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Response History Analysis for the Design of New Buildings in the NEHRP Provisions and ASCE/SEI 7 Standard: Part II - Structural Analysis Procedures and Acceptance Criteria

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This paper represents the second part of a series of four publications on response history analysis for new buildings. It specifically focuses on modeling assumptions, consideration of important effects in the analysis, and interpretation of analysis results via global and local acceptance criteria. A statistical basis for development of both force- and deformation-controlled acceptance criteria consistent with the collapse probability goals of ASCE/SEI 7 is illustrated. More explicit sub-classifications of force- and deformation-controlled actions are proposed within the statistical framework. Additional philosophical discussion and simple probabilistic arguments are presented on the topic of consideration of unacceptable response, and guidance on addressing unacceptable response is given. Similarities and differences between the new requirements and those in other performance-based design guidelines are also enumerated.

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INTRODUCTION

This paper is the second of four companion papers on the updates to the nonlinear response history analysis procedure for the 2015 NEHRP provisions and eventual consideration in Chapter 16 of the 2016 edition of ASCE/SEI 7, hereafter referred to simply as Chapter 16. The paper first focuses on describing the intent of modeling criteria and analysis requirements. The underlying framework for the global and local acceptance criteria are then explained along with a discussion on interpretation of analysis results and appropriate demand measures. Finally, required components of design review are enumerated.

Nonlinear response history analysis offers several advantages, including the ability to model a wide variety of nonlinear material behaviors, geometric nonlinearities (including large displacement effects), gap opening and contact behavior, and non-classical damping, and to identify the likely spatial and temporal distributions of inelasticity. Nonlinear response history analysis also has several disadvantages, including 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 non-seismic and seismic load effects.

Once limited by computing power, software and consensus on appropriate hysteretic modeling approaches, nonlinear response history analysis has become increasingly used in the design of new and existing structures. Chapter 16 has built upon recommendations in other resource documents and standards, a summary of which appear in Part 1 (Haselton et al. 2015). It is the hope of the authors that Chapter 16 will help to increase the use of nonlinear response history analysis for the design of buildings and other structures in earthquake country.

MODELING AND ANALYSIS

INCLUSION OF GRAVITY SYSTEM

Analytical models constructed to perform linear analysis (e.g. equivalent lateral force or modal response spectrum analysis) typically only include members that compose the selected lateral force-resisting system. This ensures that the selected lateral force-resisting system is capable of providing the code-specified strength and stiffness. However, the gravity system and nonstructural components can add strength and stiffness, sometimes significant enough to change the predicted response of the structure when conducting nonlinear response history analysis. A more thorough discussion and study of the effect of including versus excluding the gravity system when applying the Chapter 16 procedures can be found in Part IV (Jarrett et al 2015). Chapter 16 does not mandate the inclusion of the gravity system. It can however be modeled if desired.

CONSIDERATION OF VERTICAL SEISMIC EFFECTS

Vertical seismic effects are not explicitly required in Chapter 16 for most buildings. This is justified because vertical seismic effects are considered in the Chapter 12 analysis that precedes the Chapter 16 evaluation, thereby ensuring similar margins of safety for vertical seismic effects in nonlinear response history analysis as equivalent lateral force or modal response spectrum analysis. Although it is preferable to include vertical ground motions for all structures, the added complexity is currently only thought to be necessary for structures specifically sensitive to vertical motions (e.g. buildings with long spans, significant vertical discontinuities, or large cantilevers).

TWO-DIMENSIONAL VERSUS THREE-DIMENSIONAL MODELS

Chapter 16 requires the use of three-dimensional models. Although two-dimensional models are valuable for focused studies or preliminary design, a three-dimensional model should be used for final design to account for three-dimensional effects such as multidirectional shaking and torsion.

GRAVITY LOADS

Nonlinear response history analysis is load path dependent with the results depending on combined gravity and lateral load effects. The gravity load case preceding the Maximum Considered Earthquake (MCE_R) shaking should represent a mean estimate of the gravity loading at the time of the MCE_R. The initial application of gravity load is critical to the analysis so member stresses and displacements due to ground shaking are appropriately added to the initially stressed and displaced structure. A single gravity load case of $1.0D + 0.5L$, where L is the reduced design live load, is used in Chapter 16. A proportion of the reduced design live is selected to consider 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_R shaking will occur simultaneously. A single gravity load case is considered sufficient in Chapter 16 because multiple gravity load combinations will already have been enforced in the Chapter 12 analysis step.

P-DELTA EFFECTS

P-Delta effects should be realistically included regardless of the value of the elastic story stability coefficient from Chapter 12. The elastic story stability coefficient is not a reliable indicator of the importance of P-Delta for inelastic deformations. This is especially true for dynamic analyses with large inelastic deformations because significant ratcheting can occur. Ratcheting is characterized by an increase in deformation with cycles of loading that is not recovered or reversed. It typically occurs after global strength begins to deteriorate. Consideration of P-Delta is especially critical in such situations because P-Delta effects further reduce the post-elastic stiffness of the structure and will result in larger residual drifts or an increased number of dynamic instabilities compared to a model neglecting P-Delta effects.

P-Delta effects are captured in nonlinear models through application of gravity loads on (a) the explicitly modeled elements and (b) leaning columns. Leaning columns are columns which have no lateral resistance and are placed in a model to capture the gravity loads acting on elements not explicitly modeled (e.g. the gravity system unless it has been included). While a single leaning column at the center of gravity of the building has been used in the past, this practice should be discouraged. Instead, either several leaning-columns arranged to reflect the distribution of gravity loads or direct modeling of the gravity system should be undertaken. This ensures that the P-Delta contribution to torsional effects will be captured.

DIAPHRAGM MODELING

Modeling of diaphragms can increase the runtime of analysis models significantly, particularly, although uncommonly, if diaphragm inelasticity is included. Therefore, diaphragms are typically only explicitly modeled at locations where force transfer is likely to occur and idealized diaphragm flexibility assumptions (e.g. rigid or flexible) are used elsewhere. When explicitly modeled, diaphragms are typically composed of a finite element mesh to capture the diaphragm’s material properties, configuration, thickness and openings. The stiffness of the diaphragm relative to the lateral force resisting system can have a significant effect on building response where discontinuities in lateral resistance occur. Stiffness modifiers on the gross-section stiffness properties are often used to approximate the effective stiffness of a diaphragm where cracking or other softening mechanisms are expected to occur. Due to the complex and uncertain behavior of a diaphragm's effective stiffness, it may be appropriate to analyze the structure using upper-bound and lower-bound effective stiffness assumptions to envelope the true behavior.

TORSION

Inherent torsion is actual torsion caused by differences in the location of the structure's center of mass and center of rigidity at each level of the building, wherein the structure's center of rigidity changes in time as differential yielding occurs in the lateral force-resisting system. Inherent torsion is automatically included in the three-dimensional analytical model through appropriate distribution of the mass and assignment of the element properties.

Accidental torsion, on the other hand, is intended to account for uncertainties in mass, stiffness and strength distribution, and for torsional components of ground motion. It is often approximated by shifting the center of mass a fixed distance in each direction from its expected location. Chapter 16 does not require the application of accidental torsion in nonlinear response history analysis. It instead ensures an equivalent margin of safety to the equivalent lateral force and modal response spectrum procedures through the Chapter 12 torsion provisions. Although this ensures consistency with other code-conforming buildings, Part 4 and other research (Jordan et al. 2015, Jordan et al. 2014) discuss the potential consequences of not including accidental torsion in nonlinear response history analysis.

STIFFNESS OF ELEMENTS MODELED WITH ELASTIC PROPERTIES

It is not currently computationally practical to explicitly model all phenomena that affect element stiffness in a nonlinear analysis model, especially when such behaviors can be more simply represented without significant loss of accuracy. It is important to remember that the model is intending to capture response of the entire structure and can therefore not be as detailed as a model used solely for representing the response of tested subassemblies or individual structural members. As such, many element actions are treated as linear with an effective stiffness used in place of a low-strain stiffness. Chapter 16 requires that specific actions such as cracking in concrete and panel zone deformation in steel be represented, most likely through the use of stiffness modifiers on an elastic element. If, however, extensive nonlinearity is expected in an element, an explicit representation of the nonlinearity should be pursued as described in the next section.

NONLINEAR ELEMENT MODELS

Element actions for which inelastic deformation is expected are required by Chapter 16 to be explicitly modeled. In contrast to other standards such as ASCE 41, however, Chapter 16 does not prescribe or supply the modeling parameters. Instead it emphasizes the important phenomena to consider and leaves the detailed model development to the designer. Models typically account for, at a minimum, monotonic response (failure deformation and post-failure behavior) and cyclic strength and stiffness deterioration. The loading and unloading behavior of the element (e.g. pinching) is also important in some cases but is generally less so than capturing the monotonic behavior and deterioration due to cyclic loading.

Cyclic deterioration can be represented using one of several methods. Ideally the mathematical model will explicitly account for cyclic deterioration at the component level, and modify the hysteretic response based on the deformation history. Alternatively, the cyclic deterioration can be incorporated into the force-displacement "backbone" curve (either by factoring the monotonic load-displacement curve, or by using a cyclic envelope curve) with deterioration implicitly, rather than explicitly, considered in the model. Finally, deterioration can be conservatively accounted for by modeling a full loss of component capacity at a deformation level consistent with the onset of cyclic deterioration.

Modeling of nonlinear effects at large deformation levels can add considerable complexity to a model. Simplifying a model by not incorporating phenomena that occurs at large deformations is generally acceptable, provided that it can be demonstrated that the element deformation does not exceed the deformation at which the effect becomes important. For example, it would not be necessary to model strength loss if no element in the model had reached a deformation at which strength loss began. This check would need to be performed for every ground motion, however.

DAMPING

Energy dissipation not associated with modeled inelastic behavior is generally represented through the use of viscous damping. This "inherent" damping represents material damping in portions of elements that remain elastic, friction in connections and steel-concrete interfaces, friction in nonstructural and architectural components, and many other sources. Generally, this damping is represented by combined mass and stiffness proportional damping. To ensure that the viscous damping does not exceed the target level in the primary modes of response, the damping is typically set at the target level for two periods (one below and one above) the fundamental period considering also that the period will elongate due to nonlinear behavior. This will ensure damping at or below the target level in the primary modes. For very tall buildings the higher modes can have significant contributions to response, and the damping should be set such that these higher mode (shorter period) responses are not inadvertently over-damped. Viscous damping may alternatively be represented by modal damping, which allows for the explicit specification of the target damping in each mode. Care must be taken in specification of damping in nonlinear response history analysis. The reader is referred to several papers (Charney 2008, Hall 2006, Priestly and Grant 2005, Zareian and Medina 2010) for further discussion on this topic.

The level of inherent damping can range from 1.0 to 5.0 percent critical, depending on the nature of the structural and nonstructural systems. Chapter 16 limits inherent viscous damping to 3.0 percent of critical. Energy dissipation due to the incorporation of supplemental damping devices (e.g viscous fluid dampers, viscoelastic solid dampers, etc.) should be explicitly accounted for with component-level models and not included in the inherent viscous damping term.

INTERPRETATION OF STRUCTURAL RESPONSE PREDICTIONS

TREATMENT OF RESPONSE VARIABILITY

Eleven ground motions are selected and modified using an average-spectrum-based procedure as described more fully in Part 1 (Haselton et al. 2015). This overall approach ensures that the mean structural response is captured. The variability in response between ground motions and response under individual motions are not explicitly controlled in selecting and modifying ground motions to form a suite. These measures are therefore not relied upon in Chapter 16 (except for the case of unacceptable response as discussed later).

It is also often desirable to predict the variability in structural response (e.g., the standard deviation, σ) to judge margins against undesirable performance. For example, the PEER TBI guidelines (PEER 2009) use the variability in the element force demands to help provide a greater level of conservatism in the design of some force-controlled elements. Even though predicting the variability in structural response is desirable, it is also difficult to accomplish in any statistically meaningful manner without the use of more than eleven ground motions.

Of the target spectrum approaches available for use in Chapter 16, only the conditional spectrum method provides a mechanism to consider structural response variability in selection of ground motions. However, a suite of ground motions on the order of thirty, rather than eleven, would be required to provide a meaningful estimate of response variability. Since the conditional spectrum approach and an increased number of ground motions are not required by Chapter 16, variability in response is not used to determine building conformance (except for the case of unacceptable response).

The acceptance criteria checks in Chapter 16 therefore almost exclusively use only the mean responses. Yet, the reality of response variability, as well as other design uncertainties, has been taken into account when developing the acceptance criteria.

TREATMENT OF UNACCEPTABLE RESPONSE

Chapter 16 defines an “unacceptable response” under any single ground motion as dynamic instability, collapse, non-convergence, response significantly exceeding the valid range of modeling, or force demand that exceeds the mean strength of a critical force-controlled component. As previously mentioned, Chapter 16 prescribes a mean target spectrum without a defined variability in spectral values. When using this procedure, the observance or non-observance of an unacceptable response will depend on how the ground motions were selected and modified (e.g. scaled or matched) to meet the target spectrum as well as the detail of modeling employed. This difficulty in the reliable prediction of unacceptable response cases leaves an open question for how to interpret the meaning of one or more unacceptable responses in a suite of analyses.

This question becomes how to treat unacceptable responses in the acceptance criteria in Chapter 16. Many other standards and guidelines are silent on this issue (e.g. ASCE/SEI 7-10, ASCE/SEI 41-13). Some engineers presume that acceptance criteria requirements, which are often checked using average response, effectively disallow unacceptable responses (because you cannot average an undefined or infinite response). Others presume that average could also be interpreted as median, thus allowing nearly half of ground motions to produce unacceptable responses.

The treatment of unacceptable response quickly becomes a philosophical debate, with two clear “camps” of thought. The rationale of Philosophical Camp #1 is as follows:

- The responses of a few outliers are not statistically meaningful and should not be used in the acceptance criteria.
- Acceptance criteria should be based on median values because this is the only stable predictor of response.
- As an example, if 5 out of 11 ground motions cause an unacceptable response, it is not statistically meaningful. Chapter 16 should therefore not prohibit such a case.

The rationale of Philosophical Camp #2 is as follows:

- Even though the responses of a few outliers are not statistically meaningful (this assertion is typically undisputed), a conscientious engineer will be concerned by what outliers indicate about the structure. For example, does an outlier suggest a potential weakness in the structural design or is its response spectrum just much larger than the target?
- Acceptance criteria should include a check on unacceptable responses because such responses potentially indicate that the collapse safety goal is not being met.
- As an example, if 5 out of 11 ground motions cause an unacceptable response (it is acknowledged that this does not imply a collapse probability of 5/11), it strongly suggests a collapse probability greater than 10% and is cause for concern. Chapter 16 should therefore prohibit such a case.

To help bring the two philosophical camps into agreement, some simple statistical studies were pursued to better understand the meaning of observing one or more unacceptable responses in a suite of analyses. See Appendix A for greater detail. From this work, the following conclusions can be made:

- A 10% collapse probability goal is not necessarily met even if zero unacceptable responses are observed in a set of eleven analyses. Therefore other acceptance criteria (e.g. criteria for drifts and element demands) must be relied upon to achieve this goal.
- Even if a building has a 10% probability of collapse, there is some chance that one unacceptable response will be observed in a set of eleven analyses (i.e. a "false positive"). Therefore an acceptance criterion of "no unacceptable responses" would be violated quite often by a building that meets the 10% collapse probability goal.
- If the 10% collapse probability goal is met, it is highly unlikely that two or more unacceptable responses will be observed in a set of eleven analyses.

These observations are for scaled (non-matched) ground motions. Based on the statistics presented in Appendix A, the reduced ground motion variability when spectral matching is used indicates that even one unacceptable response is highly unlikely for a building meeting the 10% collapse probability goal. Similarly, it is highly unlikely for a building having a 6% or 3% probability of collapse (i.e. Risk Category III or IV, respectively) to experience even one unacceptable response under either scaled or spectrally matched ground motions.

ACCEPTANCE CRITERIA

Acceptance criteria are intended to provide confidence that the structure's response is stable and within a range predictable by analysis and substantiated by testing. While not a guarantee of acceptable collapse performance, they do provide confidence that this is the case. Some of the acceptance criteria have, however, been specifically developed based on the collapse performance targets (e.g. criteria for strength of brittle components) while others are more historically-based (e.g. story drift criteria). Part 1 (Haselton et al. 2015) includes a literature review comparing acceptance criteria in other recent standards and guidelines with those in Chapter 16. Two categories of acceptance criteria are included in Chapter 16: global and local.

GLOBAL ACCEPTANCE CRITERIA

Average Story Drifts

The limits on average story drift are developed for consistency with the equivalent lateral force and modal response spectrum procedures in ASCE 7. When nonlinear response history analysis is performed, ASCE 7-10 Table 12.12-1 story drift limits are:

- Increased by 1.5 to reflect that the analysis is being completed at the MCE_R ground motion level rather than at the design level (i.e. 2/3 of the MCE_R).
- Increased by 1.25 to reflect an average ratio of R/Cd for conforming lateral force-resisting systems.
- Increased by another small factor on the order of 5% to reflect the greater confidence in drift prediction using response history analysis.

These three effects result in an average story drift limit twice those found in ASCE 7-10 Table 12.12-1. The justification for the increase for R/Cd has been substantiated by prior research (Uang and Maarouf 1994) and is already included in ASCE 7-10 for members spanning between structures (i.e. ASCE 7-10 Section 12.12.4). For systems which possess both R/Cd ratios that exceed the average and are drift rather than strength controlled (e.g. steel special moment-resisting frames), the Chapter 16 drift limits may actually be more restrictive than those in Chapter 12. See the Part III paper (Zimmerman et al. 2015) for an example of such a case.

Maximum Story Drifts

The PEER TBI guidelines (PEER 2009) limit the maximum story drift from any one analysis in the full suite. Such a requirement is not included in Chapter 16 for two reasons: (1) some reasonable limits are already imposed for unacceptable responses and (2) the maximum story drift is both difficult to predict reliably and will depend heavily on the details of the ground motion set.

Residual Story Drifts

The PEER TBI guidelines (PEER 2009) limit the average residual story drift. Such a requirement is not included in Chapter 16 because a residual drift acceptance criterion is not needed for enforcing the collapse safety goals. 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 ASCE 7. For Risk Category I and II buildings, ASCE 7 is primarily meant to ensure the protection of life safety. Additionally, residual drifts are more difficult to reliably predict than peak drifts with available structural analysis tools and standard modeling approaches. Part 3 (Jarrett et al. 2015) explicitly evaluates residual drift within the framework of Chapter 16 using several case studies.

Loss in Story Strength

A limit on building (or story) strength loss was also considered, similar to the 20% maximum story strength loss required in the PEER TBI guidelines (PEER 2009). If one assumes that the structural model is able to appropriately simulate the effects of component strength loss (as is required by the limitations imposed on unacceptable responses), then a limit on building strength loss is theoretically unnecessary. In the development of the above-mentioned PEER TBI requirements, this assumption was not made. A requirement for limiting building strength loss is also not pursued in Chapter 16 because (1) story strength loss could not be simply defined or determined, and (2) for most structural systems, drift limits will likely limit the degree of strength loss.

Unacceptable Responses

The primary acceptance criteria are the story drift and element-level criteria. However for Risk Category I and II structures when ground motion scaling (not spectral matching) is employed, one unacceptable response in a suite of 11 motions is acceptable. For all other cases (e.g. spectral matching, higher Risk Category structures), no unacceptable responses are permitted. While this criterion does not ensure that the collapse safety goal has been met, it is intended to screen out designs that are likely not to meet the collapse safety goals.

When an unacceptable response occurs, it is not possible to compute a mean value of the building responses for use with other acceptance criteria because one of the eleven response quantities is undefined. In this case, rather than mean response estimates, design values are taken as the counted median response multiplied by 1.2, but not less than the mean response from the remaining ten motions. To compute the median value, the unacceptable response should be assumed as larger than the other responses and then the counted median value should be taken to be the 6th largest response from the set of eleven.

The factor of 1.2 was developed based on an approximation of the typical ratio of mean to median values for a lognormal distribution (β = 0.4 results in mean/median = 1.08, β = 0.5 results in mean/median = 1.13, β = 0.6 results in mean/median = 1.20, and β = 0.7 results in mean/median = 1.28). The requirement to check the mean of the remaining ten response results is simply an added safeguard to ensure that 1.2*median is not an under-prediction.

ELEMENT-LEVEL ACCEPTANCE CRITERIA

Element-level acceptance criteria are dependent on a two-tier classification system. The first tier is classifying each action as either force- or deformation-controlled. Note that this is done for each element action, rather than for each element. For the example of a reinforced concrete column, the flexural behavior may be classified as a deformation-controlled action whereas the shear behavior may be classified as a force-controlled action.

Deformation-controlled actions are those that have reliable inelastic deformation capacity without substantial strength decay, whereas force-controlled actions are associated with brittle modes where inelastic deformation capacity cannot be assured. Based on the structure of the acceptance criteria, any element action that is modeled elastically must be classified as force-controlled.

Examples of force-controlled actions include:

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

Examples of deformation-controlled actions include:

- Shear in diagonally reinforced coupling beams
- Flexure in reinforced concrete columns and walls
- Axial yielding in buckling restrained braces

- Flexure in moment frames

The second tier of classification is to take the force- and deformation-controlled actions separately and further distinguish each action based on its consequence of failure (with failure defined as the action exceeding its strength or deformation limit). The consequence classifications are critical, ordinary, and non-critical and are defined as:

- *Critical element actions* - Those in which failure would result in the collapse of multiple bays of multiple stories of the building or would result in a significant reduction of the seismic resistance of the structure.
- *Ordinary element actions* - Those in which failure would result in only local collapse, comprising not more than one bay in a single story, and would not result in a significant reduction of the seismic resistance of the structure.
- *Non-critical element actions* - Those in which failure would not result in either collapse or substantive loss of the seismic resistance of the structure.

Acceptance criteria for both force- and deformation-controlled actions are developed separately for each of the consequence classifications. This section describes the underlying assumptions that produce the acceptance criteria in Chapter 16.

Acceptance Criteria for Force-Controlled Actions

The proposed acceptance criteria for force-controlled actions follow the load and resistance factor framework presented in the PEER TBI guidelines (PEER 2009):

$$\lambda F_u \leq \phi F_{n,e} \tag{1}$$

The parameter λ is a calibration parameter explained in this section, Fu is the mean demand for the response parameter of interest, ϕ is the strength reduction factor from a material standard, and $F_{n,e}$ is the nominal strength computed from a material standard using expected material properties. In the PEER TBI guidelines, the value of λ is 1.5 for general critical responses, 1.2 for critical response parameters limited by a well-defined mechanism, and 1.0 for non-critical response parameters. These values of λ were arbitrarily selected with the intention of providing a low, but undefined, probability of collapse considering response variability and other uncertainties. The PEER TBI effort performed limited reliability studies to benchmark these values against the collapse goals for tall shear wall buildings but did not evaluate their adequacy for other classes of structures. During the development of Chapter 16, further studies were undertaken to generalize these values for broader application.

To determine appropriate benchmark values of λ for use in Chapter 16, the collapse probability goals for Risk Category I and II are used. Those for Risk Category III and IV are incorporated later. The collapse probability goals for Risk Category I and II are defined in ASCE/SEI 7-10 by 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, it is assumed that failure of a critical force-controlled component would result in total or partial structural collapse of the building. To state this in more mathematical terms, it is assumed that the probability of structural collapse, conditioned on failure of a critical force-controlled component, is 100%. It is further assumed that the probability of failure of the element is 100% when the acceptance criteria are exceeded. Both assumptions are conservative.

Force demand and component capacity are taken as lognormally distributed random variables while $F_{n,e}$ is assumed to represent the true expected component strength. Figure 1 shows example lognormal distributions of the component capacity and component demand. The benchmark λ/ϕ value is calibrated by convolving the distributions of demand and capacity, consistent with load and resistance factor design, until the 10% collapse probability goal is achieved.

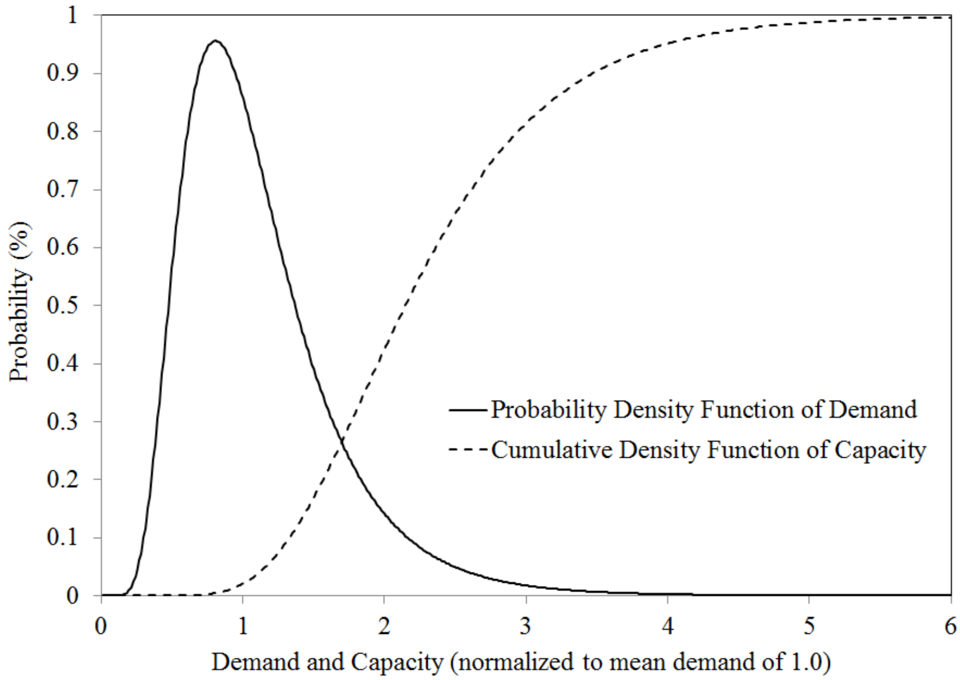

Figure 1. Illustration of lognormal distributions for demand and capacity (normalized to a mean demand of 1.0). The mean capacity is calibrated to achieve P[C|MCE$_R$] = 10%.

This calibration process is highly dependent on the uncertainties in both demand and capacity of the component. Table 1 shows the variability and uncertainty values assumed for force demand for analyses at the MCE$_R$ hazard level in developing the Chapter 16 force-controlled acceptance criteria. Both the general case and the case where the response parameter is limited by a well-defined yield mechanism are provided. Table 2 shows variability and uncertainty values assumed for component capacity. The values shown in Table 1 and 2 are based on Biskinis et al. (2004), Haselton et al. (2008) and Wallace et al. (2013) as well as the collective experience and professional judgment of those involved in developing Chapter 16.

Table 1. Assumed Variability and Uncertainty Values for Force-Controlled Component Demand.

Demand Dispersion (β_D)		**Variabilities and Uncertainties in the Force Demand**
General	**Well-Defined Mechanism**	

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 2. Assumed Variability and Uncertainty Values for Force-Controlled Component Capacity.

Capacity Dispersion (β_C)		**Variabilities and Uncertainties in the Final As-Built Capacity of the Component**
General	**Well-Defined Mechanism**	
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 data)
0.20	0.20	Uncertainty in as-built strength due to construction quality and potential errors
0.37	**0.37**	$\beta_{C\text{-}TOTAL}$

Table 3 presents the results of the calibration process for critical force-controlled actions as the “required ratio of λ/ϕ”. Since it was determined that the required ratio of λ/ϕ for Risk Category I and II was 2.1 for the general case and 1.9 for the case of a well-defined mechanism, no dependence on the existence of a well-defined mechanism is introduced in the Chapter 16 force-controlled acceptance criteria. Calculations for Risk Category III and IV were similar to those presented previously with the collapse probability goal modified to 6% and 3%, respectively. Critical force-controlled actions in Chapter 16 are thus required to satisfy:

$$2.0 I_e F_u \leq F_e \qquad (2)$$

The parameter I_e is the importance factor (equal to 1.00, 1.00, 1.25 and 1.50 for Risk Categories I, II, III and IV, respectively), F_u is the mean demand for the response parameter of interest, and F_e is the mean expected strength of the component taken as $F_{n,e}$ adjusted for when test data indicates conservatism or unconservatism in $F_{n,e}$. The value $2.0I_e$ is also shown in Table 3 for comparison against that required by the calibration process. It shows reasonable agreement with the theoretical value. Note that Equation 2 is equivalent to the PEER TBI acceptance criterion for critical force-controlled actions (PEER 2009 Equation 8-2) for the case that ϕ = 0.75 and $F_e = 1.0F_{n,e}$.

Table 3. Results of calibration process for critical force-controlled actions

Risk Category	Collapse Probability Goal	Required Ratio of λ / ϕ	Chapter 16
I and II	10%	2.1	2.0
III	6%	2.4	2.5
IV	3%	3.0	3.0

As described previously, force-controlled actions are sub-classified by their failure consequence. To develop the acceptance criteria for ordinary (as opposed to critical) force-controlled components, the calibration process was re-performed using a 25% chance of a failure that could result in endangerment of individual lives. This represents the second performance objective target specified in ASCE/SEI 7-10 for Risk Category I and II. Based on these calculations, ordinary force-controlled actions in Chapter 16 are required to satisfy:

$$1.5 I_e F_u \leq F_e \tag{3}$$

Finally, for non-critical force-controlled actions, statistics is abandoned and the acceptance criterion in Chapter 16 is adopted (consistent with the PEER TBI approach) as:

$$1.0 I_e F_u \leq F_e \tag{4}$$

Acceptance Criteria for Deformation-Controlled Actions

Development of acceptance criteria for deformation-controlled actions follows a similar process as that for critical and ordinary force-controlled actions, namely a calibration process to achieve an intended probability considering variability and uncertainty in both demand and capacity. See Figure 1.

Table 4 shows the assumed uncertainties in deformation demand for structural analyses under MCE_R ground motions. Table 5 shows the assumed uncertainties for component deformation capacity at the point where loss in vertical-load-carrying-capacity (LVCC) occurs. Note that the capacity for deformation-controlled elements is actually specified by the deformation at which it can no longer carry vertical (gravity) loads rather than by a specified amount of lateral strength loss. The values shown in Table 4 and 5 are based on Fardis and Biskinis (2003) and Haselton et al. (2008) as well as the collective experience and professional judgment of those involved in developing Chapter 16. The values of β_C for deformation-controlled actions are greater than for force-controlled actions because of increased uncertainty in quantifying the deformation at which LVCC occurs.

Table 4. Assumed Variability and Uncertainty Values for Deformation-Controlled Component Demand

Demand Dispersion (β_D)	Variabilities and Uncertainties in the Deformation Demand
0.40	Record-to-record variability (for MCE_R ground motions)
0.20	Uncertainty from estimating deformation demands using structural model
0.13	Variability from estimating deformation demands from mean of only 11 ground motions
0.46	$\beta_{D\text{-}TOTAL}$

Table 5. Assumed Variability and Uncertainty Values for Deformation-Controlled Component Capacity

Capacity Dispersion (β_C)	Variabilities and Uncertainties in the Final As-Built Deformation Capacity of the Component
0.60	Typical variability in prediction equation for deformation capacity (from available data)
0.20	Typical uncertainty in prediction equation for def. capacity (extrapolation beyond data)
0.20	Uncertainty in as-built deformation capacity due to construction quality and errors
0.66	$\beta_{C\text{-}TOTAL}$

Calibration of the risk equations shows that mean deformation capacity of critical components must be 3.2x greater than the mean MCE_R deformation demand to meet the 10% collapse safety objective for Risk Categories I and II. Inverting this value and rounding slightly leads to the requirement that the mean deformation demand be less than 0.3 of the mean deformation capacity. The calibration was also redone for Risk Categories III and IV, using the 6% and 3% collapse probability thresholds, respectively. This resulted in required ratios of 0.25 and 0.19, respectively, and is accounted for by adding $1/I_e$ to the requirement, which is similar to the I_e approach used for force-controlled components.

For this assessment it was assumed, similar to force-controlled components, that total or partial collapse takes place when any single component reaches the deformation when LVCC occurs. In mathematical terms, this can be read as a 100% conditional probability of failure given that the deformation capacity is exceeded. These assumptions can be realistic under uncommon conditions; however, in most cases, at least one alternative load path for gravity forces exist. Where an alternative load path for gravity forces can be identified, and under the assumption that a 40% (rather than a 100%) conditional probability is more appropriate for this condition, the required ratio becomes ($0.5/I_e$) instead of ($0.3/I_e$). The 40% conditional probability was based on professional judgment.

The above requirements were developed for critical deformation-controlled components. Similar to requirements for ordinary force-controlled components, requirements for ordinary deformation-controlled components were developed by re-performing the calculations for a 25% chance of a failure. The resulting requirements are shown alongside those for critical deformation-controlled components in Table 6.

For non-critical deformation-controlled actions, the failure of such component would neither result in collapse nor result in substantive loss in the seismic strength of the structure. Accordingly for such cases, the inelastic deformation is not limited by the local acceptance criterion in Chapter 16 (because there is no meaningful consequence of failure for such components). The inelastic deformation for such components is instead limited by the requirement that response be within the valid range of modeling.

Table 6. Acceptance Criteria Limits for Deformation-Controlled Components

Classification	Limit When Data Available	Limit Based on ASCE 41 (when data not available)
Critical, no redistribution	$(0.3/I_e)\theta_{LVCC}$	$(0.5/I_e)\theta_{ASCE41}$
Critical, with redistribution	$(0.5/I_e)\theta_{LVCC}$	$(0.75/I_e)\theta_{ASCE41}$
Ordinary, no redistribution	$(0.5/I_e)\theta_{LVCC}$	$(0.75/I_e)\theta_{ASCE41}$
Ordinary, with redistribution	$(0.7/I_e)\theta_{LVCC}$	$(1.0/I_e)\theta_{ASCE41}$
Non-critical	No Limit	No Limit

I_e is the importance factor, θ_{LVCC} is the inelastic deformation at loss of vertical load-carrying capacity, and θ_{ASCE41} is the ASCE 41-13 collapse prevention acceptance criterion

Use of ASCE41 Acceptance Criteria When Test Data are Unavailable

In order to use the acceptance criteria for deformation controlled components, it is necessary to have laboratory data on the deformations at which a component can be expected to lose its gravity load-carrying capacity. In reality, such data is seldom available as most laboratory testing of structural components is terminated before complete strength loss occurs. There are a number of reasons for this including: (1) conventional wisdom considered the end of a component's useful range of response to be at a point where strength degraded to 80% of the peak value, thus many tests were terminated once degradation reached this value or a somewhat larger value; (2) testing components to complete failure may require imposed deformation in excess of the testing apparatus capacity; (3) testing to complete failure can be dangerous and destructive to the laboratory equipment and sensors. Since this data is not commonly available, an option is provided in Chapter 16 to use acceptance criteria listed in ASCE 41 (ASCE 2013).

To determine the appropriate manner in which to use the ASCE 41 tabular acceptance criteria (ASCE 2013), or acceptance criteria derived using the experimental procedures of ASCE 41, the statistical basis for the acceptance criteria in ASCE 41 must first be understood. Note that in the 2013 edition of ASCE 41, the distinction between primary and secondary acceptance criteria for nonlinear modeling was removed. Section 7.6 of ASCE 41-13 defines how the collapse prevention acceptance criteria is based on the mean deformation at which gravity-carrying capacity is lost. Although this is the intention of the collapse prevention acceptance criteria, many of the material chapters did not strictly follow this definition in establishing a limit. For example, the acceptance criteria for conforming reinforced concrete columns are conservatively based on the 15th percentile value of test data for non-conforming columns (Elwood et. al. 2007) while those for structural steel components are based on a deformation at substantial strength loss (conservative estimate of LVCC occurs). Overall, the statistical basis is not consistent for the various tabular Collapse Prevention acceptance criteria values in ASCE 41.

If the statistical basis of the ASCE 41 Collapse Prevention acceptance criterion is known for a specific type of structural component, one could then determine how to appropriately translate the ASCE 41 tabular values into Chapter 16. In the absence of such information, it can be assumed that the tabular ASCE 41 Collapse Prevention values represent 2/3 of the true mean values at which LVCC occurs. Based on this assumption, the Table 6 acceptance criteria were developed (right-most column) by replacing θ_{LVCC} with $1.5\theta_{ASCE41}$ and rounding.

TREATMENT OF THE GRAVITY SYSTEM

Chapter 16 requires that the deformation-compatibility requirement of ASCE 7-10 Section 12.12.5 be satisfied for components that are not part of the established seismic force-resisting system. However the deformation demands are taken from the nonlinear response history analyses under MCE_R ground motions rather than from the equivalent lateral force or modal response spectrum analyses under 2/3 of MCE_R. Elements of the gravity system which are explicitly modeled as either force- or deformation-controlled actions may be assessed using the respective acceptance criteria for those actions.

DESIGN REVIEW

Every building code, standard, or guideline document that has permitted the use of nonlinear response history analysis has required independent peer review as part of the process. The primary reason for this is that the results of such analyses are highly dependent on the modeling assumptions and ground motions used. Seemingly minor differences in the way damping is included, hysteretic characteristics are modeled, or ground motions are scaled can result in substantially different predictions of response. Another important reason to require independent review is that the provisions governing nonlinear response history analysis are generally non-prescriptive in nature and require significant judgment on the part of the engineer performing the work. Finally, many building departments lack the sophistication necessary to perform appropriate reviews of calculations employing nonlinear analysis. When properly performed, independent peer review can assist the engineer of record in identifying and resolving design issues they might otherwise overlook, while providing additional assurance that the design will meet the selected performance objectives. For these reasons, Chapter 16 also requires such review, termed Design Review.

Chapter 16 requires that Design Review be performed by a person or persons who have knowledge of: the requirements of the Standard; selection and scaling of ground motions; nonlinear analytical modeling, including the use of laboratory test data to develop modeling parameters and acceptance criteria; and the behavior of structural systems of the type being designed when subjected to strong ground motions.

The scope of Design Review includes:

1. Project-specific statements of performance objectives.
2. Any exceptions to the prescriptive criteria that will be incorporated in the design.
3. A description of analytical methods, modeling assumptions, software to be used, element hysteretic properties, damping and acceptance criteria.

4. The project geotechnical investigation report including soil shear strength, stiffness, and damping characteristics; recommended foundation types and design parameters; seismic hazard evaluation, including both design spectra and selection and scaling of ground motions
5. Actual analytical modeling.
6. The building’s overall dynamic behavior including natural frequencies, mode shapes and mass participation factors.
7. Key structural response parameter results
8. General conformance of the analytical model to the structure as portrayed in the construction documents.
9. Detailing of critical structural elements

Though not required, it is anticipated that the first four items noted above would be submitted early in the review process to enable the reviewer to identify potential issues before the designer has expended substantial effort and assist with early resolution of these concerns. This also helps to avoid scenarios wherein a designer may select a suite of ground motions, find that one or more of these produce unacceptable response, then select and analyze additional motions but present data only for those motions for which acceptable response is obtained.

Review of the drawings and detailing is deemed important to assure that the analytical assumptions appropriately reflect the actual structure and also that elements with anticipated large strength or nonlinear deformation demands are adequately designed to resist these demands.

Upon completion of Design Review, the reviewer(s) must provide the authority having jurisdiction a letter of completion documenting: the scope of review performed; concurrence with the analysis and its applicability to the design; agreement with any exceptions to the requirements of the Standard; and any unresolved concerns with the design or analysis. This latter requirement is included in recognition that the engineer of record and design reviewer may not always reach concurrence on all issues raised. In such cases, the authority having jurisdiction must resolve the disagreement, either by making an independent judgment or seeking the advice of additional reviewers.

CONCLUSION AND TOPICS FOR FUTURE STUDY

This paper has described the provisions relating to structural analysis, acceptance criteria and design review of Chapter 16 of the 2016 NEHRP Provisions which are to be incorporated, with changes, into ASCE/SEI 7-2016. Significant progress has been made in developing acceptance criteria and unacceptable response provisions consistent with the collapse safety goal of ASCE/SEI 7. At the same time, there remains room for additional study in these areas, hopefully aided by the statistical foundation set in this paper. Topics for future research include:

- *Minimum base shear requirements* – The minimum base shear requirements control the design of many tall buildings and are based on historic precedent with limited verification. Future study would be useful to further investigate the minimum base shear requirements and how they relate to the collapse safety goals of ASCE/SEI 7 for various structural systems. Such study could also utilize recent earthquake data to revisit the near-source basis of ASCE/SEI 7-10 Equation 12.8-6.
- *Conformance to collapse safety goals* – While substantial progress has been made in Chapter 16 to link the acceptance criteria more directly to the collapse safety goals of ASCE/SEI 7, further development would be beneficial. More specifically, additional research could refine the calibration of the Chapter 16 implicit satisfaction of the collapse safety goals with more explicit methods. Additionally, drift acceptance criteria in ASCE/SEI 7 have remained unchanged for many years and were simply adjusted for use in Chapter 16. Both topics would benefit from future study.
- *Structure of acceptance criteria* – The acceptance criteria of Chapter 16 are developed by individually calibrating each acceptance criterion to the collapse safety goals. Since collapse generally involves multiple components simultaneously, a future effort could look at how the collapse probability of a building is affected by the interaction between multiple individual element acceptance criteria. As part of this work, consideration

should be given to grouping of similar elements (e.g. due to symmetry), a requirement not explicitly included in Chapter 16.

- *Consequences of failure* – When developing the acceptance criteria for force- and deformation-controlled actions, assumptions were made as to the probability of total or partial collapse conditioned on the exceedance of a single component (e.g. 100% for critical force-controlled actions, 40% for critical deformation-controlled actions with an alternate load path). Future work could study in greater depth the consequences of failure and potentially refine the provisions. The topic of this study would overlap with that presented under "structure of acceptance criteria."
- *Dependence on strength reduction factor* – The acceptance criteria for force-controlled components in Chapter 16 are structured in such a way that the final criterion is independent of the strength reduction factor (ϕ). Further refinement may indicate a preference for making the acceptance criteria dependent on the value of ϕ.
- *Compatibility between ASCE 7 and ASCE 41 requirements* – There is currently a lack of consistency between Chapter 16 acceptance criteria for force-controlled components, deformation-controlled components, and consideration of unacceptable response, and the corresponding criteria in ASCE 41-13. For example, Chapter 16 recommends the use of 0.5x the collapse prevention acceptance criteria from ASCE 41 in cases where ASCE 41 permits 1.0x. In another example, ASCE 41's force-controlled acceptance criteria checks are based on mean, unamplified demands from the response history analysis whereas Chapter 16 recommends an amplification dependent on criticality. The acceptance criteria in both documents should be reviewed with the goal of creating consistency or, at the very least, providing commentary where consistency is lacking.

With the advances in technical understanding and computation power in recent years, response history analysis has become a viable tool for more accurate prediction of expected building seismic performance. Part I and this paper (Part II) have described the response history analysis provisions and documented the underlying framework used in developing them. Later papers in this series (Zimmerman et al. 2015, Jarrett et al. 2015) go on to implement the provisions and further study assumptions made in the development process.

Haselton et al. (2017). “Response-History Analysis for the Design of New Buildings in the NEHRP Provisions and ASCE/SEI 7 Standard: Part II – Structural Analysis Procedures and Acceptance Criteria.” Earthquake Spectra, 33(2), 397–417. https://doi.org/10.1193/020416EQS028M

Haselton et al. (2017). "Response-History Analysis for the Design of New Buildings in the NEHRP Provisions and ASCE/SEI 7 Standard: Part II – Structural Analysis Procedures and Acceptance Criteria." Earthquake Spectra, 33(2), 397–417. https://doi.org/10.1193/020416EQS028M

APPENDIX A: SIMPLE COLLAPSE STATISTICS - THE MEANING OF OBSERVING UNNACEPTABLE RESPONSES AND THE LIKLIHOOD OF FALSE-POSITIVES

The purpose of this appendix is to present some simple statistics to help better interpret the meaning of a collapse or other type of unacceptable response. The goal is that this will provide a clearer basis for making decisions regarding treatment of unacceptable responses in the acceptance criteria.

The simple statistics of this appendix are based on predicting the occurrence of collapse (or other unacceptable response) based on the assumptions listed below. To keep the terminology of this appendix concise, all unacceptable responses will be lumped together and referred to as "collapse" in this appendix.

- The likelihood of collapse is predicted using the binomial distribution.
- The collapse probability of the building is 10% at the MCE_R level (i.e. $P[C|MCE_R] = 10\%$), which is for Risk Category I & II structures.
- The total uncertainty in collapse capacity, in terms of the lognormal standard deviation $\beta_{COL,TOT}$, is 0.6. This value includes all sources of uncertainty and variability (record-to-record variability, modeling uncertainty, etc.). The value of 0.6 is the same value used in creating the risk-consistent hazard maps for ASCE 7-10 (ASCE 2010) and is consistent with the values used in FEMA P695 (FEMA 2009).
- The record-to-record variability, $\beta_{COL,RTR}$, ranges from 0.25 to 0.40. This is the variability in the collapse capacity that would be expected from the analytical model. This $\beta_{COL,RTR}$ value is highly dependent on the details of the ground motion selection and scaling; values of 0.35-0.45 are expected for motions that are not fit tightly to the target spectrum and values of 0.2-0.3 are expected for spectrally matched motions (FEMA 2009, Haselton and Deierlein 2008).

Based on the above assumptions, Figure A1 shows the collapse fragility curves for a building that meets the $P[C|MCE_R] = 10\%$ performance goal, with an assumed $\beta_{COL,RTR} = 0.40$. This 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 response history analysis assessment procedure.

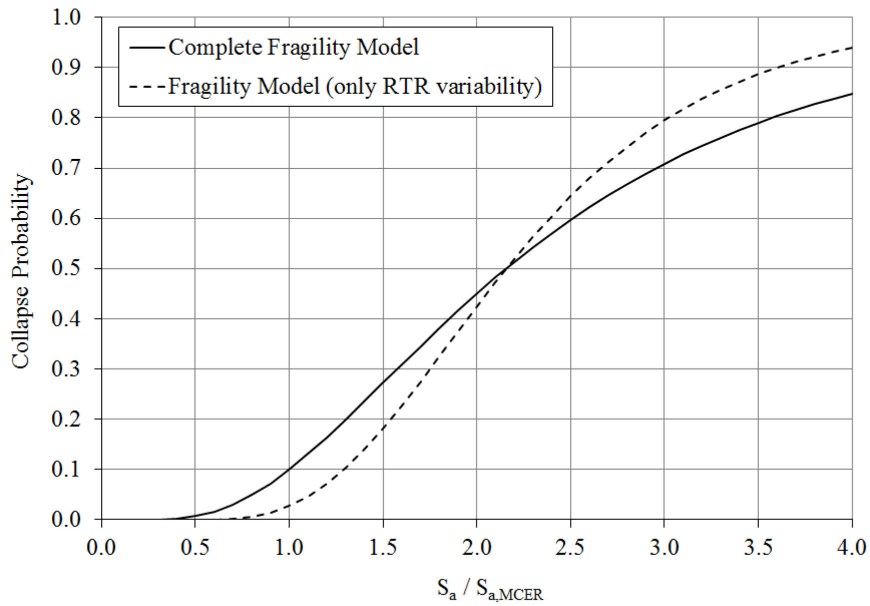

Figure A1. Collapse fragilities for a building with $P[C|MCE_R] = 10\%$ and $\beta_{COL,RTR} = 0.40$.

The binomial distribution is now used to determine the likelihood of observing collapses for buildings with various levels of collapse safety, with Table A1 showing the results for an assumed value of $\beta_{COL,RTR} = 0.40$. For a building meeting the $P[C|MCE_R] = 10\%$ performance goal, Table A1a shows that 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. Table A1b shows the same results from a slightly difference perspective; this table shows that, even for a building that meets the $P[C|MCE_R] = 10\%$ performance goal, there is a 26% chance of observing one or more collapses. From these tables, the following conclusions can be made (for $\beta_{COL,RTR} = 0.40$):

- Even if no collapses are observed in the set of eleven 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 non-collapse acceptance criteria (e.g. criteria for drifts and element demands) must be relied upon to enforce the 10% collapse probability goal.

- Even if the $P[C|MCE_R] = 10\%$ performance goal is met, there is still a 26% chance that one or more collapses will be observed in the set of eleven records (i.e. there is a 26% likelihood of a "false positive"). Therefore, an acceptance criterion of "no collapses allowed" would be commonly violated by a building that fully meets the collapse safety goals.
- 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 eleven records. Therefore, an acceptance criterion that prohibits two collapses would be reasonable.

Table A1. Results of simple statistics for the likelihood of observing collapses for $\beta_{COL,RTR}$ = 0.40.

Number of Collapses	**Likelihood for Various P[C|MCE_R]**				
	5%	**10%**	**15%**	**20%**	**30%**
0 of 11	93%	74%	51%	30%	7%
1 of 11	7%	23%	36%	38%	21%
2 of 11	0%	3%	11%	22%	29%
3 of 11	0%	0%	2%	8%	24%
4 of 11	0%	0%	0%	2%	13%
5 of 11	0%	0%	0%	0%	5%

(a)

Number of Collapses	**Likelihood if P[C|MCE_R] = 10%**
≥ 1 of 11	26%
≥ 2 of 11	3%
≥ 3 of 11	0%
≥ 4 of 11	0%
≥ 5 of 11	0%

(b)

The collapse likelihoods shown in Table A1 are based on a relatively large record-to-record variability value of $\beta_{COL,RTR} = 0.40$. When the variability is suppressed in the ground motion selection and scaling, lower record-to-record variability values are expected, so Table A2 presents comparable results for $\beta_{COL,RTR} = 0.25$. This table shows that, for a building meeting the $P[C|MCE_R] = 10\%$ performance goal, the likelihood of observing a collapse response is very low. If spectral matching methods, or other selection-based methods, are used to suppress the record-to-record variability, then the observance of even a single unacceptable response is strongly suggestive of the building not meeting the $P[C|MCE_R] = 10\%$ performance goal.

Table A2. Results of simple statistics for the likelihood of observing collapses for $\beta_{COL,RTR}$ = 0.25.

Number of Collapses	Likelihood for Various P[C|MCE$_R$]				
	5%	**10%**	**15%**	**20%**	**30%**
0 of 11	100%	99%	93%	79%	30%
1 of 11	0%	1%	7%	19%	38%
2 of 11	0%	0%	0%	2%	22%
3 of 11	0%	0%	0%	0%	8%
4 of 11	0%	0%	0%	0%	2%
5 of 11	0%	0%	0%	0%	0%

(a)

Number of Collapses	Likelihood if P[C|MCE$_R$] = 10%
≥ 1 of 11	1%
≥ 2 of 11	0%
≥ 3 of 11	0%
≥ 4 of 11	0%
≥ 5 of 11	0%

(b)

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