings and Other Structures), and Chapter 31 of ASCE/SEI 7-22 (Minimum Design Loads and Associated Criteria for Buildings and Other Structures) provide guidance on wind tunnel data analysis. The peak responses and cumulative damage to components of the structure depend on wind demand amplitude, duration, and history. Traditional design of buildings and structures

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for wind assumes an equivalent static wind load approach. Effective static wind loads are developed for the target Mean Recurrence Interval (MRI) based on statistics of the wind tunnel measurements and the wind climate analysis.

For Performance-Based Wind Design (PBWD) of structures, it is prudent to record wind load histories of sufficient duration, and at sufficiently small time steps, to permit the structure-wind dynamic response of interest to be adequately represented.

The duration of strong wind events may be extracted from the statistics of the wind data, storm simulation, or both. For hurricane-prone regions, statistical investigations of event durations can be found in Kopp et al. (2021) and Wang and Wu (2022).

B.5—Load factors and combinations

B.5.1 For evaluating the continuous occupancy, limited interruption performance objective, required strength U shall be at least equal to the effects of factored loads in Table B.5.1.

Table B.5.1—Load combinations

Method of evaluation	Load combination	Equation
Method 1	$U = 1.0D + 1.0L + (0.5L_r \text{ or } 0.3S \text{ or } 0.5R) + 1.0W_{MRI}$	(B.5.1a)
	$U = 0.9D + 1.0W_{MRI}$	(B.5.1b)
Methods 2 and 3	$U = 1.0D + 1.0L + 1.0W_{MRI}$	(B.5.1c)

B.5.1.1 In combination Eq. (B.5.1a), S shall be taken as either the flat roof snow load or the sloped roof snow load as specified in ASCE/SEI 7.

B.5.1.2 The load factor on live load L in Eq. (B.5.1a) and (B.5.1c) shall be permitted to equal 0.5 except for (a), (b), or (c):

(a) Areas occupied as garages
(b) Areas occupied as places of public assembly
(c) Areas where L is greater than 100 lb/ft^2

B.5.1.3 Load combination B.5.1b need not be considered if (a) and (b) are satisfied:

(a) Sum of the expected in-service live load L over the entire structure does not exceed 25% of the total dead load D
(b) Live load intensity over at least 75% of the structure is less than 100 lb/ft^2

RB.5—Load factors and combinations

RB.5.1 Load combinations for response history analysis used in conjunction with this appendix are intended to follow the ASCE/SEI Prestandard for Performance-Based Wind Design (PBWD). For nonlinear response history analysis, the principles of linear superposition do not apply. Therefore, it would be incorrect to conduct separate analyses considering various loads and then combine the load effects. Instead, as stated in the ASCE/SEI Prestandard for Performance-Based Wind Design, it is necessary to conduct an analysis for each factored load combination and take the design value as the demand from the governing load combinations of the analysis results. For any nonlinear analysis including wind load effects, gravity loads are to be applied to the model first, and then the wind load histories are applied in the presence of the gravity loads.

RB.5.1.3 The exception in B.5.1.3 is based on ASCE/SEI Prestandard for Performance-Based Wind Design.

B.6—Modeling and analysis

B.6.1 Analytical models shall be three-dimensional.

RB.6—Modeling and analysis

RB.6.1 Three-dimensional models represent proper structural behavior under the spatial distribution of wind loads. Effects of bidirectional wind loading and torsion cannot be adequately represented with two-dimensional models. Certain elements of reinforced concrete structures can be designated as part of the main wind-force-resisting system. These elements should be included in the three-dimensional model. Other elements that may be essential to the gravity-load-resisting system may be designated as not part of the main wind-load-resisting system and may be excluded from the analysis model if the second-order effects due to gravity loads on such elements are accounted for in the analysis. Refer to B.6.3 for elements that are not part of the main wind-load-resisting system and should be included in the analysis model. Performance of elements that are not modeled should be evaluated for forces, deformations, or both due to compatibility with the lateral-load-resisting elements.

B.6.2 Models for linear static and linear response history analyses shall meet the requirements of B.6.2.1. Models for nonlinear response history analyses shall meet the requirements of B.6.2.2.

RB.6.2 Analysis procedures depend on the performance objective. The ASCE/SEI Prestandard for Performance-Based Wind Design outlines the requirements for analysis procedures permitted for each performance objective.

Linear static analysis of a structural system uses linear elastic properties and includes second-order effects and spatial distribution of mass and loads. Linear elastic properties for each component should reflect effective stiffness values at the expected load level. Equivalent static wind loads that include effects of dynamic response are applied. Linear analyses are typically used for models developed for the operational performance objective and minimum strength checks for Methods 1, 2, and 3 as described in Table RB.3.2.

Linear response history analysis of a structural system is based on linear elastic properties and includes second-order effects and spatial distribution of mass and loads. Linear elastic properties should reflect the effective stiffness at the expected load level. Along-wind, crosswind, and torsional wind load histories determined from wind tunnel testing are applied to the model, and dynamic response is computed step-by-step in the time domain or by use of frequency domain procedures.

Nonlinear response history analysis is similar to linear response history analysis except that the structural system explicitly accounts for changes in element and connection stiffness and strength due to cyclic nonlinear response. Along-wind, crosswind, and torsional wind loads are applied simultaneously through the application of wind load histories, and dynamic response is computed step-by-step in the time domain.

B.6.2.1 *Requirements for linear analysis models*

B.6.2.1.1 Models for linear analyses shall consider the expected effective stiffness values of members at the force level appropriate for different performance objectives and shall consider second-order effects.

RB.6.2.1 *Requirements for linear analysis models*

RB.6.2.1.1 Second-order analysis should consider the influence of axial loads, presence of cracked regions along the length of the member, and effects of load duration.

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B.6.2.1.2 Effective stiffness in accordance with B.8 shall be permitted for linear static and linear dynamic analysis.

B.6.2.2 *Requirements for nonlinear analysis models*

B.6.2.2.1 Models for nonlinear analysis shall consider the effects of stiffness decay, which shall be substantiated by applicable physical test data and shall not be extrapolated beyond the limits of testing.

B.6.2.2.2 Nonlinear force-deformation curves used to estimate load-deformation relationships for elements in the analytical model shall be developed based on test results.

B.6.2.2.3 Substantiating laboratory test data for force-deformation relationships shall include deformation or load reversals with a number of inelastic cycles greater than or equal to that expected at wind demands corresponding to the continuous occupancy, limited interruption performance objective.

B.6.2.2.4 Wind loads applied to the model shall represent the effects of spatial distribution of loads vertically and horizontally throughout the building. The wind loads shall represent the spectral content established through wind tunnel testing.

B.6.2.2.5 It shall be permitted to use effective stiffness values in accordance with B.8 in nonlinear response history analysis for elements that remain elastic.

B.6.3 Analytical models for evaluation of the continuous occupancy, limited interruption performance objective shall include gravity-load-resisting elements unless it is demonstrated that their design is not affected by their interaction with the main wind-force-resisting system.

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RB.6.2.2 *Requirements for nonlinear analysis models*

RB.6.2.2.2 Multiple element formulations and material models are appropriate for use in nonlinear dynamic analysis of concrete structures. ASCE/SEI 41, ACI PRC-374.3, ACI CODE-369.1, and NIST GCR 17-917-46 provide guidance on modeling for seismic design and may be applicable to wind design. Force-deformation curves provided in these documents for seismic design, however, may not necessarily be appropriate for wind design. Mean values of modeling parameters from experimental data should be used as is recommended in ASCE/SEI 41, ACI PRC-374.3, ACI CODE-369.1, and NIST GCR 17-917-46 because this approach produces more reliable results.

RB.6.2.2.3 Duration of loading is important to consider any ratcheting of drifts in a particular direction and to account for the effect of fatigue on the behavior of the component.

RB.6.2.2.4 Effects of distribution of wind loads may be simulated with concentrated lateral loads applied to rigid floor diaphragms with additional moments to account for torsion. In consultation with the wind testing consultant, wind loads may be applied in multi-story loading blocks provided the resulting main wind-force-resisting system shear and flexure effects are maintained.

RB.6.3 It is difficult to accurately determine forces on certain elements due to compatibility of displacements unless they are modeled with the main wind-force-resisting system. Elements listed below are examples of gravity-load-resisting elements that should be modeled regardless of whether they are designated to resist lateral wind load:

(a) Columns in proximity to structural walls
(b) Columns supporting or supported by transfer beams
(c) Sloping columns
(d) Transfer beams

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Performance of elements that are not modeled should be evaluated for forces, deformations, or both due to compatibility with the lateral-load-resisting elements.

B.7—Action classification and requirements

B.7.1 Actions shall be classified as deformation-controlled or force-controlled in accordance with B.7.2 and B.7.3, respectively, and shall satisfy the requirements of B.10 through B.12.

RB.7—Action classification and requirements

B.7.2 *Deformation-controlled actions*

B.7.2.1 The following actions shall be permitted to be designated as deformation-controlled:

(a) Moment in slabs, beams, and coupling beams
(b) Shear in diagonally reinforced coupling beams that meet the requirements of 18.10.7.4
(c) Moment combined with axial force in tension-controlled columns
(d) Moment in walls when combined with axial force for walls controlled by tensile yielding of longitudinal reinforcement.
(e) Axial tensile force
(f) Other actions accepted by the independent structural design reviewer based on substantiating test data or analysis

RB.7.2 *Deformation-controlled actions*

RB.7.2.1 Refer to B.11.3.3 for determination of columns that may be classified as deformation-controlled. Yielding is permitted in members with deformation-controlled actions provided there is capability to resist gravity load. Therefore, axial force and moment in columns that are classified as tension controlled are permitted to be considered as deformation-controlled actions whereas axial force and moment in compression-controlled or transition columns are considered as force-controlled actions. Axial force and flexure in deep foundations may be considered as a deformation-controlled action depending on the strain demand in the reinforcing steel following the requirements of B.7.2.1(e). Similarly, flexure in shallow foundation members, including spread footings and mat foundations, may be considered a deformation-controlled action.

B.7.3 *Force-controlled actions*

B.7.3.1 Actions not designated as deformation-controlled shall be classified as force-controlled.

B.8—Effective stiffness

B.8.1 Methods that include the effects of deformations due to flexure, shear, axial elongation or shortening, and slip of reinforcement along embedment length shall be permitted to determine the stiffness of an element subjected to the level of wind force considered. Alternatively, effective stiffness values substantiated by physical test data shall be permitted.

RB.8—Effective stiffness

RB.8.1 It is important to recognize that the effective stiffness of a reinforced concrete member depends on the demand level, and wind gust effects increase as the structural stiffness decreases. Software for analysis is capable of directly calculating deformations due to flexure, shear, and axial forces, including the effects of cracking. Additional reduction in stiffness may occur due to slip of longitudinal reinforcement embedment. The effective stiffness should consider the combination of these effects. If such effects are deemed important to the performance of the structure, appropriate assumptions should be included in the analytical model, either directly or by adjustment of stiffness. Effective stiffness values are often used in linear analysis by adjusting the elastic stiffness of the gross concrete section to account for such additional deformations. Table A.8.4 provides effective stiffness values for components near the onset of yielding. Higher values may be appropriate for components subjected to lower demand levels.

B.8.2 Effective member stiffness values shall be determined based on the level of cracking anticipated as a result of the combined effects of applied forces, displacements,

RB.8.2 Effects of cracking on stiffness reduction can be considered directly by using models that represent stiffness reduction as calculated stress reaches the cracking stress or indirectly by using the effective stiffness as a fraction of

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and volume change associated with shrinkage, temperature, and creep.

B.8.3 If yielding of reinforcement or nonlinear response is anticipated as a result of the combined effects of applied forces, displacements, and volume changes associated with shrinkage, temperature, and creep, the structural model shall be capable of representing member stiffness for loading near the onset of inelastic response and for post-yield behavior.

B.8.4 It shall be permitted to represent effective member stiffness based on the secant stiffness to yield level based on analysis of physical test data.

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the gross section stiffness. If the latter approach is used, the degree of stiffness reduction should be consistent with the degree of cracking anticipated considering the level of wind loading. The effective member stiffness is also affected by the large number of loading cycles associated with wind loading.

RB.8.3 After cracking, reinforced concrete members exhibit a reduction in stiffness as reflected in the change in slope at point B in the idealized force-deformation relationship shown in Fig. RB.8.3. Stiffness reduction past the yield point (point C in Fig. RB.8.3) should be included in the behavioral model (Abdullah et al. 2020, 2021). The deformation limit Δ_L is determined in accordance with B.11.4.2, which will typically exceed the deformation associated with $R_{ne,w}$.

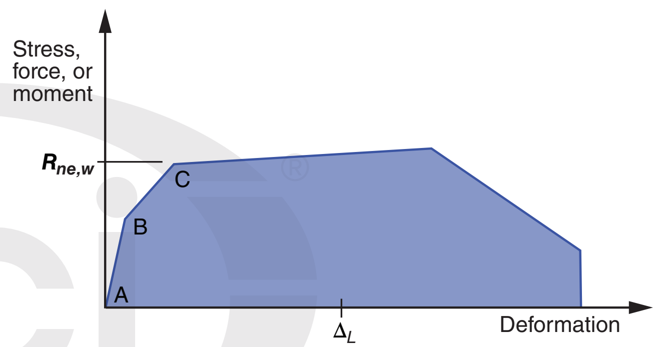

Fig. RB.8.3—Idealized force-deformation relationship.

RB.8.4 These provisions permit rational methods for evaluation of stiffness, which will depend on the nature of the building, location, demand levels, exposure, and geometry. These stiffness assumptions will be reviewed and accepted by the independent structural design reviewer as required by B.14. The values of effective stiffness for the continuous occupancy, limited interruption performance objective should consider demands corresponding to member yielding capacities. The effective stiffness of prestressed beams and slabs is expected to be greater than that of conventionally reinforced beams and slabs. Columns supporting outriggers can be subjected to tensile forces under wind loads, and the effective axial stiffness of columns under tension should be reduced based on the level of axial strain (ACI PRC-224.2). Industry standard is to use 1 and 10-year MRI winds for the occupant comfort performance objective. The ASCE/SEI Prestandard for Performance-Based Wind Design considers 10-year to 50-year MRI winds for the operational performance objective based on the risk category of the building. Similarly, 700-year to 3000-year MRI wind, based on the risk category of a building, are used for the continuous occupancy, limited interruption performance objective. Accordingly, it would be appropriate to use different effective stiffness values for evaluating the occupant comfort performance objective, the operational performance objective, and

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the continuous occupancy, limited interruption performance objective. Recommended values for effective stiffness for reinforced concrete members subjected to various wind load demands are given in ACI SP-240 (Horvilleur et al. 2006) and Manuals and Reports on Engineering Practice, MOP 243 (2020).

B.9—Expected material strength

B.9.1 Expected material strength shall be defined based on applicable project-specific data or data from projects using similar materials and construction.

B.9.2 If applicable data are not available, expected material strengths provided in Table B.9.2 shall be permitted to be used.

Table B.9.2—Expected material strengths

Concrete $f_{ce}' = 1.3f_c'$[1]						
Reinforcing steel						
Steel type	Grade 60		Grade 80		Grade 100	
	A615	A706	A615	A706	A615	A1035
f_{ye}	71,000	69,000	89,000	87,000	107,000	128,000
f_{ue}	105,000	95,000	120,000	115,000	138,000	163,000

[1]Expected strength f_{ce}' is the strength assumed to occur approximately 1 year or later.

RB.9—Expected material strength

RB.9.2 The multiplier of 1.3 on f_c' may be smaller for high-strength concrete and may be affected by the use of fly ash and other additives. Refer to ACI PRC-232.2 for discussion of effects of fly ash. Values provided in Table B.9.2 are based on data reported by Abdullah et al. (2022) and Mander and Matamoros (2019). ASTM A706 Grade 100 values are not in the table due to limited data.

B.10—Required strength and acceptance criteria for continuous occupancy, limited interruption performance objective—general

B.10.1 The continuous occupancy, limited interruption performance objective shall be evaluated using any of the three methods provided in B.3.2 and shall meet the requirements of B.10.2 through B.10.5.

B.10.2 Nominal strength R_n shall be in accordance with Chapter 22.

B.10.3 Expected strength $R_{ne,w}$ is permitted to be defined in accordance with the nominal strength provisions of Chapter 22, with f_{ce}' substituted for f_c' and f_{ye} substituted for f_y.

B.10.4 ϕ shall be determined in accordance with Chapter 21.

B.10.5 For any member that is part of the lateral force resisting system, response beyond the onset of strength decay in any of the response history analyses of this Appendix shall not be permitted.

RB.10—Required strength and acceptance criteria for continuous occupancy, limited interruption performance objective—general

RB.10.1 The ASCE/SEI Prestandard for Performance-Based Wind Design provides three methods to evaluate the performance objective for continuous occupancy, limited interruption. Method 1 requires provision of minimum strength for equivalent static wind loads, followed by linear response history analysis or nonlinear response history analysis. Refer to Figure RB.11.1 for the Method 1 flowchart. Methods 2 and 3 require minimum strength for equivalent static wind loads and use of nonlinear response history analysis for carrying out reliability-based conditional probability assessment and fully coupled reliability assessment respectively.

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B.10.6 Method 1 requirements and acceptance criteria shall be in accordance with B.11.

B.10.7 Method 2 and Method 3 requirements and acceptance criteria shall be in accordance with B.12.

B.10.8 Where analysis indicates an element is yielding, design shear force shall be taken as 1.1 times the forces calculated by nonlinear response history analysis.

B.11—Required strength and acceptance criteria for continuous occupancy, limited interruption performance objective by Method 1

B.11.1 If Method 1 is used, the requirements of B.11.2 through B.11.4 shall be satisfied.

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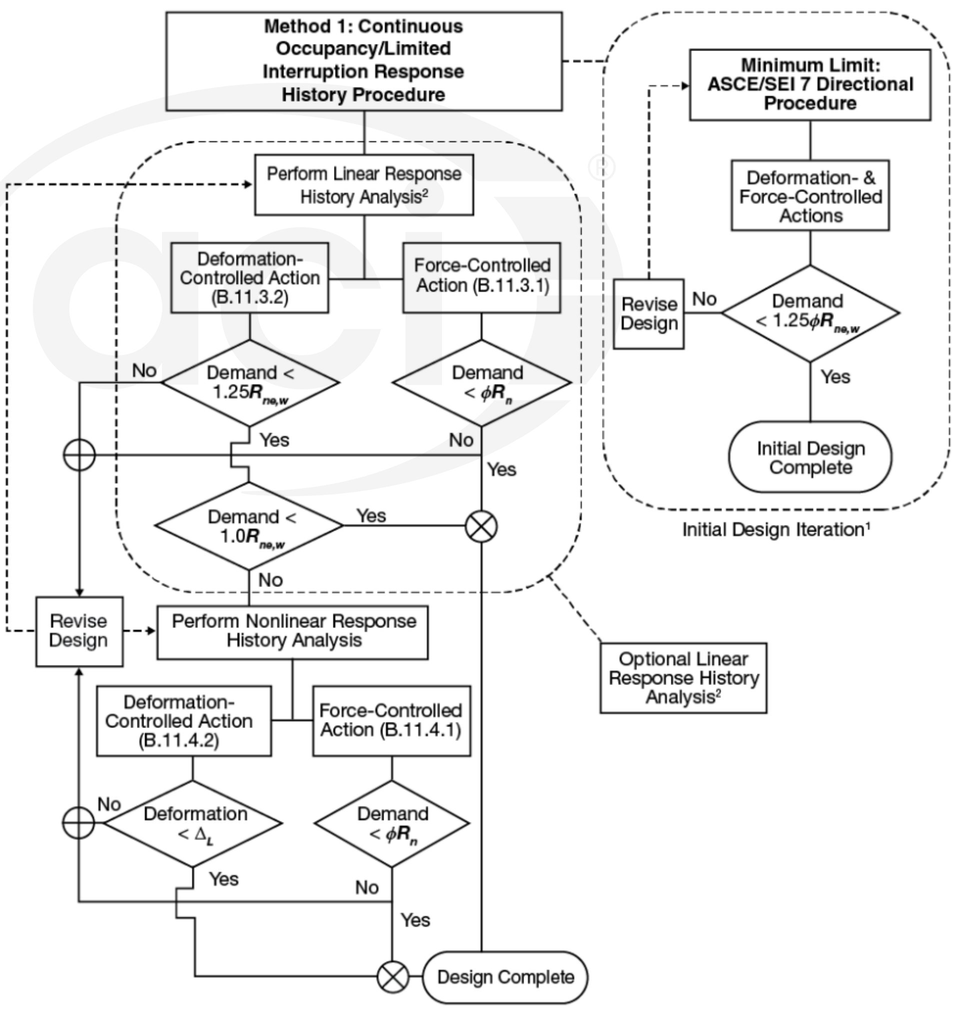
RB.10.8 Design shear forces are amplified to account for actual yield strength variability versus f_{ye}.

RB.11—Required strength and acceptance criteria for continuous occupancy, limited interruption performance objective by Method 1



Notes:
1. Minimum requirement utilizing the ASCE/SEI 7 Directional Procedure or vertical distribution from wind tunnel static loads.
2. It is permitted to omit the linear response history analysis and proceed directly to a nonlinear response history analysis.

Fig. RB.11.1—Method 1 for continuous occupancy, limited interruption performance objective.

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B.11.2 Required strength from a linear analysis using equivalent static wind loads associated with force-controlled and deformation-controlled actions shall not exceed $1.25\phi R_{ne,w}$.

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RB.11.2 The minimum strength provisions in the ASCE/SEI Prestandard for Performance-Based Wind Design stipulate lower bound limits for demands from the static wind loads according to ASCE/SEI 7 or the wind tunnel procedure according to ASCE/SEI 7-22 Chapter 31. Minimum strength for members with force-controlled and deformation-controlled actions is established by requiring an analysis using equivalent static wind loads and associated acceptance criteria to provide a consistent minimum strength level regardless of response history analysis results. Note that if an equivalent static analysis is performed for Method 1, ϕR_n will govern for force-controlled actions in accordance with B.11.3.1.

B.11.3 If required strength is calculated using linear static or linear response history analysis, the requirements of B.11.3.1 through B.11.3.3 shall be satisfied.

RB.11.3 The ASCE/SEI Prestandard for Performance-Based Wind Design permits elastic static or linear response history analysis. A nonlinear response history analysis may be required according to B.11.4.

B.11.3.1 Required strength in members for force-controlled actions shall not exceed ϕR_n.

B.11.3.2 Required strength in members for deformation-controlled actions shall not exceed $1.25R_{ne,w}$.

RB.11.3.2 Yielding is permitted in members for deformation-controlled actions provided there is ability to resist gravity load post-yielding.

B11.3.3 For columns, where P_u and M_u occur simultaneously, (a) or (b) shall be satisfied:

(a) If the column axial load is less than or equal to the design axial strength associated with the tension-controlled limit, the action shall be permitted to be classified as deformation-controlled and B.11.3.2 shall apply. The tension-controlled limit shall be determined based

RB11.3.3 Actions can only be classified as deformation-controlled if the axial load is below the tension-controlled limit to ensure there is ability to resist gravity loads. Refer to Fig. RB.11.3.3 for a representative interaction curve indicating the distinction between the strengths associated with deformation-controlled and force-controlled actions for combined axial load and moment. Refer to 21.2.2 for more information.

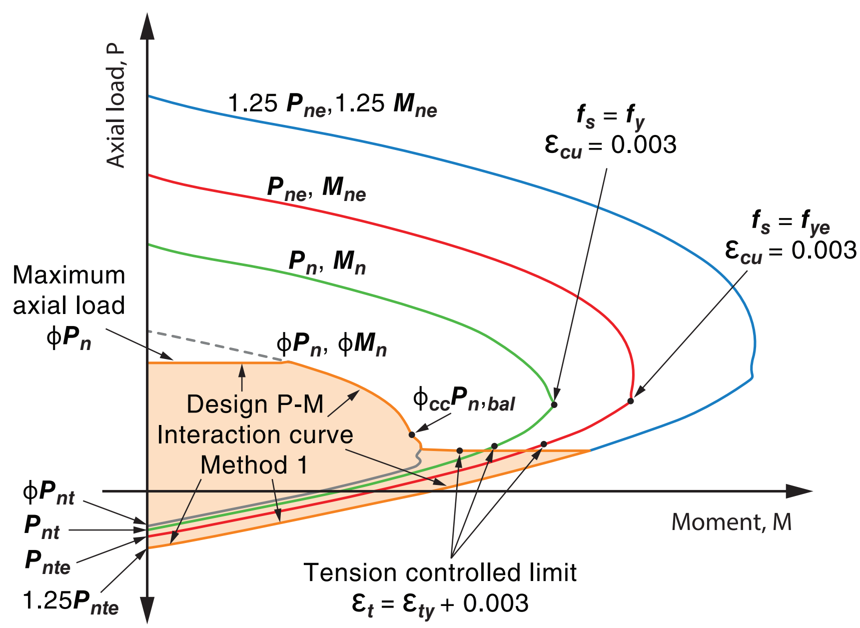

Fig. RB.11.3.3—P-M diagrams for Method 1.

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on the ϕ factor in accordance with Chapter 21 and the nominal strength in accordance with Chapter 22.

(b) The action shall be classified as force-controlled and B.11.3.1 shall apply.

B.11.4 For cases where linear response history analysis according to B.11.3 indicates required strength in members for deformation-controlled actions exceed $R_{ne,w}$, or if a linear response history analysis is not performed, a nonlinear response history analysis shall be required. The nonlinear response history analysis shall satisfy the requirements of B.11.4.1 through B.11.4.2.

B.11.4.1 Required strength in members for force-controlled actions shall not exceed ϕR_n.

B.11.4.2 The response of members with deformation-controlled actions shall not exceed the deformation limit Δ_L, where $\Delta_L$shall be determined by (a), (b), or (c):

(a) For beams and columns modeled using lumped plasticity models, Δ_L, evaluated at the chord rotation, shall not exceed 1.5 θ_{ye}.

(b) For walls modeled using distributed plasticity models, Δ_L shall be evaluated using the average vertical strain at the extreme compression and tension fibers over a story height, and the strain shall not exceed the greater of 0.0015 in compression and 1.5ε_{ye} in tension.

(c) Other values accepted by the independent structural design reviewer based on substantiating test data or analysis.

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RB.11.4.2 The deformation limit on beams and columns is established to provide a reliable mechanism to dissipate energy without compromising strength and stability of the member or structure (Abdullah and Wallace 2021). The expected yield rotation may be determined as the rotation associated with the expected strength based on the laboratory test results used to develop the nonlinear force-deformation curves required by B.6.2.2.2. For walls, a higher value of average vertical strain over the story height may be permitted where confinement reinforcement consistent with a special boundary element (Section 18.10.6.4) is provided. Inelastic strain at 1.5 times reinforcement yield should be limited to approximately 10 cycles, unless higher limits are shown acceptable through testing. A higher number of lesser magnitude inelastic cycles are acceptable if validated through laboratory testing or calculations using an approach such as the Coffin Manson relationship (Coffin 1954). The deformation limit may also be substantiated by test data or analysis provided that the testing meets or exceeds the magnitude and quantity of inelastic cycles required by the response history analysis. The testing or analysis should demonstrate adequate performance at an appropriate number of cycles to prevent low-cycle fatigue failure.

B.12—Required strength and acceptance criteria for continuous occupancy, limited interruption performance objective by Methods 2 and 3

B.12.1 If Method 2 or 3 is used, the requirements in B.12.2 through B.12.5 shall be satisfied.

RB.12—Required strength and acceptance criteria for continuous occupancy, limited interruption performance objective by Methods 2 and 3

RB.12.1 Method 2 outlined in the ASCE/SEI Prestandard for Performance-Based Wind Design is based upon a conditional probability assessment in which the response of the structure is characterized through nonlinear response history analysis. Method 3 is based on a fully coupled reliability-based assessment in which the response of the structure is characterized through nonlinear response history analysis or other appropriate nonlinear analysis methods. The two methods differ as follows:

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Method 2 requires a design wind scenario to be determined where structural performance is characterized using simplified probabilistic calculations.

Method 3 requires the consideration of many possible wind scenarios and does not consider simplified probabilistic calculations in estimating performance.

B.12.2 Required strength from a linear analysis using equivalent static wind loads associated with force-controlled and deformation-controlled actions shall not exceed $1.5\phi R_{ne,w}$.

RB.12.2 The minimum strength provisions in the ASCE/SEI Prestandard for Performance-Based Wind Design stipulate lower bound limits for demands estimated from the static wind loads according to ASCE/SEI 7 or the wind tunnel procedure according to ASCE/SEI 7-22 Chapter 31. Minimum strength for members with force-controlled and deformation-controlled actions is established by requiring an analysis using equivalent static wind loads and associated acceptance criteria to provide a consistent minimum strength level regardless of response history analysis results. Yielding is permitted in members for deformation-controlled actions provided there is the ability to resist gravity loads after yielding has occurred. In addition to the acceptance criteria for members with force-controlled and deformation-controlled actions, the structure also needs to meet the target reliability provided in ASCE/SEI 7.

B.12.3 Using nonlinear response history analysis, members with force-controlled actions, deformation-controlled actions, or both in the main wind-force-resisting system shall satisfy the target reliability stated in the general building code. The reliability analysis shall consider uncertainties in capacity in accordance with B.12.4 and B.12.5. Depending on the capacity model adopted, uncertainty in the parameters of the model shall be included and documented for peer review.

RB.12.3 The ASCE/SEI Prestandard for Performance-Based Wind Design provides a framework to demonstrate satisfactory response using reliability analysis. Target reliabilities for components and connections are based on ASCE/SEI 7-22 Table 1.3.1a. Target reliabilities for the system are provided in the ASCE/SEI Prestandard for Performance-Based Wind Design. With more detailed reliability analysis, minimum strength requirements using static loads (B.12.2.1) are reduced compared to the methodology used in B.11.2.

B.12.4 Compressive strength of concrete shall consider a lognormal probability distribution with f_{ce}' for the mean, where f_{ce}' is defined in B.9, and a coefficient of variation ≥ 0.2.

RB.12.4 The value for the minimum coefficient of variation was established based on analysis of material test data reported in Nowak et al. (2011).

B.12.5 The yield strength of reinforcement shall consider a lognormal probability distribution with f_{ye} for the mean, where f_{ye} is defined in B.9, and a coefficient of variation ≥ 0.1.

RB.12.5 The value for the minimum coefficient of variation was established based on analysis of material test data reported in Nowak et al. (2011).

B.13—Detailing requirements

B.13.1.1 Requirements of B.13.2 through B.13.8 shall apply to portions of the main wind-force-resisting system where deformations calculated by nonlinear response history analysis exceed yield deformations.

B.13.1.2 The requirements of B.13.9 shall apply to all elements or parts of elements that are not part of the main wind-force-resisting system.

B.13.1.3 If requirements in B.13 conflict with other requirements in this Code, the most conservative requirement(s) shall govern.

RB.13—Detailing requirements

RB.13.1.1 The requirements of this section are intended to ensure moderately ductile response consistent with the deformation and force limits specified in this appendix and avoid brittle failures.

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B.13.2 *Structural walls and wall piers*

B.13.2.1 The distributed web longitudinal reinforcement ratio ρ_ℓ in structural walls and wall piers shall not be less than 0.0025.

B.13.2.2 Structural walls or wall piers with $h_w/\ell_w \geq 2.0$ shall satisfy (a) through (c).

(a) Longitudinal reinforcement ratio within $0.15\ell_w$ from the end of a vertical wall segment, and over a width equal to the wall thickness, shall be at least $6\sqrt{f_c'}/f_y$ at the critical section.
(b) Longitudinal reinforcement required by (a) shall extend vertically above and below the critical section at least the greater of ℓ_w and $M_u/3V_u$.
(c) No more than 50% of the reinforcement required by (a) shall be terminated at any one section.

B.13.3 *Beams*

B.13.3.1 Provisions 9.6.1.3, 9.6.2.2, and Table 9.6.3.1 shall not apply to beams.

B.13.3.2 A minimum area of shear reinforcement, $A_{v,min}$, as defined in Table 9.6.3.4, shall be provided along the entire length of beams for which analysis indicates yielding of longitudinal reinforcement may occur.

B.13.3.3 Spacing of transverse reinforcement shall not exceed the smaller of (a) through (d) in beams for which analysis indicates yielding of longitudinal reinforcement may occur:

(a) $d/4$
(b) Eight times the diameter of the smallest longitudinal bar enclosed
(c) 24 times the diameter of the smallest transverse reinforcing bar
(d) 12 in.

B.13.3.4 Beams shall have at least two continuous top and two continuous bottom bars. Continuous bottom bars shall

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RB.13.2 *Structural walls and wall piers*

Minimum longitudinal reinforcement is intended to result in sectional flexural capacities exceeding the cracking moment to promote the formation of well-distributed flexural cracks in yielding regions. The critical section refers to any section where flexural yielding is indicated by analysis to occur. The requirement for the longitudinal reinforcement ratio within $0.15\ell_w$ from the end of a vertical wall segment is the same as 18.10.2.4(a).

RB.13.3 *Beams*

RB.13.3.1 This provision is intended to result in flexural strength exceeding the cracking strength regardless of the analysis results. Provisions 9.6.1.3, 9.6.2.2, and Table 9.6.3.1 are not permitted for beams of main wind-force-resisting systems if nonlinear response history shows yielding because those provisions bypass requirements meant to produce deformation-controlled elements.

RB.13.3.2 Minimum shear reinforcement is required to produce deformation-controlled beams.

RB.13.3.3 Continuity of longitudinal reinforcement is required in beams of the main wind-force-resisting systems indicated by analysis to yield. This requirement does not exist elsewhere in the code for interior beams in non-seismic applications.

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have a total area not less than one-fourth the maximum area of bottom bars along the span. These bars shall develop f_{ye} in tension at the face of the support in accordance with 25.4 by substituting a bar stress of f_{ye} for f_y.

B.13.4 *Columns*

B.13.4.1 Columns indicated by analysis to yield shall be spirally reinforced in accordance of 25.7.3 or shall meet the requirements of B.13.4.1.1 through B.13.4.1.5.

B.13.4.1.1 At both ends of the column, hoop and crosstie reinforcement satisfying 25.7.2 and 25.7.4 shall be provided at spacing s_o over a length ℓ_o measured from the joint face.

B.13.4.1.2 Spacing s_o shall not exceed the least of (a) through (c):

(a) For Grade 60, the smaller of $8d_b$ of the smallest longitudinal bar enclosed and 8 in.
(b) For Grade 80, the smaller of $6d_b$ of the smallest longitudinal bar enclosed and 6 in.
(c) One-half of the smallest cross-sectional dimension of the column

B.13.4.1.3 Length ℓ_o shall not be taken less than the greatest of (a), (b), and (c):

(a) Depth of the column at the joint face or at the section where flexural yielding is likely to occur
(b) One-sixth of the clear span of the column
(c) 18 in.

B.13.4.1.4 The first hoop shall be located not more than $s_o/2$ from the joint face.

B.13.4.1.5 Outside of length ℓ_o, spacing of hoops or ties shall be in accordance with 10.7.6.5.2.

B.13.4.2 Columns supporting reactions from discontinuous stiff members, such as walls, shall be reinforced with spirals satisfying 25.7.3 or hoops and crossties satisfying 25.7.2, 25.7.4, and B.13.4.1.2 over the full height beneath the level at which the discontinuity occurs if the portion of factored axial compressive force in these members related to wind load effects exceeds $f_c'A_g/10$. If design forces have been magnified to account for the overstrength of the vertical elements of the main wind-force-resisting system, the limit of $f_c'A_g/10$ shall be increased to $f_c'A_g/4$. Transverse reinforcement shall extend above and below the column at least ℓ_d of the largest longitudinal column bar. Where the column terminates on a footing or mat, the required transverse reinforcement shall extend at least 12 in. into the footing or mat.

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RB.13.4 *Columns*

RB.13.4.1 Minimum detailing is required for columns indicated by analysis to yield to ensure their response is ductile.

RB.13.4.1.1 Columns indicated by analysis to yield are required to have hoops within a distance ℓ_o from joint faces. Outside the distance ℓ_o, either hoops or ties can be provided.

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B.13.5 *Beam-column joints*

B.13.5.1 Beam-column joints shall be in accordance with B.13.5.2 if analysis indicates yielding in one or more components framing into the joint.

B.13.5.2 Beam-column joints shall be in accordance with Chapter 15. In 15.4.2.1, V_u shall be calculated on a plane at mid-height of the joint using column shear and tensile and compressive beam forces consistent with expected moment strengths.

B.13.6 *Anchors*

B.13.6.1 Anchors of structural elements indicated by analysis to yield shall be in accordance with 17.10.5 and 17.10.6, with wind-induced forces substituted for earthquake-induced forces.

B.13.6.2 Anchor reinforcement used in main wind-force-resisting system shall be deformed reinforcement and shall be in accordance with the anchor reinforcement requirements of 20.2.2.

B.13.6.3 Post-installed anchors are not permitted to be used in portions of the main wind-force-resisting system indicated by analysis to yield.

B.13.7 *Development and lap splice lengths*

B.13.7.1 Within a distance equal to the member depth from the column or beam face for moment frames, and within a distance equal to the lesser of the story height and wall length from critical sections for structural walls where analysis indicates yielding of the reinforcement, (a) and (b) shall be satisfied.

(a) Development length and lap-splice length of longitudinal bars in tension shall be calculated using f_{ye}.
(b) Concrete cover measured to the lap-spliced longitudinal bars shall be at least $1.5d_b$ of the largest longitudinal bar unless confinement reinforcement is provided such that $K_{tr} \geq 1.35$. Clear spacing between lap-spliced longitudinal bars shall be at least $3d_b$ of the longitudinal bar.

B.13.8 *Foundations*

B.13.8.1 If analysis indicates yielding of column reinforcement and structural wall reinforcement due to forces induced by wind load effects, column and wall longitudinal reinforcement shall extend into the footing, mat, or pile cap and shall develop f_{ye} in tension at the interface.

B.13.9 *Members not designated as part of the main wind-force-resisting system*

B.13.9.1 Members not designated as part of the main wind-force-resisting system shall have sufficient deformation capacity to accommodate the deformations calculated to occur in the main wind-force-resisting system.

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RB.13.8 *Foundations*

RB.13.8.1 Reinforcement in foundations is required to provide a load path for tensile vertical forces associated with yielding of the superstructure.

RB.13.9 *Members not designated as part of the main wind-force-resisting system*

RB.13.9.1 The term member is used here in the same sense it is used in 18.14 and refers exclusively to structural members. Nevertheless, the engineer should give consideration to the effects of the expected deformations and motions on nonstructural building components and equipment.

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B.13.9.2 For slab-column connections of two-way slabs without beams, slab shear reinforcement satisfying the requirements of B.13.9.2.2 and either 8.7.6 or 8.7.7 shall be provided at any slab critical section defined in 22.6.4.1 for the following conditions:

(a) Nonprestressed slabs where $\delta_{x,w}/h_{sx} \geq 0.035 - (1/20)(v_{uv}/\phi v_c)$

(b) Unbonded post-tensioned slabs with f_{pc} in each direction meeting the requirements of 8.6.2.1, where $\delta_{x,w}/h_{sx} \geq 0.040 - (1/20)(v_{uv}/\phi v_c)$

The value of $(\delta_{x,w}/h_{sx})$ shall be taken as the greater of the values of the adjacent stories above and below the slab-column connection, and v_c shall be calculated in accordance with 22.6.5 except that the value of V_p shall be taken as zero for unbonded post-tensioned slabs.

B.13.9.2.1 The shear reinforcement requirements of B.13.9.2 need not be satisfied if (a) or (b) is met:

(a) $\delta_{x,w}/h_{sx} \leq 0.005$ for nonprestressed slabs

(b) $\delta_{x,w}/h_{sx} \leq 0.01$ for unbonded post-tensioned slabs with f_{pc} in each direction meeting the requirements of 8.6.2.1

B.13.9.2.2 Required slab shear reinforcement shall provide $v_s \geq 3.5\sqrt{f_c'}$ at the slab critical section and shall extend at least four times the slab thickness from the face of the support adjacent to the slab critical section.

B.14—Independent structural design review

B.14.1 The analysis and design shall be reviewed by an independent structural design reviewer. The independent structural design reviewer shall act under the direction of the building official.

B.14.2 The independent structural design review shall be performed by one or more individuals acceptable to the building official and shall possess, at a minimum, knowledge of (a) through (d):

(a) Selection of critical wind directions and their use in linear and nonlinear response history analysis.

(b) Behavior of structural systems of the type under consideration subjected to static and dynamic wind loading.

(c) Analytical structural modeling for use in nonlinear response history analysis, including the use of physical

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RB.13.9.2 Slab-column connections should be proportioned to reduce the likelihood of slab punching shear failure. v_{uv} should be obtained from load combination 5.3.1d, and drift $\delta_{x,w}$ should be taken as the maximum story drift expected for wind demands based on nonlinear response history analyses.

RB.14—Independent structural design review

RB.14.1 The independent structural design reviewer (also known as a peer reviewer) should provide an independent, objective, technical review of those aspects of the structural design of the building that relate to wind performance and advises the building official on whether the design meets the acceptance criteria and the expected building performance. A review by the independent structural design reviewer is not intended to replace quality assurance measures ordinarily exercised by the licensed design professional, nor is it intended to replace regular plan check review by the building official. Responsibility for the structural design remains solely with the licensed design professional in responsible charge of the structural design.

RB.14.2 On many projects, an independent structural design review may be provided by a review team approved by the building official. Each member of the review team should possess specialized knowledge and expertise and jointly meet the requirements of B.14.2. An independent structural design reviewer should not have conflicts of interest with respect to the project and should not be part of the design team for the project.

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tests in the creation and calibration of the structural analysis models and knowledge of parameters that affect wind loading, including damping.
(d) Requirements of Appendix B as pertaining to the design of the type of structure under consideration.

B.14.3 The scope of the independent structural design review shall be approved by the building official and shall include a minimum of (a) through (h):

(a) Basis of design document, including the performance objectives related to wind loading, the overall wind force-resistant design methodology, and acceptance criteria
(b) Proposed structural system
(c) Wind load determination by wind tunnel testing and selection of the critical wind directions
(d) Modeling approaches for components
(e) Structural analysis model, including soil-structure interaction as applicable, and verification that the structural analysis model adequately represents the properties of the structural system
(f) Structural analysis results and determination of whether calculated response meets approved acceptance criteria
(g) Design and detailing of structural components
(h) Drawings, specifications, and quality control/quality assurance and inspection provisions in the construction documents

B.14.4 The independent structural design review shall be documented and submitted to the building official in accordance with jurisdictional requirements.

B.14.4.1 Absent any formal requirements by the building official, local jurisdiction, or both, the independent structural design review shall be documented as follows:

(a) The independent structural design reviewer shall issue comments and questions to the licensed design professional.
(b) The licensed design professional shall provide written responses to the independent structural design reviewer.
(c) The independent structural design reviewer shall summarize the review in a letter addressed to the building official that shall include a log of all questions or comments and responses. Any items that lack resolution or consensus shall be clearly explained with reasons for lack of agreement.

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RB.14.3 The scope of the independent structural design review should be clearly defined and acceptable to the building official.

RB.14.4 A statement of agreement with the design should be provided. However, there may be occasions where complete agreement between the independent structural design reviewer and the licensed design professional cannot be reached. These items should be documented in the summary review letter.

Notes