CHAPTER 16
NONLINEAR RESPONSE HISTORY ANALYSIS

16.1 GENERAL REQUIREMENTS

16.1.1 Scope It shall be permitted to use nonlinear response history analysis, in accordance with the requirements of this chapter, to demonstrate acceptable strength, stiffness, and ductility to resist maximum considered earthquake (MCE_R) shaking with acceptable performance. When nonlinear response history analysis is performed, the design shall also satisfy the requirements of Section 16.1.2. Nonlinear response history analysis shall include the effects of horizontal motion, and where required by Section 16.1.3, vertical motion. Documentation of the design and analysis shall be prepared in accordance with Section 16.1.4. Ground motion acceleration histories shall be selected and modified in accordance with the procedures of Section 16.2. The structure shall be modeled and analyzed in accordance with the criteria in Section 16.3. Analysis results shall meet the acceptance criteria of Section 16.4. Independent structural design review shall be performed in accordance with the requirements of Section 16.5.

16.1.2 Linear Analysis In addition to nonlinear response history analysis, a linear analysis in accordance with one of the applicable procedures of Chapter 12 shall also be performed. The structure's design shall meet all applicable criteria of Chapter 12.

EXCEPTIONS:

1. For Risk Category I, II, and III structures, Sections 12.12.1 and 12.12.5 do not apply to the linear analysis. Where mean computed drifts from the nonlinear analyses exceed 150% of the permissible story drifts per Section 12.12.1, deformation-sensitive nonstructural components shall be designed for 2/3 of these mean drifts.
2. The overstrength factor, Ω_0, is permitted to be taken as 1.0 for the seismic load effects of Section 12.4.3.
3. The redundancy factor, ρ, is permitted to be taken as 1.0.
4. Where accidental torsion is explicitly modeled in the nonlinear analysis, it shall be permitted to take the value of A_x as unity in the Chapter 12 analysis.

16.1.3 Vertical Response Analysis Nonlinear response history analysis shall explicitly include the effects of vertical response where any of the following occur:

1. Vertical elements of the gravity force-resisting system are discontinuous.
2. For nonbuilding structures, when Chapter 15 requires consideration of vertical earthquake effects.

16.1.4 Documentation Before performing the nonlinear analysis, project-specific design criteria shall be approved by the independent structural design reviewer(s) and the Authority Having Jurisdiction. The project-specific criteria shall identify the following:

1. The selected seismic and gravity force-resisting systems and procedures used in the structural design.
2. Geotechnical parameters including soil characteristics, recommended foundation types, design parameters, seismic hazard evaluation, target spectra, and selection and scaling of acceleration histories.
3. Design loading, including gravity and environmental loads.
4. Analytical modeling approach and assumptions, including software to be used, definition of mass, identification of force-controlled and deformation-controlled behaviors, description of which component actions are modeled elastically and inelastically, expected material properties, basis for hysteretic component modeling, component initial stiffness assumptions, joint stiffness assumptions, diaphragm modeling, damping, and procedure for modeling foundation–soil interaction.
5. Summaries of laboratory test and other applicable data used to justify the hysteretic component modeling or used to justify acceptable structural performance.
6. Specific acceptance criteria values used for evaluating performance of elements of the seismic force-resisting system. Associated documentation shall also include identification of component failure modes deemed indicative of collapse.
7. Where drifts exceed 150% of the values permitted in Section 12.12, the criteria used to demonstrate acceptable deformation compatibility of components of the gravity force-resisting system.

Following completion of the analysis process, the following documentation shall be prepared and presented to the independent structural design reviewer(s) and the Authority Having Jurisdiction:

1. Final geotechnical report, including soil shear strength, stiffness, and damping characteristics; recommended foundation types and design parameters; and seismic hazard evaluation, including both the target spectra and selection and scaling of ground motions.
2. Overall building dynamic behavior, including natural frequencies, mode shapes, and modal mass participation.
3. Key structural system response parameter results and comparisons with the acceptance criteria of Section 16.4.
4. Detailing of critical elements.

16.2 GROUND MOTIONS

16.2.1 Target Response Spectrum A target, 5%-damped, MCE_R response spectrum shall be developed using either the procedures of Section 16.2.1.1 or Section 16.2.1.2. It shall be permitted to consider the effects of base slab averaging and foundation embedment in accordance with Chapter 19.

Where the effects of vertical earthquake shaking are included in the analysis, a target MCE_R vertical spectrum shall also be constructed.

16.2.1.1 Method 1 A single target response spectrum shall be developed, based on the requirements of either Section 11.4.6 or Section 11.4.7.

16.2.1.2 Method 2 Two or more site-specific target response spectra shall be developed. When this method is used, the following requirements shall be fulfilled, in addition to the other requirements of this chapter:

1. Two or more periods shall be selected, corresponding to those periods of vibration that significantly contribute to the inelastic dynamic response of the building in two orthogonal directions. In the selection of periods, lengthening of the elastic periods of the model shall be considered.
2. For each selected period, a target spectrum shall be created that either matches or exceeds the MCE_R value at that period. When developing the target spectrum, (1) site-specific disaggregation shall be performed to identify earthquake events that contribute most to the MCE_R ground motion at the selected period and (2) the target spectrum shall be developed to capture one or more spectral shapes for dominant magnitude and distance combinations revealed by the disaggregation.
3. The envelope of the target spectra shall not be less than 75% of the spectral values computed using Method 1 of Section 16.2.1.1, for all periods in the range specified in Section 16.2.3.1.
4. For each target response spectrum, a ground motion suite for response history analyses shall be developed and used in accordance with Sections 16.2.3 through 16.2.4. The acceptance criteria requirements of Section 16.4 shall be evaluated independently for each of the ground motion suites.

Variations on the procedures described in this section are permitted to be used when approved by the design review.

16.2.2 Ground Motion Selection A suite of not less than 11 ground motions shall be selected for each target spectrum. Ground motions shall consist of pairs of orthogonal horizontal ground motion components and, where vertical earthquake effects are considered, a single vertical ground motion component. Ground motions shall be selected from events within the same general tectonic regime and having generally consistent magnitudes and fault distances as those controlling the target spectrum and shall have a spectral shape similar to the target spectrum. For near-fault sites, as defined in Section 11.4.1, and other sites where MCE_R shaking can exhibit directionality and impulsive characteristics, the proportion of ground motions with near-fault and rupture directivity effects shall represent the probability that MCE_R shaking will exhibit these effects. Where the required number of recorded ground motions is not available, it shall be permitted to supplement the available records with simulated ground motions. Ground motion simulations shall be consistent with the magnitudes, source characteristics, fault distances, and site conditions controlling the target spectrum.

16.2.3 Ground Motion Modification Ground motions shall either be amplitude-scaled in accordance with the requirements of Section 16.2.3.2 or spectrally matched in accordance with the requirements of Section 16.2.3.3. Spectral matching shall not be used for near-fault sites unless the pulse characteristics of the ground motions are retained after the matching process has been completed.

16.2.3.1 Period Range for Scaling or Matching A period range shall be determined, corresponding to the vibration periods that contribute significantly to the building's lateral dynamic response. This period range shall have an upper bound, greater than or equal to twice the largest first-mode period in the principal horizontal directions of response, unless a lower value, not less than 1.5 times the largest first-mode period, is justified by dynamic analysis under MCE_R ground motions. The lower bound period shall be established such that the period range includes at least the number of elastic modes necessary to achieve 90% mass participation in each principal horizontal direction. The lower bound period shall not exceed 20% of the smallest first-mode period for the two principal horizontal directions of response. Where vertical response is considered in the analysis, the lower bound period used for modification of vertical components of ground motion need not be taken as less than the larger of 0.1 seconds, or the lowest period at which significant vertical mass participation occurs.

16.2.3.2 Amplitude Scaling For each horizontal ground motion pair, a maximum-direction spectrum shall be constructed from the two horizontal ground motion components. Each ground motion shall be scaled, with an identical scale factor applied to both horizontal components, such that the average of the maximum-direction spectra from all ground motions generally matches or exceeds the target response spectrum over the period range defined in Section 16.2.3.1. The average of the maximum-direction spectra from all the ground motions shall not fall below 90% of the target response spectrum for any period within the same period range. Where vertical response is considered in the analysis, the vertical component of each ground motion shall be scaled such that the average of the vertical response spectra envelops the target vertical response spectrum over the period range specified in Section 16.2.3.1.

16.2.3.3 Spectral Matching Each pair of ground motions shall be modified such that the average of the maximum-direction spectra for the suite equals or exceeds 110% of the target spectrum over the period range defined in Section 16.2.3.1. Where vertical response is considered in the analysis, the vertical component of each ground motion shall be spectrally matched to the target vertical response spectrum, such that the average of the matched spectra does not fall below the target vertical spectrum in the scaling range of Section 16.2.3.1.

16.2.4 Application of Ground Motions to the Structural Model Ground motions shall be applied to the supports of the structural model. For near-fault sites, as defined in Section 11.4.1, each pair of horizontal ground motion components representative of a nearby fault source shall be rotated to the fault-normal and fault-parallel directions of the causative fault and applied to the building in such orientation. For all other selected ground motions at near-fault sites, and for all ground motions at other sites, each pair of horizontal ground motion components shall be applied to the building at orthogonal orientations, such that the average (or mean) of the component response spectrum for the records applied in each direction is within ±10% of the mean of the component response spectra of all records applied for the period range specified in Section 16.2.3.1.

16.3 MODELING AND ANALYSIS

16.3.1 Modeling Mathematical models shall be three-dimensional and shall conform to the requirements of this section and Section 12.7. For structures that have subterranean levels, the structural model shall extend to the foundation level and ground motions shall be input at the foundation level. All elements that significantly affect seismic response when subjected to MCE_R ground motions shall be included. Modeling of element nonlinear hysteretic behavior shall be consistent with ASCE 41 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.

Analysis models shall be capable of representing the flexibility of floor diaphragms where this is significant to the structure's response. Diaphragms at horizontal and vertical discontinuities in lateral resistance shall be explicitly modeled in a manner that permits capturing the force transfers and resulting deformations.

16.3.2 Gravity Load The modeling of, and demands on, elements in the analysis model shall be determined considering earthquake effects acting in combination with expected gravity loads, both with and without live load. Gravity loads with live load shall be taken as $1.0D + 0.5L$, where L shall be taken as 80% of unreduced live loads that exceed $100\,\mathrm{lb/ft^2}$ ($4.79\,\mathrm{kN/m^2}$) and 40% of all other unreduced live loads. Gravity loads without live load shall be taken as $1.0D$.

EXCEPTION: Where the sum, over the entire structure, of the expected live load ($0.5L$), as defined above, does not exceed 25% of the total dead load, D, and the live load intensity, L_0, over at least 75% of the structure is less than 100 psf ($4.79\,\mathrm{kN/m^2}$), the case without live load need not be considered.

16.3.3 P-Delta Effects P-delta effects considering the spatial distribution of gravity loads shall be included in the analysis.

16.3.4 Torsion Inherent eccentricity resulting from any offset in the centers of mass and stiffness at each level shall be accounted for in the analysis. In addition, where a Type 1 horizontal structural irregularly exists, as defined in Section 12.3.2.1, accidental eccentricity consisting of an assumed displacement of the center of mass each way from its actual location by a distance equal to 5% of the diaphragm dimension of the structure parallel to the direction of mass shift shall be considered. The required 5% displacement of the center of mass need not be applied in both orthogonal directions at the same time.

16.3.5 Damping Hysteretic energy dissipation of structural members shall be modeled directly. Additional inherent damping, not associated with inelastic behavior of elements, shall be modeled appropriate to the structure type and shall not exceed 2.5% equivalent viscous damping in the significant modes of response.

16.3.6 Explicit Foundation Modeling When soil spring and/or dashpot elements are included in the structural model, horizontal input ground motions shall be applied to the horizontal soil elements rather than being applied to the foundation directly.

16.4 ANALYSIS RESULTS AND ACCEPTANCE CRITERIA

Structures shall be demonstrated to meet the global acceptance criteria of Section 16.4.1 and the element-level acceptance criteria of Section 16.4.2.

The mean value of story drift, and element demand, Q_u, shall be used to evaluate acceptability.

EXCEPTION: Where a ground motion produces unacceptable response as permitted in Section 16.4.1.1, 120% of the median value, but not less than the mean value obtained from the suite of analyses producing acceptable response shall be used.

16.4.1 Global Acceptance Criteria

16.4.1.1 Unacceptable Response Unacceptable response to ground motion shall consist of any of the following:

1. Analytical solution fails to converge,
2. Predicted demands on deformation-controlled elements exceed the valid range of modeling,
3. Predicted demands on critical or ordinary force-controlled elements, as defined in Section 16.4.2, exceed the element capacity,
4. Predicted deformation demands on elements not explicitly modeled exceed the deformation limits at which the members are no longer able to carry their gravity loads,
5. Peak transient story drift ratio exceeds 150% of the permissible value of mean transient story drift, as per Section 16.4.1.2, or
6. For structures exceeding 240 ft (73m) in height, the residual story drift for any story exceeds a value of $0.015\,h_{sx}$.

Unacceptable response to ground motion shall not be permitted.

EXCEPTION: For Risk Category I and II structures, where spectral matching of ground motion is not used, not more than one motion shall be permitted to produce unacceptable response.

16.4.1.2 Transient Story Drift The mean transient story drift for all building heights, $\overline{\Delta}$ in ft (m), shall not exceed two times the limits of Table 12.12-1. Additionally, for structures exceeding 100 ft (30 m) in height, $\overline{\Delta}$ shall not exceed the value obtained from Equation (16.4-1):

$$\overline{\Delta} \le h_{sx}(4.71x10^{-2} - 7.14x10^{-5}h_n) \tag{16.4-1}$$

$$\overline{\Delta} \le h_{sx}(4.71x10^{-2} - 2.34x10^{-4}h_n) \tag{16.4-1.SI}$$

where h_n and h_{sx} are measured in ft (m). The value obtained from Equation (16.4-1) need not be taken less than $0.03\,h_n$.

The transient story drift ratio shall be computed as the absolute value of the largest difference of the deflections of vertically aligned points at the top and bottom of the story under consideration along any of the edges of the structure within a single response history analysis. For masonry shear wall structures, the limits of Table 12.12-1 applicable to masonry cantilever wall structures and other masonry wall structures shall not apply and these structures shall instead comply with the limits for other structures.

16.4.1.3 Residual Story Drift For structures exceeding 240 ft (73 m) in height, the mean residual story drift shall not exceed $0.01\,h_{sx}$, where residual story drift is taken as the maximum value of story drift in a structure at rest, following response to an earthquake motion.

16.4.2 Element-Level Acceptance Criteria All element actions shall be classified either as force-controlled or deformation-controlled, in accordance with ACI 318 for reinforced concrete elements or ASCE 41 for elements of other materials.

For each element action, the quantity, Q_u, shall be computed. Q_u shall be taken as the mean value of the response parameter of interest obtained from the suite of analyses.

Force-controlled actions shall be evaluated for acceptability in accordance with Section 16.4.2.1. Deformation-controlled actions shall be evaluated for acceptability in accordance with Section 16.4.2.2. Where required by Section 16.4.2.1, element actions shall be categorized as Critical, Ordinary, or Noncritical.

16.4.2.1 Force-Controlled Actions Force-controlled actions shall satisfy Equation (16.4-1) and (16.4-2):

$$(1.2 + 0.12S_{MS})D + 0.5L + 1.3I_e(Q_u - Q_{ns}) \leq \phi BR_n \quad (16.4\text{-}1)$$

$$(0.9 - 0.12S_{MS})D + 1.3I_e(Q_u - Q_{ns}) \leq \phi BR_n \quad (16.4\text{-}2)$$

where D and L are as defined in Section 16.3.2, S_{MS} is the site-adjusted Maximum Considered Earthquake Spectral Acceleration at a period of 0.2 seconds; I_e is the Importance Factor prescribed in Section 1.5.1; Q_u is the mean value of the demand computed from the suite of analyses; Q_{ns} is the portion of the demand caused by loads other than seismic; R_n is the nominal strength specified by the applicable material standard. The resistance factor ϕ for Critical elements shall be taken as the value specified by the applicable material standard. The resistance factor ϕ for Ordinary elements shall be taken as 0.9. The resistance factor ϕ for Noncritical elements shall be taken as 1.0. B is a factor to account for differences between expected strength R_{ne} and nominal resistance R_n. It is permitted to assign B a value of 1.0, or, alternatively, B can be taken as $0.9R_{ne}/R_n$, where R_{ne} is the expected strength of the element.

Where an industry standard referenced in Chapter 14 defines expected strength, that value shall be used. Where this is not defined, it shall be permitted to calculate expected strength as the nominal strength defined in industry standards, except that expected material properties as defined in ACI 318 for reinforced concrete elements and ASCE 41 for elements of other materials shall be used in lieu of specified values.

EXCEPTIONS:

1. Noncritical force-controlled actions that are modeled, including consideration of strength loss effects, need not satisfy Equations (16.4-1) or (16.4-2).
2. Force-controlled actions limited by formation of a yield mechanism, other than shear in structural walls, need only satisfy Equations (16.4-3) and (16.4-4):

$$(1.2 + 0.12S_{MS})D + 0.5L + 0.2S + E_{mc} \leq \phi BR_n \quad (16.4\text{-}3)$$

$$(0.9 - 0.12S_{MS})D + E_{mc} \leq \phi BR_n \quad (16.4\text{-}4)$$

   Where E_{mc} is the capacity-limited earthquake effect associated with developing the plastic capacity of yielding components, determined in accordance with the applicable material standard, or alternatively, determined by rational analysis considering expected material properties including strain hardening effects where applicable.
3. Where response to vertical earthquake shaking is directly included in the analysis, the first term in equations 16.4-1 through 16.4-4 can be taken as 1.2D (16.4-1 and 16.4-3) or 0.9D (16.4-2 and 16.4-4).

16.4.2.2 Deformation-Controlled Actions The valid range of modeling for deformation-controlled element actions shall be as established in the applicable material design standard. Where the material design standard does not specify the valid range of modeling, this parameter shall be established as the maximum value of the parameter at which the element model is capable of replicating the hysteretic behavior and load-carrying capability observed in laboratory testing of similar elements. Where suitable test data is not available, either the valid range of modeling of the deformation capacity specified in ACI 318 for reinforced concrete elements or the maximum deformation parameter as specified by ASCE 41 for elements of other materials shall be used. It shall be permitted to extend the valid range of modeling for an element beyond these deformations if the element strength and stiffness are degraded to negligible values once these deformations are reached.

16.4.2.3 Elements of the Gravity Force-Resisting System Elements that are not part of the seismic force-resisting system shall be demonstrated to be capable of supporting gravity loads using the mean building displacements from the suite of nonlinear response history analyses.

16.5 DESIGN REVIEW

An independent structural design review shall be performed in accordance with the requirements of this section. Upon completion of the review, the reviewer(s) shall provide the Authority Having Jurisdiction and the registered design professional with a letter attesting to:

1. Scope of review performed,
2. Whether the reviewer(s) concur with the analysis and its applicability to the design,
3. Conformance of the design to applicable requirements of the standard, and
4. Any items relating to the design or analysis that require further resolution by the Authority Having Jurisdiction.

16.5.1 Reviewer Qualifications Reviewer(s) shall consist of one or more individuals acceptable to the Authority Having Jurisdiction and possessing knowledge of the following items:

1. The requirements of this standard and the standards referenced herein, as they pertain to design of the type of structure under consideration.
2. Selection and scaling of ground motions for use in nonlinear response history analysis.
3. Analytical structural modeling for use in nonlinear response history analysis, including use of laboratory tests in the creation and calibration of the structural analysis models, and including knowledge of soil–structure interaction if used in the analysis or the treatment of ground motions.
4. Behavior of structural systems, of the type under consideration, when subjected to earthquake loading.

At least one reviewer shall be a registered design professional.

16.5.2 Review Scope The scope of review shall include the items identified in Section 16.1.4, as well as the associated project documentation that demonstrates conformance to the design criteria.

16.6 CONSENSUS STANDARDS AND OTHER REFERENCED DOCUMENTS

See Chapter 23 for the list of consensus standards and other documents that shall be considered part of this standard to the extent referenced in this chapter.

CHAPTER C16
NONLINEAR RESPONSE HISTORY ANALYSIS

C16.1 GENERAL REQUIREMENTS

C16.1.1 Scope Response history analysis is a form of dynamic analysis in which response of the structure to a suite of ground motions is evaluated through numerical integration of the equations of motions. In nonlinear response history analysis, the structure's stiffness matrix is modified throughout the analysis to account for the changes in element stiffness associated with hysteretic behavior and P-delta effects. When nonlinear response history analysis is performed, the R, C_d, and Ω_0 coefficients considered in linear procedures are not applied because the nonlinear analysis directly accounts for the effects represented by these coefficients.

Nonlinear response history analysis is permitted to be performed as part of the design of any structure and is specifically required to be performed for the design of certain structures incorporating seismic isolation or energy dissipation systems. Nonlinear response history analysis is also frequently used for the design of structures that use alternative structural systems or do not fully comply with the prescriptive requirements of the standard in one or more ways. Before this edition, ASCE 7 specified that nonlinear response history analyses had to be performed using ground motions scaled to the design earthquake level and that design acceptance checks be performed to ensure that mean element actions do not exceed two-thirds of the deformations at which loss of gravity-load-carrying capacity would occur. In this edition of ASCE 7, a complete reformulation of these requirements was undertaken to require analysis at the Risk-Targeted Maximum Considered Earthquake (MCE_R) level and also to be more consistent with the target reliabilities indicated in Section 1.3.1.3.

The target collapse reliabilities given in Table 1.3-2 are defined such that when a building is subjected to MCE_R ground motion, not greater than a 10% probability of collapse exists for Risk Category I and II structures. For Risk Category III and IV structures, these maximum collapse probabilities are reduced to 5% and 2.5%, respectively.

There are additional performance expectations for Risk Category III and IV structures that go beyond the collapse safety performance goals (e.g., limited damage and postearthquake functionality for lower ground motion levels). These enhanced performance goals are addressed in this chapter by enforcing an $I_e > 1.0$ in the linear design step (which is consistent with the approach taken in the other design methods of Chapter 12) and also by considering I_e in acceptance checks specified in Section 16.4.

It is conceptually desirable to create a Chapter 16 response history analysis (RHA) design process that explicitly evaluates the collapse probability and ensures that the performance goal is fulfilled. However, explicit evaluation of collapse safety is a difficult task requiring (a) a structural model that is able to directly simulate the collapse behavior, (b) use of numerous nonlinear response history analyses, and (c) proper treatment of many types of uncertainties. This process is excessively complex and lengthy for practical use in design. Therefore, Chapter 16 maintains the simpler approach of *implicitly* demonstrating adequate performance through a prescribed set of analysis rules and acceptance criteria. Even so, this implicit approach does not preclude the use of more advanced procedures that explicitly demonstrate that a design fulfills the collapse safety goals. Such more advanced procedures are permitted by Section 1.3.1.3 of this standard. An example of an advanced explicit procedure is the building-specific collapse assessment methodology in Appendix F of FEMA P-695 (FEMA 2009b).

C16.1.2 Linear Analysis As a precondition to performing nonlinear response history analysis, a linear analysis, in accordance with the requirements of Chapter 12, is required. Any of the linear procedures allowed in Chapter 12 may be used. The purpose of this requirement is to ensure that structures designed using nonlinear response history analyses meet the minimum strength and other criteria of Chapter 12, with a few exceptions. In particular, when performing the Chapter 12 evaluations, it is permitted to take the value of Ω_0 as 1.0 because it is felt that values of demand obtained from the nonlinear procedure are a more accurate representation of the maximum forces that will be delivered to critical elements, considering structural overstrength, than does the application of the judgmentally derived factors specified in Chapter 12. Similarly, it is permitted to use a value of 1.0 for the redundancy factor, ρ, because it is felt that the inherent nonlinear evaluation of response to MCE_R shaking required by this chapter provides improved reliability relative to the linear procedures of Chapter 12. For Risk Category I, II, or III structures, it is permitted to neglect the evaluation of story drift when using the linear procedure because it is felt that the drift evaluation performed using the nonlinear procedure provides a more accurate assessment of the structure's tolerance to earthquake-induced drift. However, linear drift evaluation is required for Risk Category IV structures because it is felt that this level of drift control is important to attaining the enhanced performance desired for such structures.

As with other simplifications permitted in the linear analysis required under this section, it is also permitted to use a value of 1.0 for the torsional amplification, A_x, when performing a nonlinear analysis if accidental torsion is explicitly modeled in the nonlinear analysis. Although this does simplify the linear analysis somewhat, designers should be aware that the resulting structure may be more susceptible to torsional instability when performing the nonlinear analysis. Therefore, some designers may find it expedient to use a value of A_x consistent with the linear procedures as a means of providing a higher likelihood that the nonlinear analysis will result in acceptable outcomes.

C16.1.3 Vertical Response Analysis Most structures are not sensitive to the effects of response to vertical ground shaking, and there is little evidence of the failure of structures in earthquakes resulting from vertical response. However, some nonbuilding structures and building structures with long spans, cantilevers, prestressed construction, or vertical discontinuities in their gravity-load-resisting systems can experience significant vertical earthquake response that can cause failures. The linear procedures of Chapter 12 account for these effects in an approximate manner, through use of the $0.2S_{DS}D$ term in the load combinations. When nonlinear response history analysis is performed for structures with sensitivity to vertical response, direct simulation of this response is more appropriate than the use of the approximate linear procedures. However, in order to properly capture vertical response to earthquake shaking, it is necessary to accurately model the stiffness and distribution of mass in the vertical load system, including the flexibility of columns and horizontal framing. This effort can considerably increase the complexity of analytical models. Rather than requiring this extra effort in all cases where vertical response can be significant, this chapter continues to rely on the approximate approach embedded in Chapter 12 for most cases. However, where the vertical load path is discontinuous and where vertical response analysis is required by Chapter 15, Chapter 16 does require explicit modeling and analysis of vertical response. Since in many cases the elements sensitive to vertical earthquake response are not part of the seismic force-resisting system, it is often possible to decouple the vertical and lateral response analyses, using separate models for each.

Appropriate accounting for the effects of vertical response to ground shaking requires that horizontal framing systems, including floor and roof systems, be modeled with distributed masses and sufficient vertical degrees of freedom to capture their out-of-plane dynamic characteristics. This increased fidelity in modeling of the structure's vertical response characteristics will significantly increase the size and complexity of models. As a result, the chapter requires direct simulation of vertical response only for certain structures sensitive to those effects and relies on the procedures of Chapter 12 to safeguard the vertical response of other structures.

C16.1.4 Documentation By its nature, most calculations performed using nonlinear response history analysis are contained within the input and output of computer software used to perform the analysis. This section requires documentation, beyond the computer input and output, of the basic assumptions, approaches, and conclusions so that thoughtful review may be performed by others including peer reviewers and the Authority Having Jurisdiction. This section requires submittal and review of some of these data before the analyses are performed, in order to ensure that the engineer performing the analysis/design and the reviewers are in agreement before substantive work is performed.

C16.2 GROUND MOTIONS

C16.2.1 Target Response Spectrum The target response spectrum used for nonlinear dynamic analysis is the maximum direction MCE_R spectrum determined in accordance with Chapter 11 or Chapter 21. Typical spectra, determined in accordance with those procedures, are derived from uniform hazard spectra (UHSs) and modified to provide a uniform risk spectrum (URS), or alternatively, a deterministic MCE spectrum. Since the 1980s, UHSs have been used as the target spectra in design practice. The UHS is created for a given hazard level by enveloping the results of seismic hazard analysis for each period (for a given probability of exceedance). Accordingly, it is generally a conservative target spectrum if used for ground motion selection and scaling, especially for large and rare ground motions, unless the structure exhibits only elastic first-mode response. This inherent conservatism comes from the fact that the spectral values at each period are not likely to all occur in a single ground motion. This limitation of the UHS has been noted for many years (e.g., Bommer et al. 2000, Naeim and Lew 1995, Reiter 1990). The same conservatism exists for the URS and deterministic MCE spectra that serve as the basis for Method 1.

Method 2 uses the conditional mean spectrum (CMS), an alternative to the URS that can be used as a target for ground motion selection in nonlinear response history analysis (e.g., Baker and Cornell 2006, Baker 2011, Al Atik and Abrahamson 2010).

To address the conservatism inherent in analyses using URSs as a target for ground motion selection and scaling, the CMS instead conditions the spectrum calculation on a spectral acceleration at a single period and then computes the mean (or distribution of) spectral acceleration values at other periods. This conditional calculation ensures that the resulting spectrum is reasonably likely to occur and that ground motions selected to match the spectrum have an appropriate spectral shape consistent with naturally occurring ground motions at the site of interest. The calculation is no more difficult than the calculation of a URS and is arguably more appropriate for use as a ground motion selection target in risk assessment applications. The spectrum calculation requires disaggregation information, making it a site-specific calculation that cannot be generalized to other sites. It is also period-specific, in that the conditional response spectrum is based on a spectral acceleration value at a specified period. The shape of the conditional spectrum also changes as the spectral amplitude changes (even when the site and period are fixed). Figure C16.2-1 provides examples of CMSs for an example site in Palo Alto, California, anchored at four different candidate periods. The UHS for this example site is also provided for comparison.

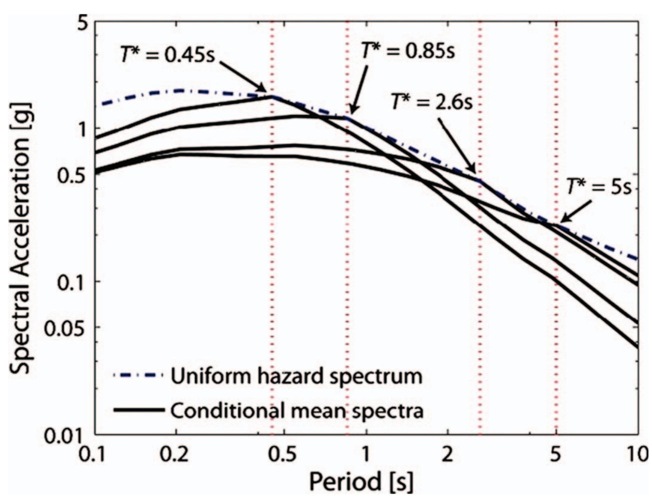

Figure C16.2-1. Example conditional mean spectra for a Palo Alto site, anchored for 2% in 50-year motion at $T = 0.45$s, 0.85s, 2.6s, and 5s.
Source: NIST (2011).

As previously discussed, the URS is a conservative target spectrum for ground motion selection, and the use of CMS target spectra is more appropriate for representing anticipated MCE_R ground motions at a specified period. A basic CMS-type approach was used in the analytical procedures of the FEMA P-695 (FEMA 2009b) project, the results of which provided the initial basis for establishing the 10% probability of collapse goal shown in Table 1.3-2. Therefore, the use of CMS target spectra in the Chapter 16 RHA design procedure is also internally consistent with how the collapse probability goals of Table 1.3-2 were developed.

The URS (or deterministic MCE) target spectrum is retained in Section 16.2.1.1 (as a simpler and more conservative option) as the specified target spectrum, and the CMS is permitted as an alternate in Section 16.2.1.2. Whereas CMS appropriately captures the earthquake energy and structural response at a particular period resulting from a particular scenario earthquake, it is not capable of capturing the MCE_R level response associated with other scenarios that are relevant to the MCE_R spectrum. Therefore, when using CMS, it may be necessary to use several conditioning periods and associated targets to develop conditional mean spectra, in order to fully capture the structure's response to different earthquake scenarios. The recommended procedure includes the following steps for creating the site-specific scenario response spectra.

1. Select those periods that correspond to periods of vibration that significantly contribute to the building's inelastic dynamic response. This selection includes a period near the fundamental period of the building, or perhaps a slightly longer period to account for inelastic period lengthening (e.g., $1.5T_1$). In buildings where the fundamental response periods in each of two orthogonal axes is significantly different, a conditioning period associated with each direction is needed. It also likely requires periods near the translational second-mode periods. When selecting these significant periods of response, the elastic periods of response should be considered (according to the level of mass participation for each of these periods), and the amount of first-mode period increase caused by inelastic response effects should also be considered.
2. For each period selected above, create a scenario spectrum that matches or exceeds the MCE_R value at that period. When developing the scenario spectrum, (a) perform site-specific disaggregation to identify earthquake events likely to result in MCE_R ground shaking, and then (b) develop the scenario spectrum to capture one or more spectral shapes for dominant magnitude and distance combinations revealed by the disaggregation.
3. Enforce that the envelope of the scenario spectra not be less than 75% of the MCE_R spectrum (from Method I) for any period within the period range of interest (as defined in Section 16.2.3.1).

After the target spectra are created, each target response spectrum is then used in the remainder of the response history analyses process and the building must be shown to meet the acceptance criteria for each of the scenarios.

The primary purpose of the 75% limit value is to provide a basis for determining how many target spectra are needed for analysis. For small period ranges, fewer targets are needed, and more target spectra are needed for buildings where a wider range of periods are important to the structural response (e.g., taller buildings). When creating the target spectra, some spectral values can also be artificially increased to meet the requirements of this 75% limit. A secondary reason for the 75% limit is to enforce a reasonable lower bound. The specific 75% threshold value was determined using several examples. The intention is that this 75% floor requirement will be fulfilled through the use of two target spectra, in most cases. From the perspective of collapse risk, the requirement of being within 75% of the MCE_R at all periods may introduce some conservatism, but the requirement adds robustness to the procedure by ensuring that the structure is subjected to ground motions with near-MCE_R-level intensities at all potentially relevant periods. In addition, this requirement ensures that demands unrelated to collapse safety, such as higher mode-sensitive force demands, can be reasonably determined from the procedure.

C16.2.2 Ground Motion Selection Before ASCE 7-16, Chapter 16 required a minimum of three ground motions for nonlinear response history analysis. If three ground motions were used, the procedures required evaluation of structural adequacy using the maximum results obtained from any of the ground motions. If seven or more motions were used, mean results could be used for evaluation. Neither three nor seven motions are sufficient to accurately characterize either mean response or the record-to-record variability in response. In the 2016 edition of the standard, the minimum number of motions was increased to 11. The requirement for this larger number of motions was not based on detailed statistical analyses, but rather was judgmentally selected to balance the competing objectives of more reliable estimates of mean structural responses (through use of more motions) against computational effort (reduced by using fewer motions). An advantage of using this larger number of motions is that if unacceptable response is found for more than one of the 11 motions, this indicates a significant probability that the structure will fail to meet the 10% target collapse reliability for Risk Categories I and II structures of Section 1.3.1.3. This advantage is considered in the development of acceptance criteria discussed in Section C16.4.

All real ground motions include three orthogonal components. For most structures, it is only necessary to consider response to horizontal components of ground shaking. However, consideration of vertical components is necessary for structures defined as sensitive to vertical earthquake effects.

Section 11.4.1 defines near-fault sites as sites located within 9.3 mi (15 km) of the surface projection of faults capable of producing earthquakes of magnitude 7.0 or greater and within 6.2 mi (10 km) of the surface projection of faults capable of producing earthquakes of magnitude 6.0 or greater, where the faults must meet minimum annual slip rate criteria. Such near-fault sites have a reasonable probability of experiencing ground motions strongly influenced by rupture directivity effects. These effects can include pulse-type ground motions (e.g., Shahi et al. 2011) observable in velocity histories and polarization of ground motions, such that the maximum direction of response tends to be in the direction normal to the fault strike. The issue of pulse-type ground motions affects the manner by which individual ground motions should be selected for the site and applied to the structure.

Selection of Ground Motions for Sites That Are Not Near-Fault. The traditional approach has been to select (and/or simulate) ground motions that have magnitudes, fault distances, source mechanisms, and site soil conditions that are roughly similar to those likely to cause the ground motion intensity level of interest (e.g., Stewart et al. 2002) and not to consider the spectral shape in the ground motion selection. In many cases, the response spectrum is the property of a ground motion most correlated with the structural response (Bozorgnia et al. 2009) and should be considered when selecting ground motions.

When spectral shape is considered in the ground motion selection, the allowable range of magnitudes, distances, and site conditions can be relaxed so that a sufficient number of ground motions with appropriate spectral shapes are available.

The selection of recorded motions typically occurs in two steps, as explained in the following illustration. Step 1 involves preselecting the ground motion records in the database (e.g., Anchenta et al. 2015) that have reasonable source mechanisms, magnitude, site soil conditions, a range of usable frequencies, and site-to-source distance. In completing this preselection, it is permissible to use relatively liberal ranges because Step 2 can involve selecting motions that provide good matches to a target spectrum of interest (and matching to a target spectrum tends to implicitly account for many of the above issues). Step 2 in the selection process is to select the final set of motions from those preselected in Step 1.

In the first step, the following criteria should be used to filter out ground motions that should not be considered as candidates for the final selection process:

- **Source Mechanism:** Ground motions from differing tectonic regimes (e.g., subduction versus active crustal regions) often have substantially differing spectral shapes and durations, so recordings from appropriate tectonic regimes should be used whenever possible.
- **Magnitude:** Earthquake magnitude is related to the duration of ground shaking, so using ground motions from earthquakes with appropriate magnitudes should already have approximately the appropriate durations. Earthquake magnitude is also related to the shape of the resulting ground motion's response spectrum, though spectral shape is considered explicitly in Step 2 of the process, so this is not a critical factor when identifying ground motions from appropriate magnitude earthquakes.
- **Site Soil Conditions:** Site soil conditions (Site Class) exert a large influence on ground motions but are already reflected in the spectral shape used in Step 2. For Step 1, reasonable limits on site soil conditions should be imposed but should not be too restrictive as to unnecessarily limit the number of candidate motions.
- **Usable Frequency of the Ground Motion:** Only processed ground motion records should be considered for RHA. Processed motions have a usable frequency range; in active regions, the most critical parameter is the lowest usable frequency. It is important to verify that the usable frequencies of the record (after filtering) accommodate the range of frequencies important to the building response; this frequency (or period) range is discussed in the next section on scaling.
- **Period/Frequency Sampling:** Ground motion recordings are discretized representations of continuous functions. The sampling rate for the recorded data can vary from as little as 0.001 s to as much as 0.02 s, depending on the recording instrument and processing. If the sampling rate is too coarse, important characteristics of the motion, particularly in the high-frequency range, can be lost. On the other hand, the finer the sampling rate, the longer the analysis will take. Particularly for structures with significant response at periods less than 0.1 s, caution should be used to ensure that the sampling rate is sufficiently fine to capture the motion's important characteristics. As a general guideline, discretization should include at least 100 points per decade of significant response. Thus, for a structure with significant response at a period of 0.1 s, time steps should not be greater than 0.001 s.
- **Site-to-Source Distance:** The distance is a lower priority parameter to consider when selecting ground motions. Studies investigating this property have all found that response history analyses performed using ground motions from different site-to-source distances, but otherwise equivalent properties, produce practically equivalent demands on structures.

Once the preselection process has been completed, Step 2 is undertaken to select the final set of ground motions according to the following criteria:

- **Spectral Shape:** The shape of the response spectrum is a primary consideration when selecting ground motions.
- **Scale Factor:** It is also traditional to select motions such that the necessary scale factor is limited; an allowable scale factor limit of approximately 0.25 to 4 is not uncommon.
- **Maximum Motions from a Single Event:** Many also think it important to limit the number of motions from a single seismic event, such that the ground motion set is not unduly influenced by the single event. This criterion is deemed less important than limiting the scale factor, but imposing a limit of only three or four motions from a single event would not be unreasonable for most cases.

Further discussion of ground motion selection is available in NIST GCR 11-917-15 (NIST 2011), *Selecting and Scaling Earthquake Ground Motions for Performing Response-History Analyses*.

Near-fault sites have a probability of experiencing pulse-type ground motions. This probability is not unity, so only a certain fraction of selected ground motions should exhibit pulselike characteristics, while the remainder can be nonpulse records, selected according to the standard process defined previously. The probability of experiencing pulselike characteristics is dependent principally on (1) distance of site from fault, (2) fault type (e.g., strike slip or reverse), and (3) location of hypocenter relative to site, such that rupture occurs toward or away from the site.

Criteria (1) and (2) are available from conventional disaggregation of probabilistic seismic hazard analysis. Criterion (3) can be computed as well in principle, but is generally not provided in a conventional hazard analysis. However, for the long ground motion return periods associated with MCE_R spectra, it is conservative and reasonable to assume that the fault rupture is toward the site for the purposes of evaluating pulse probabilities. Empirical relations for evaluating pulse probabilities in consideration of these criteria are given in NIST GCR 11-917-15 (2011) and in Shahi et al. (2011).

Once the pulse probability is identified, the proper percentage of pulselike records should be enforced in the ground motion selection. For example, if the pulse probability is 30% and 11 records are to be used, then 3 or 4 records in the set should exhibit pulselike characteristics in at least one of the horizontal components. The PEER Ground Motion Database can be used to identify records with pulse-type characteristics. The other criteria described in the previous section should also be considered to identify pulselike records that are appropriate for a given target spectrum and set of disaggregation results.

C16.2.3 Ground Motion Modification Two procedures for modifying ground motions for compatibility with the target spectrum are available: amplitude scaling and spectral matching. Amplitude scaling consists of applying a single scaling factor to the entire ground motion record, such that the variation of earthquake energy with structural period found in the original record is preserved. Amplitude scaling also preserves record-to-record variability; however, individual ground motions that are amplitude scaled can significantly exceed the response input of the target spectrum at some periods, which can tend to overstate the importance of higher mode response in some structures. In spectral matching techniques, shaking amplitudes are modified by differing amounts at differing periods, and in some cases additional wavelets of energy are added to or subtracted from the motions, such that the response spectrum of the modified motion closely resembles the target spectrum. Some spectral matching techniques are incapable of preserving important characteristics of velocity pulses in motions and should not be used for near-fault sites where these effects are important. Spectral matching does not generally preserve the record-to-record response variability observed when evaluating a structure for unmodified motions, but it can capture the mean response well, particularly if the nonlinear response is moderate.

Vertical response spectra of earthquake records are typically significantly different from the horizontal spectra. Therefore, regardless of whether amplitude scaling or spectral matching is used, separate scaling of horizontal and vertical effects is required.

C16.2.3.1 Period Range for Scaling or Matching The period range for scaling of ground motions is selected such that the ground motions accurately represent the MCE_R hazard at the structure's fundamental response periods, periods somewhat longer than this to account for period lengthening effects associated with nonlinear response and shorter periods associated with a higher mode response. Before the 2016 edition of the standard, ground motions were required to be scaled between periods of $0.2T$ and $1.5T$. The lower bound was selected to capture higher mode response, and the upper bound, period elongation effects. In the 2016 edition, nonlinear response history analyses were performed at the MCE_R ground motion level. Greater inelastic response is anticipated at this level compared to the design spectrum, so the upper bound period has accordingly been raised from $1.5T$ to $0.2T$, where T is redefined as the *maximum* fundamental period of the building (i.e., the maximum of the fundamental periods in both translational directions and the fundamental torsional period). This increase in the upper bound period is also based on recent research, which has shown that the $1.5T$ limit is too low for assessing ductile frame buildings subjected to MCE_R motions (Haselton and Baker 2006).

For the lower bound period, the $0.2T$ requirement is now supplemented with an additional requirement that the lower bound should also capture the periods needed for 90% mass participation in both directions of the building. This change is made to ensure that when used for tall buildings and other long-period structures, the ground motions are appropriate to capture response in higher modes that have significant response.

In many cases, the substructure is included in the structural model, and this inclusion substantially affects the mass participation characteristics of the system. Unless the foundation system is being explicitly designed using the results of the response history analyses, the above 90% modal mass requirement pertains only to the superstructure behavior; the period range does not need to include the very short periods associated with the subgrade behavior.

C16.2.3.2 Amplitude Scaling This procedure is similar to those found in earlier editions of the standard, but with the following changes:

1. Scaling is based directly on the maximum direction spectrum, rather than the square root of the sum of the squares spectrum. This change was made for consistency with the MCE_R ground motion now being explicitly defined as a maximum direction motion.
2. The approach of enforcing that the average spectrum "does not fall below" the target spectrum is replaced with requirements that (a) the average spectrum "matches the target spectrum" and (b) the average spectrum does not fall below 90% of the target spectrum for any period within the period range of interest. This change was made to remove the conservatism associated with the average spectrum being required to *exceed* the target spectrum at *every* period within the period range.

The scaling procedure requires that a maximum direction response spectrum be constructed for each ground motion. For some ground motion databases, this response spectrum definition is already precomputed and publicly available (e.g., Ancheta 2012). The procedure basically entails computing the maximum acceleration response to each ground motion pair for a series of simple structures that have a single mass. This procedure is repeated for structures of different periods, allowing construction of the spectrum. A number of software tools can automatically compute this spectrum for a given time–history pair.

Figure C16.2-2 shows an example of the scaling process for an example site and structure. This figure shows how the average of the maximum direction spectra meets the target spectrum (a) and shows more detail for a single Loma Prieta motion in the scaled ground motion set (b).

C16.2.3.3 Spectral Matching Spectral matching of ground motions is defined as the modification of a real recorded earthquake ground motion in some manner, such that its response spectrum matches a desired target spectrum across a period range of interest. There are several spectral matching procedures in use, as described in the NIST GCR 11-917-15 report (NIST 2011). The recommendations in this report should be followed regarding appropriate spectral matching techniques to be applied.

This section requires that when spectral matching is applied, the average of the maximum direction spectra of the matched motions must exceed the target spectrum over the period range of interest; this is intentionally a more stringent requirement compared to the requirement for scaled unmatched motions, because the spectral matching removes variability in the ground motion spectra and also has the potential to predict lower mean response (e.g., Luco and Bazzurro 2007, Grant and Diaferia 2012).

The specific technique used to perform spectral matching is not prescribed. It is possible to match both components of motion to a single target spectrum or to match the individual components to different spectra, as long as the average maximum direction spectra for the matched records meets the specified criteria.

Spectral matching is not allowed for near-fault sites, unless the pulse characteristics of the ground motions are retained after the matching process has been completed. This is based on the concern that when common spectral matching methods are used, the pulse characteristics of the motions may not be appropriately retained.

C16.2.4 Application of Ground Motions to the Structural Model This section explains the guidelines for ground motion application for both non-near-fault and near-fault sites.

Sites That Are Not Near-Fault. In this standard, the maximum direction spectral acceleration is used to describe the ground motion intensity. This spectral acceleration definition causes a

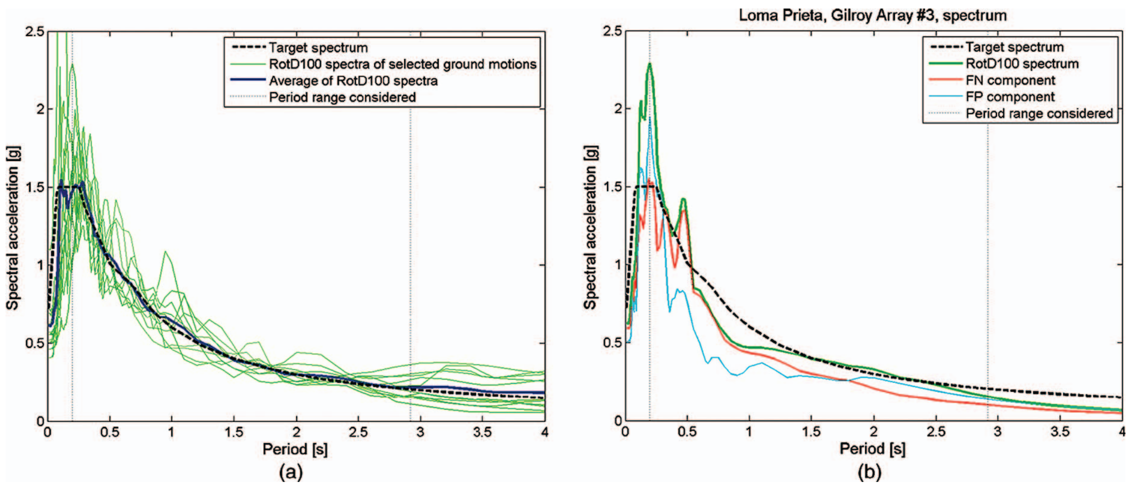

Figure C16.2-2. Ground motion scaling for an example site and structure, showing (a) the ground motion spectra for all 11 motions and (b) an example for the Loma Prieta, Gilroy array No. 3 motion.

perceived directional dependence to the ground motion. However, the direction in which the maximum spectral acceleration occurs is random at distances beyond 5 km (3.1 mi) from the fault (Huang et al. 2008), does not necessarily align with a principal direction of the building, and is variable from period to period. Accordingly, for the analysis to result in an unbiased prediction of structural response, the ground motions should be applied to the structure in a random orientation to avoid causing a biased prediction of structural response. True random orientation is difficult to achieve. Instead, the standard specifies that the average of the spectra applied in each direction should be similar to each other, such that unintentional bias in the application of motion, with one building axis experiencing greater demand than the other, is avoided.

Near-Fault Sites. Some recorded ground motions obtained from instruments located near zones of fault rupture have exhibited motions of significantly different character in one direction compared to the other. When this effect, known as directionality, occurs, it is common for the component of motion perpendicular to the fault to be stronger than that parallel to the fault and also for the fault-normal component to exhibit large velocity pulses. Sites located close to faults that can experience motion having these characteristics are termed near-fault in this standard. For such sites, the fault-normal and fault-parallel components of recorded ground motions should be maintained and applied to the corresponding orientations of the structure.

It is important to note that not all near-fault records exhibit these characteristics and also that when records do have these characteristics, the direction of maximum motion is not always aligned perpendicular to the fault strike. If an appropriate selection of records is performed, some of the records used in the analysis should have these characteristics and some should not. For those records that exhibit directionality, the direction of strong shaking is generally aligned at varying azimuths, as occurred in the original recordings. It is also important to note that because ground motions have considerable variability in their characteristics, it is specifically not intended that buildings be designed weaker in the fault-parallel direction than in the fault-normal direction.

ASCE 7-16 required all ground motions to be rotated to the fault-normal and fault-parallel directions for near-fault sites regardless of whether the selected motions were representative of a nearby fault source or a distant fault source. This provision only applied to sites where the hazard was solely from nearby faults; however, it was subsequently recognized that in some areas, the hazard is controlled by both near-fault and non near-fault seismic sources. For these areas, the suite of selected ground motions will generally consist of a combination of records from near-fault and non near-fault seismic sources. Therefore, the current version of the standard requires that for ground motions that have been selected to be representative of a nearby fault source, the fault-normal and fault-parallel components of recorded ground motions should be maintained and applied to the corresponding orientations of the structure. For ground motions selected to be representative of a distant fault source, the standard specifies that the average of the spectra applied in each direction should be similar to each other, such that unintentional bias in the application of motion, with one building axis experiencing greater demand than the other, is avoided.

C16.3 MODELING AND ANALYSIS

C16.3.1 Modeling Nonlinear response history analysis offers several advantages over linear response history analysis, including the ability to model a wide variety of nonlinear material behaviors, geometric nonlinearities (including P-delta and large displacement effects), gap opening and contact behavior, and nonlinear viscous damping, and to identify the likely spatial and temporal distributions of inelasticity. Nonlinear response history analysis has several disadvantages, however including an increased effort to develop the analytical model, increased time to perform the analysis (which is often complicated by difficulties in obtaining converged solutions), sensitivity of computed response to system parameters, large amounts of analysis results to evaluate, and the inapplicability of superposition to combine live, dead, and seismic load effects.

While computation of collapse probability is not necessary, it is important to note that mathematical models used in the analysis should have the capability to determine if collapse occurs when the structure is subjected to MCE_R level ground motions. The ability to predict collapse is important because the global acceptance criteria in Section 16.4.1.1 allow collapse (or unacceptable response) to occur for only one of the 11 ground motions for Risk Category I and II buildings and allows no such responses for Risk Categories III and IV buildings. Development of models with the ability to predict collapse requires attributes such as cyclic loss of strength and stiffness, low cycle fatigue failure, and geometric nonlinearity.

Although analytical models used to perform linear analysis, in accordance with Chapter 12, typically do not include representation of elements other than those that compose the intended lateral-force-resisting system, the gravity-load-carrying system and some nonstructural components can add significant stiffness and strength. Because the goal of nonlinear response history analysis is to accurately predict the building's probable performance, it is important to include such elements in the analytical model and also to verify that the behavior of these elements will be acceptable. This inclusion may mean that the contribution of stiffness and strength from elements considered as nonparticipating elements in other portions of this standard should be included in the response history analysis model. Since structures designed using nonlinear response history analysis must also be evaluated using linear analyses, this analysis ensures that the strength of the intended seismic force-resisting system is not reduced relative to that of structures designed using only the linear procedures.

Expected material properties are used in the analysis model, attempting to characterize the expected performance as closely as possible. It is suggested that expected properties be selected considering actual test data for the proposed elements. Where test data are not readily available, the designer may consider estimates as found in ASCE 41 and the PEER TBI Guidelines (Bozorgnia et al. 2009). Guidance on important considerations in modeling may also be found in *Nonlinear Structural Analysis for Seismic Design,* NIST GCR 10-917-5 (NIST 2010).

Two-dimensional structural models may be useful for initial studies and for checking some specific issues in a structure; however, the final structural model used to confirm the structural performance should be three-dimensional.

For certain structures, the response under both horizontal and vertical ground motions should be considered. NIST GCR 11-917-15 (NIST 2011) provides some guidance to designers considering the application of vertical ground motions. To properly capture the nonlinear dynamic response of structures where vertical dynamic response may have a significant influence on structural performance, it is necessary to include vertical mass in the mathematical model. Typically, the vertical mass must be distributed across the floor and roof plates to properly capture vertical response modes. Additional degrees of freedom (e.g., nodes at quarter points along the span of a beam) need to be added to capture this effect, or horizontal elements need to be modeled with consistent mass. Numerical convergence problems caused by large oscillatory vertical accelerations have been noted (NIST 2012), where base rotations caused by wall cracking in fiber wall models are the primary source of vertical excitation. See also the Commentary on Chapter 22.

Consideration of the additional vertical load of $(0.2S_{DS}) * D$, as per Section 12.4.2, is inappropriate for response history analysis. Response history analyses are desired to reflect actual building response to the largest extent possible. Applying an artificial vertical load to the analysis model before application of a ground motion results in an offset in the yield point of elements carrying gravity load because of the initial artificial stress. Similarly, applying an artificial vertical load to the model at the conclusion of a response history analysis is not indicative of actual building response. If vertical ground motions are expected to significantly affect response, application of vertical shaking to the analysis model is recommended. It should be noted that vertical response often occurs at higher frequencies than lateral response, and hence, a finer analysis time-step might be required when vertical motions are included.

For structures composed of planar seismic force-resisting elements connected by floor and roof diaphragms, the diaphragms should be modeled as semirigid in plane, particularly where the vertical elements of the seismic force-resisting system are of different types (such as moment frames and walls). Biaxial bending and axial force interaction should be considered for corner columns, nonrectangular walls, and other similar elements.

Nonlinear response history analysis is load path-dependent, with the results depending on combined gravity and lateral load effects. The MCE shaking and design gravity load combinations required in ASCE 7 have a low probability of occurring simultaneously. Therefore, the gravity load should instead be a realistic estimate of the expected loading on a typical day in the life of the structure. In this chapter, two gravity load cases are used. One includes an expected live loading characterizing probable live loading at the time of the MCE shaking, and the other, no live load. The case without live load is required to be considered only for those structures where live load constitutes an appreciable amount of the total gravity loading. In those cases, structural response modes can be significantly different, depending on whether the live load is present. The dead load used in this analysis should be determined in a manner consistent with the determination of seismic mass. When used, the live load is reduced from the nominal design live load to reflect both the low probability of the full design live load occurring simultaneously throughout the building and the low probability that the design live load and MCE shaking will occur simultaneously.

The reduced live load values of $0.8L_0$ for live loads that exceed $100\,\text{lb/ft}^2$ ($4.79\,\text{kN/m}^2$) and $0.4L_0$ for all other live loads, were simply taken as the maximum reduction allowable in Sections 4.7.2 and 4.7.3.

Gravity loads are to be applied to the nonlinear model first and then ground shaking simulations are applied. The initial application of gravity load is critical to the analysis, so member stresses and displacements caused by ground shaking are appropriately added to the initially stressed and displaced structure.

C16.3.3 P-delta Effects P-delta effects should be realistically included, regardless of the value of the elastic story stability coefficient $\theta = P\Delta I_e/(Vh)$. The elastic story stability coefficient is not a reliable indicator of the importance of P-delta during large inelastic deformations. This problem is especially important for dynamic analyses with large inelastic deformations because significant ratcheting can occur. During these types of analyses, when the global stiffness starts to deteriorate and the tangent stiffness of story shear to story drift approaches zero or becomes negative, P-delta effects can cause significant ratcheting (which is a precursor to dynamic instability) of the displacement response in one direction. The full reversal of drifts is no longer observed, and the structural integrity is compromised. To ascertain the full effect of P-delta effects for a given system, a comparison of static pushover curves from a P-delta model and non-P-delta model can be compared.

When including P-delta effects, it is important to capture not only the second-order behavior associated with lateral displacements but also with global torsion about the vertical axis of the system. In addition, the gravity load used in modeling P-delta effects must include 100% of the gravity load in the structure. For these reasons, the use of a single "leaning column," where much of a structure's vertical weight is lumped at a single vertical coordinate, is discouraged, and instead, the structure's vertical load should be distributed throughout the structure in a realistic manner, either through direct modeling of the gravity system or by appropriately distributed "leaning columns."

In some structures, in addition to considering P-delta effects associated with global structural deformation, it is also important to consider local P-delta effects associated with the local deformation of members. This is particularly important for slender elements that are subject to buckling.

C16.3.4 Torsion Inherent torsion is actual torsion caused by differences in the location of the center of mass and center of rigidity throughout the height of the structure. Accidental torsion effects, as per Section 12.8.4.2, are artificial effects that attempt to account for actual variations in load and material strengths during building operation that differ from modeling assumptions. Some examples of this difference would be nonuniformity of the actual mass in the building, openings in the diaphragm that are unaccounted for, torsional foundation input motion caused by the ground motion being out of phase at various points along the base, the lateral stiffness of the gravity framing, variation in material strength and stiffness caused by typical construction tolerances, and incidental stiffness contribution by the nonstructural elements.

When the provision for accidental torsion was first introduced, it was to address buildings that have no inherent torsion but are sensitive to torsional excitation. Common examples of this type of configuration are cruciform core or I-shaped core buildings. In reality, many things can cause such a building to exhibit some torsional response. None of the aforementioned items are typically included in the analysis model; therefore, the accidental torsion approach was introduced to ensure that the structure has some minimum level of resistance to incidental twisting under seismic excitation.

The accidental torsion also serves as an additional check to provide more confidence in the torsional stability of the structure. During the initial proportioning of the structure using linear analysis (as per Section 16.1.1), accidental torsion is required to be enforced in accordance with Section 12.8.4.2. When there is no inherent torsion in the building, accidental torsion is a crucial step in the design process because this artificial offset in the center of mass is a simple way to force a minimum level of twisting to occur in the building. The accidental torsion step (i.e., the required 5% force offsets) is also important when checking for plan irregularities in symmetric and possibly, torsionally flexible buildings. Where there is already inherent torsion in the building, additional accidental torsion is not generally a crucial requirement (though still required, in accordance with Section 12.8.4.2) because the building model will naturally twist during analysis, and no additional artificial torsion is required for this twisting to occur. However, for buildings exhibiting either torsional or extreme torsional irregularities, inclusion of accidental torsion in the nonlinear analysis is required by this standard to assist in identification of potential nonlinear torsional instability.

C16.3.5 Damping Viscous damping can be represented by combined mass and stiffness (Rayleigh) damping. To ensure that the viscous damping does not exceed the target level in the primary response modes, the damping is typically set at the target level for two periods, one above the fundamental period and one below the highest mode frequency of significance. For very tall buildings, the second and even third modes can have significant contributions to response; in this case, the lower multiple on T_1 may need to be reduced to avoid excessive damping in these modes.

Viscous damping may alternatively be represented by modal damping, which allows for the explicit specification of the target damping in each mode.

Various studies have shown that the system damping may vary with time as the structure yields, and in some cases, damping well above the target levels can temporarily exist. Zareian and Medina (2010) provide recommendations for the implementation of damping in such a way that the level of viscous damping remains relatively constant throughout the response.

The level of structural damping caused by component-level hysteresis can vary significantly based on the degree of inelastic action. Typically, hysteretic damping provides a contribution less than or equal to 2.5% of critical damping.

Damping and/or energy dissipation caused by supplemental damping and energy dissipation elements should be explicitly accounted for with component-level models and not included in the overall viscous damping term.

C16.3.6 Explicit Foundation Modeling The PEER TBI guidelines (Bozorgnia et al. 2009) and NIST GCR 12-917-21 (NIST 2012) both recommend the inclusion of subterranean building levels in the mathematical model of the structure. The modeling of the surrounding soil has several possible levels of sophistication, two of which are depicted below in (b) and (c) of Figure C16.3-1, which are considered most practical for current practice. For a MCE_R-level assessment, which is the basis for the Chapter 16 RHA procedure, the rigid bathtub model is preferred by PEER TBI (Bozorgnia et al. 2009) and NIST (2012) (Figure C16.3-1c). This model includes soil springs and dashpots, and identical horizontal ground motions are input at each level of the basement. Such a modeling approach, where the soil is modeled in the form of springs and/or dashpots (or similar methods) placed around the

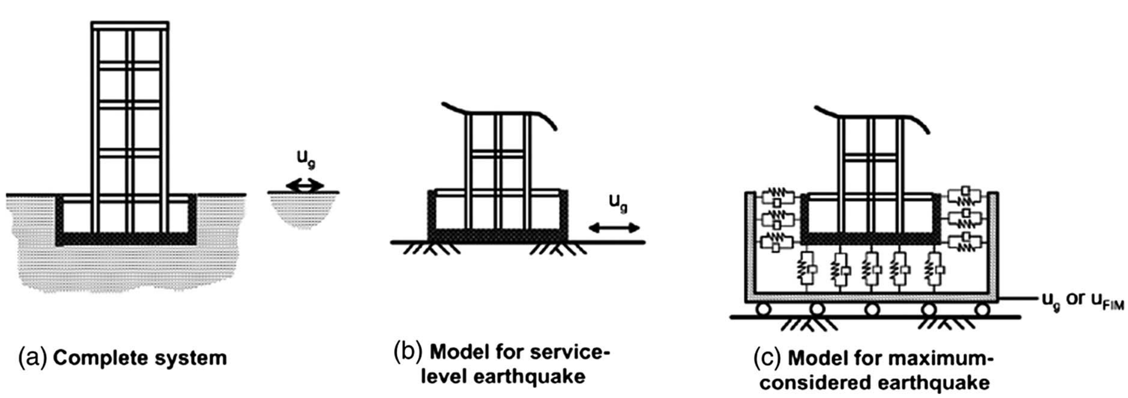

Figure C16.3-1. Illustration of the method of inputting ground motions into the base of the structural model.
Source: NIST (2011).

foundation, is encouraged but is not required. When spring and dashpot elements are included in the structural model, horizontal input ground motions are applied to the ends of the horizontal soil elements rather than being applied to the foundation directly. A simpler but less accurate model is to exclude the soil springs and dashpots from the numerical model and apply the horizontal ground motions at the bottom level of the basement (Figure C16.3-1b), which is fixed at the base. Either the fixed-base (Figure C16.3-1b) or bathtub (Figure C16.3-1c) approach is allowed, but the bathtub approach is encouraged because it is more accurate.

For the input motions, the PEER TBI (Bozorgnia et al. 2009) guidelines allow the use of either the free-field motion, which is the motion defined in Section 16.2.2, or a foundation input motion modified for kinematic interaction effects. Guidelines for modeling kinematic interaction are contained in NIST (2012).

More sophisticated procedures for soil–structure interaction modeling, including the effects of multisupport excitation, can also be applied in RHA. Such analyses should follow the guidelines presented in NIST (2012).

Approximate procedures for the evaluation of foundation springs are provided in Chapter 19 of this standard.

C16.4.1 Global Acceptance Criteria

C16.4.1.1 Unacceptable Response This section summarizes the criteria for determining unacceptable response and how the criteria were developed. It must be made clear that these unacceptable response acceptance criteria are not the primary acceptance criteria that ensure adequate collapse safety of the building but rather the story drift criteria and the element-level criteria discussed later in Section C16.4. The unacceptable response acceptance criteria were developed to be a secondary protection to supplement the primary criteria. Unacceptable responses result in instabilities and loss of gravity load support. Consequently, if it can be shown that after a deformation-controlled element reaches its (collapse prevention) limit, the model is able to redistribute demands to other elements, this would not constitute an unacceptable response. The acceptance criteria were intentionally structured in this manner because there is high variability in unacceptable response (as described in this section) and the other primary acceptance criteria are much more stable and reliable (because they are based on mean values of 11 motions rather than the extreme response of 11 motions).

When performing nonlinear analysis for a limited suite of ground motions, the observance of a single unacceptable response (or, conversely, the observance of no unacceptable responses) is statistically insignificant. That is, it is reasonably probable that no collapses will be observed in a small suite of analyses, even if the structure has a greater than 10% chance of collapse at MCE_R shaking levels. It is also possible that a structure with less than a 10% chance of collapse at MCE_R shaking levels will still produce an unacceptable response for one ground motion in a small suite. In order for statistics on the number of unacceptable responses in a suite of analyses to produce a meaningful indication of collapse probability, a very large suite of analyses must be performed. Furthermore, the observance or nonobservance of an unacceptable response depends heavily on how the ground motions were selected and scaled (or spectrally matched) to meet the target spectrum.

Since the observance or nonobservance of an unacceptable response is not statistically meaningful, the standard does not rely heavily on the prohibition of unacceptable responses in the attempt to "prove" adequate collapse safety. The many other acceptance criteria of Section 16.4 are relied upon to implicitly ensure adequate collapse safety of the building. If one desired to expand the unacceptable response acceptance criteria to provide true meaningful collapse safety information about the building, a more complex statistical inference approach would need to be used. This is discussed further below.

The statistical insignificance of unacceptable response in a small suite of analyses leaves a large open question about how to interpret the meaning of such responses when they occur. Even though occurrence of a single unacceptable response is statistically meaningless, the occurrence of many unacceptable responses (e.g., 5 of 11) indicates that the collapse probability is significantly in excess of 10%. In addition, a conscientious structural designer is concerned about such occurrences, and the occurrences of unacceptable responses may provide the designer some insight into possible vulnerabilities in the structural design.

Some engineers presume that the acceptance criteria related to *average* response effectively disallow any unacceptable responses (because you cannot average in an infinite response scenario), while others presume that *average* can also be interpreted as *median,* which could allow almost half of the ground motions to cause an unacceptable response.

The statistics presented below are provided to help better interpret the meaning of observance of a collapse of the model analyzed or other type of unacceptable response in a suite of analyses. These simple statistics are based on predicting the occurrence of collapse (or other unacceptable response) using a binomial distribution, based on the following assumptions:

- The building's collapse probability is exactly 10% at the MCE_R level.
- Collapse probability is lognormally distributed and has a dispersion (lognormal standard deviation) of 0.6. This value includes all sources of uncertainty and variability (e.g., record-to-record variability, modeling uncertainty). The value of 0.6 is the same value used in creating the risk-consistent hazard maps for ASCE 7-10 (FEMA 2009a) and is consistent with the values used in FEMA P-695 (FEMA 2009b).
- The record-to-record variability ranges from 0.25 to 0.40. This is the variability in the collapse capacity that would be expected from the analytical model. This value is highly dependent on the details of the ground motion selection and scaling; values of 0.35 to 0.45 are expected for motions that do not fit tightly to the target spectrum, and values of 0.2 to 0.3 are expected for spectrally matched motions (FEMA 2009b).

Figure C16.4-1 shows collapse fragility curves for a hypothetical building that has a 10% collapse probability conditioned on MCE_R motion ($P[C|MCE_R] = 10\%$), with an assumed record-to-record collapse uncertainty of 0.40 and a total collapse uncertainty of 0.60. The figure shows that the median collapse capacity must be a factor of 2.16 above the MCE_R ground motion level, that the probability of collapse is 10% at the MCE_R when the full variability is included (as required), but that the probability of collapse is only 2.7% at the MCE_R when only the record-to-record variability is included. This 2.7% collapse probability is what would be expected from the structural model that is used in the RHA assessment procedure.

Table C16.4-1 shows the probability of observing n collapses in a suite of 11 ground motions for a structure that has different values of $P[C|MCE_R]$.

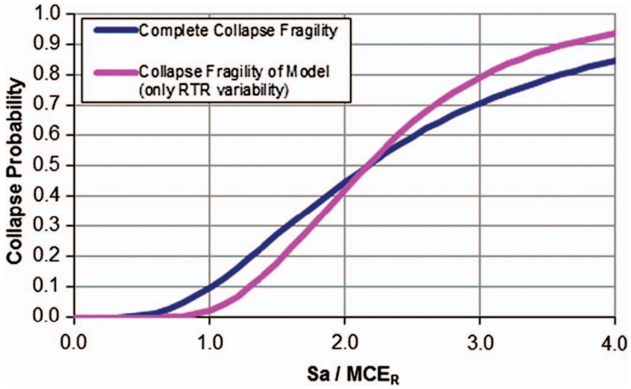

Figure C16.4-1. Collapse fragilities for a building with $P[C|MCE_R] = 10\%$ and $\beta_{COL;RTR} = 0.40$.

Table C16.4-1 shows that for a building meeting the $P[C|MCE_R] = 10\%$ performance goal, there is a 74% chance of observing no collapses, a 23% chance of observing one collapse, a 3% chance of observing two collapses, and virtually no chance of observing more than two collapses. In comparison, for a building with $P[C|MCE_R] = 20\%$, there is a 30% chance of observing no collapses, a 38% chance of observing one collapse, a 22% chance of observing two collapses, and a 10% chance of observing more than two collapses.

This table illustrates that

- Even if no collapses are observed in a set of 11 records, this does not, in any way, prove that the $P[C|MCE_R] = 10\%$ performance goal has been met. For example, even for a building with $P[C|MCE_R] = 20\%$, there is still a 30% chance that no collapses will be observed in the analysis. Therefore, the other noncollapse acceptance criteria (e.g., criteria for drifts and element demands) must be relied on to enforce the 10% collapse probability goal.
- If the $P[C|MCE_R] = 10\%$ performance goal is met, it is highly unlikely (only a 3% chance) that two collapses will be observed in the set of 11 records. Therefore, an acceptance criterion that prohibits two collapses is reasonable.

The collapse likelihoods shown in Table C16.4-1 are based on a relatively large record-to-record variability value of 0.40. Table C16.4-2 illustrates similar statistics for the case when the record-to-record variability is suppressed in ground motion selection and scaling, such as occurs with spectral matching. This table shows that, for a building meeting the $P[C|MCE_R] = 10\%$ performance goal and with record-to-record variability taken as 0.25, the likelihood of observing a collapse response is very low. This is why no unacceptable responses are permitted in the suite of analyses when spectral matching is used.

For Risk Categories I and II structures, if more than 11 ground motions are used for analysis, then additional unacceptable responses may be permissible. Two unacceptable responses would be permissible if 20 or more motions are used, and three unacceptable responses would be permissible if 30 or more motions are used. For Risk Categories III and IV structures, the collapse probability goals are 6% and 3%, respectively, at the MCE_R level. When the above computations are redone using these lower collapse probability targets, this shows that the acceptance criteria should require that no motions of the 11 produce an unacceptable response for these categories.

Typically, mean building response values (story drifts, element deformations, and forces) are used in acceptance evaluations, where the "mean" is the simple statistical average for the response parameter of interest. When an unacceptable response occurs, it is not possible to compute a mean value of the building response values because one of the 11 response quantities is

Table C16.4-1. Likelihood of Observing Collapses in 11 Analyses, Given Various MCE_R Collapse Probabilities and a Record-to-Record Uncertainty of 0.4.

Number of Collapses	Likelihood for Various $P[C|MCE_R]$ Values				
	0.05	0.10	0.15	0.20	0.30
0 of 11	0.93	**0.74**	0.51	0.30	0.07
1 of 11	0.07	**0.23**	0.36	0.38	0.21
2 of 11	0	**0.03**	0.11	0.22	0.29
3 of 11	0	**0**	0.02	0.08	0.24
4 of 11	0	**0**	0	0.02	0.13
5 of 11	0	**0**	0	0	0.05

Table C16.4-2. Likelihood of Observing Collapses in 11 Analyses, Given Various MCE_R Collapse Probabilities and a Record-to-Record Uncertainty of 0.25.

Number of Collapses	Likelihood for Various $P[C|MCE_R]$ Values				
	0.05	0.10	0.15	0.20	0.30
0 of 11	1.00	**0.99**	0.93	0.79	0.30
1 of 11	0	**0.01**	0.07	0.19	0.38
2 of 11	0	**0**	0	0.02	0.22
3 of 11	0	**0**	0	0	0.08
4 of 11	0	**0**	0	0	0.02
5 of 11	0	**0**	0	0	0

undefined. In this case, rather than the mean, the standard requires use of the counted median response multiplied by 1.2 but not less than the mean response from the remaining motions.

To compute the median value, the unacceptable response is assumed as larger than the other responses and then, assuming that 11 analyses were performed, the counted median value is taken to be the 6th largest value from the set of 11 responses. The 1.2 factor is based on a reasonable ratio of mean to median values for a lognormal distribution ($\beta = 0.4$ results in mean/median = 1.08, $\beta = 0.5$ results in mean/median = 1.13, $\beta = 0.6$ results in mean/median = 1.20, and $\beta = 0.7$ results in mean/median = 1.28).

The requirement to also check the mean of the remaining 10 response results is simply an added safeguard to ensure that the 1.2 × median value does not underpredict the mean response values that should be used when checking the acceptance criteria.

Although currently the purpose of this acceptance criterion is not meant to quantify the structure's collapse probability under MCE_R ground motions, the acceptance criterion can be recast to do so in future provisions. The collapse probability can be inferred from analysis results and compared to the target value (e.g., 10% for structures in Risk Category I or II). In this alternate light, existing statistical inference theories can be used to determine the number of acceptable responses, and the number of ground motions required to conclude that the proposed design may have an acceptable collapse probability.

As discussed in the previous section, analysis results can be thought of as following a binomial distribution. Based on this distribution, one could use the observed counts of collapsed and noncollapsed responses (indicated by unacceptable and acceptable responses) to estimate the collapse probability of the proposed design in a manner that accounts for the uncertainty in the estimated collapse probability. This uncertainty depends on the total number of ground motions. If few ground motions are used, there is a large uncertainty in the collapse probability. If many ground motions are used, there is a small uncertainty. For example, compare a set of 11 ground motions with one unacceptable response to a set of 110 ground motions with ten unacceptable responses. Both sets most likely have a unacceptable response probability of 9.1%. The design with one unacceptable and ten acceptable responses has only a 34% chance that its unacceptable response probability is 10% or less. The design with 10 unacceptable and 100 acceptable responses has a 56% chance that its unacceptable response probability is 10% or less.

In the current acceptance criterion, the choice to require 11 ground motions follows from the need to have confidence in the average values of the resulting element-level and story-level responses (Section C16.2.3.1). These element-level and story-level responses are then used to *implicitly* demonstrate adequate collapse safety. If future provisions seek to *explicitly* ensure that the proposed design has an acceptable collapse probability, then this unacceptable response acceptance criterion should be revised using statistical inference theory to establish the number of required ground motions and the maximum number of unacceptable responses, as well as the element- and story-level response limits.

C16.4.1.2 Transient Story Drift The limit on mean story drift was developed to be consistent with the linear design procedures of this standard. To this end, the basic Table 12.12-1 story drift limits are adopted with the following adjustments:

- Increased by a factor of 1.5, to reflect the analysis being completed at the MCE_R ground motion level rather than at 2/3 of the MCE_R level, and
- Increased by another factor of 1.25, to reflect an average ratio ofR/C_d.

These two increases are the basis for the requirement that the mean story drift be limited to 1.9 (which was rounded to 2.0) of the standard Table 12.12-1 limits.

The masonry-specific drift limits of Table 12.12-1 are not enforced in this section because the component-level acceptance criteria of Section 16.4.2 are expected to result in equivalent performance (i.e., a masonry building designed in accordance with Chapter 16 is expected to have similar performance to a masonry building designed using linear analysis methods and the more stringent drift limits of Table 12.12-1).

For tall structures, more restrictive drift limits are adopted than permitted under Table 12.2.1. When performing nonlinear response history analysis, the model is required to include all elements that significantly affect the stiffness and strength of the structure to resist lateral forces. This is in stark contrast to the intent of Chapter 12 analysis procedures, which consider only the seismic force-resisting system in the analysis. As a result, nonlinear response history analyses will include elements normally considered to carry only gravity loads, but which, particularly in tall buildings, add substantial stiffness to the analytical model. Analysis of these models will result in lower predictions of drift than the typical models used for linear analysis. If the same drift limits were applied, the effect would be to permit more flexible structures than permitted by Chapter 12. The drift limit applied for buildings over 240 ft is the same as that in the 2017 PEER Tall Buildings Initiative, Guidelines for Performance-Based Seismic Design of Tall Buildings, which is commonly used for the design of tall structures in the US. For buildings with heights between 100 ft (30 m) and 240 ft (73 m), Equation (16.4-1) applies and allows for a smooth transition between the requirements for tall buildings and the doubled limits of Table 12.12-1 requirement, as applied to Risk Category I and II mid-rise structures, other than masonry. For building heights above 240 ft (73 m), the story drift limit of 0.03 hsx controls over the value calculated per Equation (16.4-1).

C16.4.1.3 Residual Story Drift The standard does not require checks on residual drift for structures less than 240 ft (73 m) in height. Residual drift is the unrecoverable portion of transient drift that commonly occurs when a structure undergoes inelastic response. It is computed as the absolute value of the largest difference of the deflections of vertically aligned points at the top and bottom of the story under consideration along any edge of the structure, following completion of the structure's response to earthquake motion. Residual drifts are an indicator of potential incipient dynamic instability, and the prudent engineer checks for this instability. Limiting residual drifts is an important consideration for post-earthquake operability and for limiting financial losses, but such performance goals are not included in the scope of the ASCE 7 standard. For Risk Category I and II buildings, the ASCE 7 standard is primarily meant to ensure the protection of life safety.

Design practice for tall buildings, however, has routinely included limits on residual drift. One important reason for this is that many tall buildings are located in dense urban areas. Residual drift is commonly used to judge the post-earthquake safety of damaged buildings. If tall buildings exhibit large residual drifts after an earthquake and consequently appear to be unstable, this could prompt building officials to cordon off large areas, imposing hardship on many people and businesses. The tall building community therefore designs to reduce the probability of such occurrence. ASCE 7 adopted the acceptance directly from the recent practice for tall buildings exceeding 240 ft (73 m) in the 2022 edition to be compatible with common design practice for these structures.

C16.4.2 Element-Level Acceptance Criteria The element-level acceptance criteria requires classification of each element action as either force-controlled or deformation-controlled, similar to the procedures of ASCE 41. Note that this is done for each *element action*, rather than for each *element*. For example, for a single column element, the flexural behavior may be classified as a deformation-controlled action, whereas the axial behavior may be classified as a force-controlled action.

Deformation-controlled actions are those that have reliable inelastic deformation capacity. Force-controlled actions pertain to brittle modes where inelastic deformation capacity cannot be ensured. Based on how the acceptance criteria are structured, any element action that is modeled elastically must be classified as being force-controlled.

Some examples of force-controlled actions are

- Shear in reinforced concrete (other than diagonally reinforced coupling beams);
- Axial compression in columns;
- Punching shear in slab–column joints without shear reinforcing;
- Connections that are not explicitly designed for the strength of the connected component, such as some braces in braced frames;
- Displacement of elements resting on a supporting element without rigid connection (such as slide bearings); and
- Axial forces in diaphragm collectors.

Some examples of deformation-controlled actions are

- Shear in diagonally reinforced coupling beams;
- Flexure in reinforced concrete columns and walls;
- Axial yielding in buckling restrained braces; and
- Flexure in special moment frames.

Section 16.4.2 further requires categorization of component actions as critical, ordinary, or noncritical based on the consequence of their exceeding strength or deformation limits. Because of the differences in consequence, the acceptance criteria are developed differently for each of the above classifications of component actions. An element's criticality is judged based on the extent of collapse that may occur, given the element's failure, and is also judged on whether the effect of the element's failure on seismic resistance is substantive. An element's failure could be judged to have substantial effect on the structure's seismic resistance if analysis of a model of the building without the element present predicts unacceptable performance, while analysis with the element present does not.

Limits placed on response quantities are correlated to building performance and structural reliability. In order for compliance with these limits to meaningfully characterize overall performance and reliability, grouping of certain component actions for design purposes may be appropriate. For example, while symmetrical design forces may be obtained for symmetrical structures using equivalent lateral force and modal response spectrum analysis procedures, there is no guarantee that component actions in response history analysis of symmetrical models will be the same—or even similar—for identical components arranged symmetrically. Engineering judgment should be applied to the design to maintain symmetry by using the greater demands (that is, the demands on the more heavily loaded component determined using the appropriate factor on its mean demand) for the design of both components. For this purpose, using the mean demands of the pair of components would not be appropriate because this method would reduce the demand used for design of the more heavily loaded component.

Though this point is perhaps trivial in the case of true symmetry, it is also a concern in nonsymmetrical structures. For these buildings, it may be appropriate to group together structural components that are highly similar either in geometric placement or purpose. The demands determined using the suite mean (the mean response over all ground motions within a suite) may be very different for individual components within this grouping. This is a result both of the averaging process and the limited explicit consideration of ground motion to structure orientation in the provisions. Although the analysis may indicate that only a portion of the grouped components do not meet the provisions, the engineer ought to consider whether such nonconformance should also suggest redesign in other similar elements. Thus, response history analysis places a higher burden on the judgment of the engineer to determine the appropriate methods for extracting and interpreting meaningful response quantities for design purposes.

C16.4.2.1 Force-Controlled Actions The application of the load combinations and resistance factors in Equations (16.4-2), (16-4-2), (16.4-3), and (16.4-4) are limited to load effects determined using NLRHA for structures subject to an MCE_R level event. These load and resistance factors should not be used for structural design with the design earthquake effects, *E*, determined in accordance with Chapter 12 of the standard. When evaluating earthquake effects in accordance with Chapter 12 of the standard, the load combinations of Chapter 2 apply.

The acceptance criteria for force-controlled actions Equations 16.4-1 and 16.4-2 follow the same framework that underlies the load combinations contained in Chapter 2 of the standard, except that both the load factors and the capacity (resistance) factors have been adjusted for consistency with the NLRHA approach and to maintain compatibility with the target reliability in Table 1.3-2. The load factor of 1.3Ie on seismic demands was determined assuming a demand dispersion associated with record-to-record variability of 0.3, based on values observed for several real buildings and additional modeling uncertainty of 0.2, which was selected based on engineering judgment consistent with approaches used in FEMA P695. Based on data presented in National Bureau of Standard Special Publication 577, the typical bias in resistance, (i.e. the ratio of the expected strength of an element to the nominal strength), was taken as 1.1, while a 0.15 coefficient of variation on resistance was assumed. The materials standards have resistance factors that generally vary from values of approximately 0.7 to 0.9. An average value of 0.85 was used in computing the 1.3 load factor on seismic demand.

Exception 2 to the section permits the use of an alternative set of load combinations (16.4-3 and 16.4-4) to evaluate force-controlled actions when demands are limited by the development of a plastic mechanism. For example, shear in a column in a moment frame cannot exceed the sum of the plastic moments at the ends of the column, divided by the free height of the column. A load factor of unity is used on the capacity-limited earthquake demand in this case, recognizing the very low probability that demand can exceed the computed value and also for consistency with similar criteria in ACI 318 and AISC 341.

To determine appropriate values of λ, we begin with the collapse probability goals of Table 1.3-2 (for Risk Categories I and II) for MCE_R motions. These collapse probability goals include a 10% chance of a total or partial structural collapse and a 25% chance of a failure that could result in endangerment of individual lives. For the assessment of collapse, we then make the somewhat conservative assumption that the failure of a single

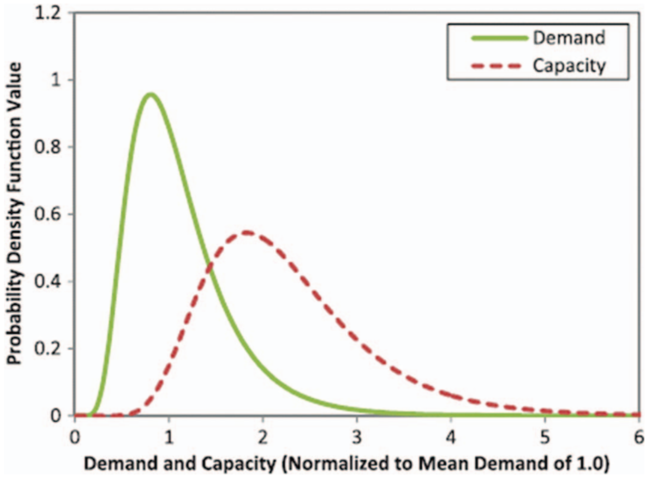

Figure C16.4-2. Illustration of component capacity and demand lognormal distributions (normalized to a mean capacity of 1.0); the mean component capacity is calibrated to achieve $P[C|MCE_R] = 10\%$.

critical force-controlled component would result in a total or partial structural collapse of the building.

Focusing first on the goal of a 10% chance of a total or partial structural collapse, we assume that the component force demand and component capacity both follow a lognormal distribution and that the estimate of $F_{n,e}$ represents the true expected strength of the component. We then calibrate the λ value required to achieve the 10% collapse probability goal. This value is depicted in Figure C16.4-2, which shows the lognormal distributions of component capacity and component demand.

The calibration process is highly dependent on the uncertainties in component demand and capacity. Table C16.4-3a shows typical uncertainties in force *demand* for analyses at the MCE_R ground motion level, for both the general case and the case where the response parameter is limited by a well-defined yield mechanism. Table C16.4-3b shows typical uncertainty values for the component *capacity*. The values are based on reference materials, as well as the collective experience and professional judgment of the development team.

In the calibration process, the λ and φ values both directly affect the required component strength. Therefore, the calibration is completed to determine the required value of λ/φ needed to fulfill the 10% collapse safety objective. This calibration is done by assuming a value ofλ/φ, convolving the lognormal distributions of demand and capacity and iteratively determining the capacity required to meet the 10% collapse safety objective by adjusting λ/φ. Table C16.4-4 reports the final λ/φ values that come from such integration.

Table C16.4-3a Assumed Variability and Uncertainty Values for Component Force Demand.

Demand Dispersion (β_D) General	Demand Dispersion (β_D) Well-Defined Mechanism	Variabilities and Uncertainties in the Force Demand
0.40	0.20	Record-to-record variability (for MCE_R ground motions)
0.20	0.20	Uncertainty from estimating force demands using structural model
0.13	0.06	Variability from estimating force demands from mean of only 11 ground motions
0.46	**0.29**	$\beta_{D\text{-Total}}$

Table C16.4-3b. Assumed Variability and Uncertainty Values for Component Force Capacity.

Capacity Dispersion (β_C) General	Capacity Dispersion (β_C) Well-Defined Mechanism	Variabilities and Uncertainties in the Final As-Built Capacity of the Component
0.30	0.30	Typical variability in strength equation for $F_{n,e}$ (from available data)
0.10	0.10	Typical uncertainty in strength equation for $F_{n,e}$ (extrapolation beyond available data)
0.20	0.20	Uncertainty in as-built strength because of construction quality and possible errors
0.37	**0.37**	$\beta_{C\text{-Total}}$

It should be clearly stated that this approach of calibrating the λ/φ ratio means that the final acceptance criterion is independent of the φ value specified by a material standard. If it is desirable for the acceptance criteria to be partially dependent on the value of φ, then the uncertainty factors of Table C16.4-3b would need to be made dependent on the φ value in some manner.

Since the values in Table C16.4-4 are similar, for simplicity, the acceptance criterion is based on $\lambda/\varphi = 2.0$ for all cases, and a separate criterion for the existence of a well-defined mechanism is not included. In addition, the strength term is defined slightly differently. For Risk Categories III and IV, this full calculation was redone using the lower collapse probability goals of 6% and 3%, respectively, and it was found that scaling the force demands by I_e sufficiently achieves these lower collapse probability goals.

This statistical calculation was then repeated for the goal of 25% chance of a failure that could result in endangerment of individual lives. This resulted in a required ratio of 1.5 for such force-controlled failure modes; deemed as "ordinary."

Force-controlled actions are deemed noncritical if the failure does not result in structural collapse or any meaningful endangerment to individual lives. This occurs in situations where gravity forces can reliably redistribute to an alternate load path and no failure will ensue. For noncritical force-controlled components, the acceptance criteria allow the use of $\lambda = 1.0$.

Where an industry standard does not define expected strength, expected (or mean) strength, F_e, is computed as follows. First, a standard strength-prediction equation is used from a material standard, using a strength reduction factor, φ, of 1.0; the expected material properties are also used in place of nominal material properties. In some cases, this estimate of strength ($F_{n,e}$) may still be conservative in comparison with the mean expected strength shown by experimental tests (F_e) caused by inherent conservatism in the strength equations adopted by the materials standards.

Table C16.4-4. Required Ratios of λ/φ to Achieve the 10% Collapse Probability Objective.

Dispersion	Required Ratios of λ/φ
General	2.1
Well-Defined Mechanism	1.9

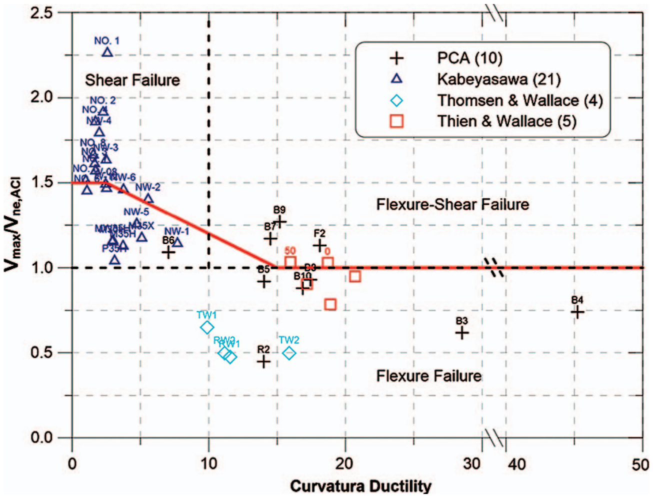

Figure C16.4-3. Expected shear strengths (in terms of $F_e/F_{n;e}$) for reinforced concrete shear walls when subjected to various levels of flexural ductility.
Source: Courtesy of John Wallace.

If such conservatism exists, the $F_{n,e}$ value may be multiplied by a "component reserve strength factor" greater than 1.0 to produce the estimate of the mean expected strength (F_e). This process is illustrated in Figure C16.4-3, which shows the $F_e/F_{n,e}$ ratios for the shear strengths from test data of reinforced concrete shear walls (Wallace et al. 2013). This figure shows that the ratio of $F_e/F_{n,e}$ depends on the flexural ductility of the shear wall, demonstrating that $F_e = 1.0\ F_{n,e}$ is appropriate for the shear strength in the zone of high flexural damage and $F_e = 1.5\ F_{n,e}$ may be appropriate in zones with no flexural damage.

For purposes of comparison, Equation (C16.4-1) is comparable to the PEER TBI acceptance criteria (Bozorgnia et al. 2009) for the case that $\varphi = 0.75$ and $F_e = 1.0F_{n,e}$.

The exception allows for the use of the capacity design philosophy for force-controlled components that are "protected" by inelastic fuses, such that the force delivered to the force-controlled component is limited by the strength of the inelastic fuse.

The following are some examples of force-controlled actions that are deemed to be critical actions:

- Steel Moment Frames (SMF):
  - Axial compression forces in columns caused by combined gravity and overturning forces
  - Combined axial force, bending moments, and shear in column splices
  - Tension in column base connections (unless modeled inelastically, in which case it would be a deformation-controlled component);
- Steel Braced Frames (BRBF - Buckling Restrained Braced Frame, SCBF - Special Concentrically Braced Frames):
  - Axial compression forces in columns caused by combined gravity and overturning forces
  - Combined axial force, bending moments, and shear in column splices
  - Tension in brace and beam connections;
  - Column base connections (unless modeled inelastically)
- Concrete Moment Frames:
  - Axial compression forces in columns caused by combined gravity and overturning forces

Figure C16.4-4. Plan view of sample building showing arrangement of concrete shear walls.

  - Shear force in columns and beams;
- Concrete or Masonry Shear Walls:
  - Shear in concrete shear wall, in cases when there is limited ability for the shear force to transfer to adjacent wall panels. For cases of isolated shear walls (i.e., WALL #1 in Figure C16.4-4), the shear force in this isolated wall is deemed as a critical action. In contrast, the shear force in a one-wall pier that is in a group of wall piers (e.g., panel #2 of Figure C16.4-5) need not be deemed a critical action (especially when determining whether an analysis is deemed to represent an unacceptable response). For this case of a group of wall piers, it may be appropriate to consider the sum of the wall shears to be the critical action (e.g., the sum of wall shears in PANELS #1, #2, and #3 of Figure C16.4-5).
  - Axial (plus flexural) compression in concrete shear wall (for most cases)
  - Axial compression in outrigger columns
  - Axial (plus flexural) tension in outrigger column splices;
- Other Types of Components:
  - Shear forces in piles and pile cap connections (unless modeled inelastically)
  - Shear forces in shallow foundations (unless modeled inelastically)
  - Punching shear in slabs without shear reinforcing (unless modeled inelastically)
  - Diaphragms that transfer a substantial amount of force (from more than one story)
  - Elements supporting discontinuous frames and walls;

The following are some examples of force-controlled actions that are deemed to be ordinary actions:

- Steel Moment Frames (SMF):
  - Shear force in beams and columns
  - Column base connections (unless modeled inelastically)

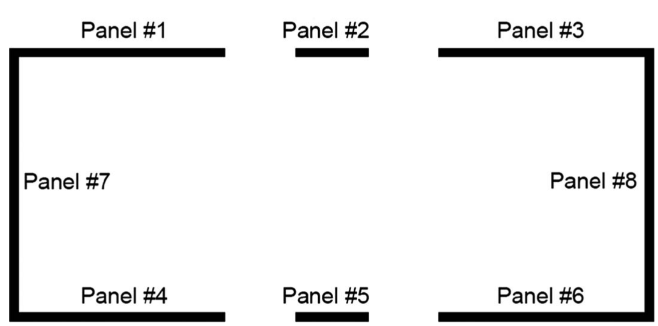

Figure C16.4-5. Plan view of sample building showing components of a reinforced concrete core shear wall.

- Welded or bolted joints (as distinct from the inelastic action of the overall connection) between moment frame beams and columns;
- Steel Braced Frames (BRBF, SCBF):
  - Axial tension forces in columns caused by overturning forces (unless modeled inelastically);
- Concrete Moment Frames:
  - Splices in longitudinal beam and column reinforcement;
- Concrete or Masonry Shear Walls:
  - An ordinary classification would only apply in special cases where failure would not cause widespread collapse and would cause minimal reduction in the building seismic resistance.
- Other Types of Components:
  - Axial forces in diaphragm collectors (unless modeled inelastically)
  - Shear and chord forces in diaphragms (unless modeled inelastically)
  - Pile axial forces;

The following are some examples of force-controlled actions that could be deemed noncritical actions:

- Any component where the failure would not result in either collapse or substantive loss of the seismic resistance of the structure.

C16.4.2.2 Deformation-Controlled Actions ASCE 7-16 included requirements to evaluate the adequacy of deformation-controlled actions against limiting deformation levels at which laboratory testing suggested either that element behavior was unpredictable or that loss of vertical load-carrying ability would occur. While substantive data exist to indicate the capacity of force-controlled actions, there are relatively few laboratory data to indicate the deformation at which a deformation-controlled element action reaches a level where loss of vertical load-carrying capacity occurs. There are a number of reasons for this, including the following: (1) the deformation at which such loss occurs can be very large and beyond the practical testing capability of typical laboratory equipment; (2) many researchers have tested such components with the aim of quantifying useful capacity for elements of a seismic force-resisting system and have terminated testing after substantial degradation in strength has occurred, even though actual failure has not yet been experienced; and (3) testing of gravity-load-bearing elements to failure can be dangerous and destructive of test equipment. In ASCE 7-22, this requirement was dropped in favor of an approach in which the demand imposed on deformation-controlled actions is evaluated against the valid range of modeling for that element in each analysis. The valid range of modeling is defined as that limiting deformation at which the model used in the analysis can be benchmarked against suitable laboratory test data. This approach has been successfully used in the design of tall buildings, using the procedures specified in the Pacific Earthquake Engineering Research Center, Guidelines for Performance-based Seismic Design of Tall Buildings, and was found to produce acceptable designs compared with prior practice. In addition, this approach will be facilitated with the adoption of data indicating the valid range of nonlinear modeling for deformation-controlled elements by both ACI and AISC in their standards.

C16.4.2.3 Elements of the Gravity Force-Resisting System The basic deformation-compatibility requirement of ASCE 7-16, Section 12.12.4, is imposed for gravity-system components, which are not part of the established seismic force-resisting system, using the deformation demands predicted from response history analysis under MCE_R-level ground motions, as opposed to evaluation under linear analysis.

If an analyst wanted to further investigate the performance of the gravity system (which is not required), the most direct and complete approach (but also the most time-consuming) would be to directly model the gravity system components as part of the structural model and then impose the same acceptance criteria used for the components of the seismic force-resisting system. An alternative approach (which is more common) would be to model the gravity system in a simplified manner and verify that the earthquake-imposed force demands do not control over the other load combinations and/or to verify that the mean gravity system deformations do not exceed the deformation limits for deformation-controlled components.

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