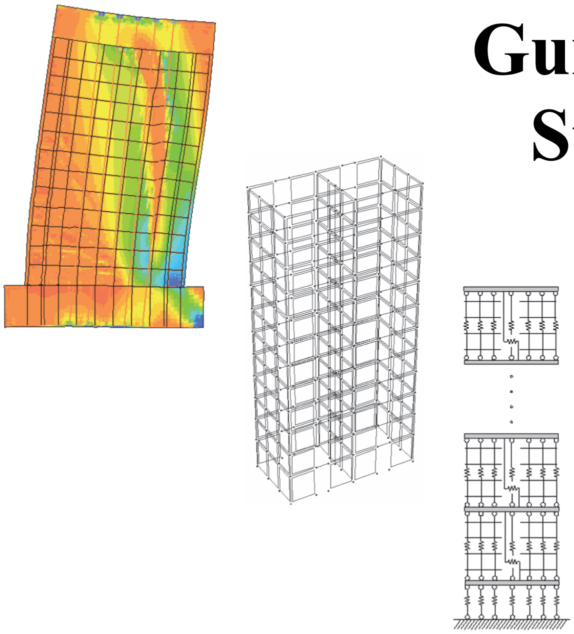
NIST GCR 17-917-46v1



Guidelines for Nonlinear Structural Analysis for Design of Buildings

Part I – General

Applied Technology Council

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National Institute of Standards and Technology
U.S. Department of Commerce

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NIST policy is to use the International System of Units (metric units) in all its publications. In this report, however, information is presented in U.S. Customary Units (inch-pound), as this is the preferred system of units in the U.S. engineering industry.

Cover image – Illustrative concrete shear wall models showing alternative strategies (Ohmura and Deierlein, 2010).

NIST GCR 17-917-46v1

Guidelines for Nonlinear Structural Analysis and Design of Buildings

Part I – General

Prepared for
U.S. Department of Commerce
Engineering Laboratory
National Institute of Standards and Technology
Gaithersburg, MD 20899-8600

By
Applied Technology Council
201 Redwood Shores Parkway, Suite 240
Redwood City, CA 94065

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Wilbur L. Ross, Jr., Secretary

National Institute of Standards and Technology
Kent Rochford, Acting NIST Director and Under Secretary of Commerce for Standards and Technology

NIST GCR 17-917-46v1

Participants

National Institute of Standards and Technology

Steven L. McCabe, Acting NEHRP Director and Group Leader
Jay Harris, Research Structural Engineer
Siamak Sattar, Research Structural Engineer
Matthew S. Speicher, Research Structural Engineer
Kevin K.F. Wong, Research Structural Engineer
Earthquake Engineering Group, Materials and Structural Systems Division, Engineering Laboratory
www.NEHRP.gov

Applied Technology Council

201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065
www.ATCouncil.org

Program Management

Jon A. Heintz (Program Manager)
Ayse Hortacsu (Associate Program Manager)
Veronica Cedillos (Associate Project Manager)

Program Committee on Seismic Engineering

Jon A. Heintz (Chair)
Michael Cochran
James R. Harris
James Jirsa
Roberto Leon
Stephen Mahin
James O. Malley
Donald Scott
Andrew Whittaker

Project Technical Committee

Gregory G. Deierlein (Project Director)
Curt Haselton (Project Director)
Stephen T. Bono
Wassim Ghannoum
Mahmoud Hachem
John D. Hooper
James O. Malley
Silvia Mazzoni
Santiago Pujol
Chia-Ming Uang

Project Review Panel

Tony Ghodsi
Jerome F. Hajjar
Yuli Huang
Michael Mehrain
Farzad Naeim
Charles Roeder
Thomas Sabol
Mark Saunders
John Wallace
Kent Yu (ATC Board Contact)

Preface

In September 2014, the Applied Technology Council (ATC) commenced a task order project under National Institute of Standards and Technology (NIST) Contract SB1341-13-CQ-0009 to develop guidance for nonlinear dynamic analysis (ATC-114 Project). The need for such guidance is identified as high-priority research and development topic (Proposed Research Initiative 6) in NIST GCR 14-917-27 report, *Nonlinear Analysis Research and Development Program for Performance-Based Seismic Engineering* (NIST, 2013), which outlines a research and development program for addressing the gap between state-of-the-art academic research and state-of-practice engineering applications for nonlinear structural analysis, analytical structural modeling, and computer simulation in support of performance-based seismic engineering. In addition, the NIST GCR 09-917-2 report, *Research Required to Support Full Implementation of Performance-Based Seismic Design* (NIST, 2009), also identified the need to improve analytical models for buildings and their components in near-collapse seismic loading.

To help fill this gap, the ATC-114 Project developed a series of reports that provide general nonlinear modeling and nonlinear analysis guidance, as well as guidance specific to the following two structural systems: structural steel moment frames and reinforced concrete moment frames. This *Part I Guidelines* document is the first in the series and provides general guidance. The companion *Part II Guidelines* (NIST GCR 17-917-46v2 and 17-917-46v3) provide further details for steel moment frame and reinforced moment frame systems, respectively. It is envisioned that these *Guidelines* will be used in conjunction with available performance-assessment provisions, or their equivalent, that are appropriate for the specific circumstances.

This *Part I Guidelines* document was developed by the members of the ATC-114 Phase 2 (*Steel Moment Frames*) and ATC-114 Phase 3 (*Reinforced Concrete Moment Frames*) project teams. ATC is indebted to the leadership of Greg Deierlein and Curt Haselton, who served as Project Directors of Phase 2 and 3, respectively. The Project Technical Committee of Phase 2 and 3, consisting of Stephen Bono, Wassim Ghannoum, Mahmoud Hachem, John Hooper, Jim Malley, Silvia Mazzoni, Santiago Pujol, and Chia-Ming Uang monitored and guided the technical efforts of the Project Working Groups, which included Dustin Cook, Ian McFarlane, Gulen Ozkula, Hee Jae Yang, and Zhi Zhou. The Project Review Panel, consisting of Tony Ghodsi, Jerry Hajjar, Yuli Huang, Mike Mehrain, Farzad Naeim, Charles Roeder, Tom Sabol, Mark Saunders, John Wallace, and Kent Yu (ATC Board Representative) provided

technical advice and consultation over the duration of the work. The names and affiliations of all who contributed to this report are provided in the list of Project Participants.

ATC also gratefully acknowledges Steven L. McCabe (Contracting Officer's Representative), Jay Harris, Siamak Sattar, Matthew Speicher, and Kevin Wong for their input and guidance throughout the project development process. ATC staff members Veronica Cedillos and Carrie Perna provided project management support and report production services, respectively.

Ayse Hortacsu Associate Program Manager	Jon Heintz Program Manager

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Chapter 1

Introduction and Scope

Applications of nonlinear structural analysis for seismic design of buildings have become increasingly prevalent in recent years for design and performance assessment of both new and existing buildings. Whereas nonlinear static (pushover) analysis was at the forefront of engineering practice in the mid- to late-1990s, today, nonlinear dynamic (response history) analysis has become more accessible for practice. Examples of performance-based seismic design and assessment approaches that employ nonlinear analysis include:

- Building code analysis for design of new building structures, such as in ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010). It is noted that the forthcoming 2016 edition of ASCE/SEI 7 includes a major update to Chapter 16, Nonlinear Response Analyses, which is based on draft provisions developed for the 2015 National Earthquake Hazard Reduction Program (NEHRP) *Recommended Seismic Provisions Update* (FEMA, 2015).
- Alternate analysis methods for the design of new tall buildings, such as those described in the Pacific Earthquake Engineering Research Center *Tall Buildings Initiative: Guidelines for Performance-Based Seismic Design of Tall Buildings* (PEER, 2010 and 2017) and the Los Angeles Tall Buildings Structure Design Council's *An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region* (LATBSDC, 2015).
- Analysis to evaluate the performance of existing buildings, such as the approach presented in ASCE/SEI 41-13, *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE, 2013 and 2017).
- Analysis to evaluate the overall seismic performance of buildings, including prediction of losses and other performance measures, as presented in FEMA P-58, *Seismic Performance Assessment of Buildings* (FEMA, 2012).
- Collapse analysis of structural building systems, such as the approach described in FEMA P-695, *Quantification of Building Seismic Performance Factors* (FEMA, 2009).

Utilizing the above guidelines and standards, nonlinear analysis can be used to design and assess the performance of buildings subjected to earthquake ground motions. Typical structural response measures that are calculated from these analyses (called "demand parameters") are the drifts of each story, accelerations of each floor, deformations of yielding or "deformation-controlled" components, and force demands in "force-controlled" components that are expected to remain elastic. These calculated demand parameters are then used to either: (a) evaluate conformance to acceptance criteria using prescriptive code-based procedures (e.g., ASCE/SEI 7; ASCE/SEI 41; PEER, 2010; or LATBSDC, 2011); (b) predict explicit performance metrics related to functionality, losses and safety (e.g., using FEMA P-58); or (c) evaluating collapse risk (e.g., using FEMA P-695).

The above guidelines have substantially advanced the state-of-the-art in performance-based earthquake engineering, but they often lack specific guidance for creating nonlinear structural models and performing the analyses for specific material and structural systems. One exception to this is ASCE/SEI 41, which does include nonlinear modeling and acceptance criteria for specific systems, although the provisions of the current 2013 edition of ASCE/SEI 41 are geared primarily to nonlinear static analysis, with limited coverage of dynamic analysis and explicit modeling of cyclic behavior. This requires users to determine appropriate modeling and analysis methods for the type of building they are evaluating, which is both time consuming and can result in inconsistencies in design practice.

The guidance included in this series of reports are intended to address the need to establish consistent modeling parameters and assumptions for nonlinear dynamic analysis of common types of structural systems used in buildings. This *Part I Guidelines* document is the first in a series of reports to help fill this gap by providing general guidance for creating nonlinear models and conducting nonlinear analyses. The companion *Part II Guidelines* provide further details for selected structural system types: NIST GCR 17-917-46v2 for steel moment frames and NIST GCR 17-917-46v3 for reinforced concrete moment frames (NIST, 2017b; 2017c). It is envisioned that these documents (*Part I* and the *Part II Guidelines* appropriate to a system type of interest) will be used in conjunction with one of the performance-assessment provisions listed above, or their equivalent, that is appropriate for the specific circumstances. Accordingly, this *Part I Guidelines* document focuses on providing practical structural modeling and analysis guidance, but it does not attempt to repeat or prescribe all of the possible performance goals and detailed numerical acceptance criteria contained in other documents.

Although these series of *Guidelines* are generally applicable to nonlinear analysis of both new and existing buildings, they emphasize applications of nonlinear dynamic (response-history) analysis for the seismic design and performance assessment of new buildings over the expected range of response commonly evaluated. By emphasizing the practical use of nonlinear dynamic analysis for new buildings, the goal is to enable the utilization of new structural systems and response modification technologies that have the potential to transform building construction. The

Guidelines do not address all of the structural deficiencies and complex modes of failures that may be encountered in existing buildings, nor do they emphasize the highly nonlinear degrading response modes that occur during collapse.

These *Part I Guidelines* are primarily intended for use by engineering practitioners who are well versed in seismic design and behavior and familiar with the concepts and limitations of nonlinear structural analysis, but who desire more detailed guidance on nonlinear modeling. Similarly, the objective of these *Part I Guidelines* is not to cover basic principles of nonlinear analysis, but rather to provide nonlinear modeling guidance to practitioners who are already experienced with these topics.

This document is intended to provide comprehensive guidelines for nonlinear analysis, and it intentionally does not repeat material from other established standards and reference documents. In addition to the reference standards cited above, users are encouraged to reference the following documents:

- NIST GCR 10-917-5, *NEHRP Seismic Design Technical Brief No. 4*, *Nonlinear Structural Analysis for Seismic Design, A Guide for Practicing Engineers* (NIST, 2010)
- NIST GCR 11-917-15, *Selection and Scaling Earthquake Ground Motions for Performing Response-History Analyses* (NIST, 2011)
- NIST GCR 12-917-21, *Soil-Structure Interaction for Building Structures* (NIST, 2012)

Chapter 2 of this provides an overview of the full process used for a building seismic performance assessment using nonlinear response-history analysis (inclusive of both modeling and other aspects of the process, such as acceptance criteria). Chapter 3 provides specific detail on the requirements for the *modeling po*rtion of this overall assessment process. Chapter 4 discusses the roles and limitations of nonlinear static analysis (which is not recommended as the final performance check for both building types), and then Chapter 5 outlines the recommended nonlinear response-history analysis procedure. Chapter 6 details how acceptance criteria are then checked to assess building performance. Appendices supplement this material, with Appendix A providing an overview of methods for response-history analysis, Appendix B providing background documentation on how uncertainties should be treated in the development of acceptance criteria, and Appendix C providing instructions on calibrating a nonlinear component model using test data.

Chapter 2

Overview of Nonlinear Modeling and Analysis Procedure

2.1 Performance-Based Seismic Design and Assessment

Nonlinear structural analysis is one of many important steps of the performance-based seismic design and assessment process. The list presented below outlines the overall performance-based design process, assuming that the design objectives and performance assessment framework have already been established, and provides some detail for each step. This *Guidelines* document focuses primarily on providing guidance for creating and running a nonlinear structural analysis model (Steps 4 and 5). The other steps of the process are also discussed in a more limited manner to provide context for how modeling fits within the overall performance assessment process and implications on the requirements of the nonlinear analysis model. Additional general guidance is provided in NIST GCR 10-917-5, *Nonlinear Structural Analysis for Seismic Design, A Guide for Practicing Engineers* (NIST, 2010).

Step 1: Define the purpose and goals of nonlinear analysis. It is important to clearly establish the purpose and goals of the analysis. What is the analysis intended to show about the behavior and performance of the building? What are the performance goals for the building and how are they to be evaluated by the nonlinear analysis? What are the ground motion level(s) for which the performance will be assessed? How much nonlinearity is expected to occur? The documents referenced in Chapter 1 can be used to help determine the purpose and goals of an analysis.

Step 2: Understand structural behavior and failure modes. As a first step to creating the structural model, all relevant behavioral and failure modes should be identified and understood, so that all of the effects that are likely to have a significant effect on the structural response are considered in the nonlinear model and performance assessment. It is not always possible or necessary to directly simulate all possible nonlinear effects in an analysis. When a failure mode is not included in the analysis model, a non-simulated failure model can be assessed in accordance with Section 2.3.4 of this *Guideline*. The behavioral and failure modes are specific to building structural system types are discussed in the system-specific *Part II Guidelines*.

Step 3: Define the demand parameters and acceptance criteria. Demand parameters must be defined to provide the basis for evaluating acceptance criteria or other performance measures. Demand parameters generally include: peak transient story drifts, residual story drifts, peak floor accelerations, inelastic deformations in ductile deformation-controlled elements, peak stress resultants in force-controlled elements, and (in some cases) the ground motion intensities that cause structural collapse or other behaviors. Chapter 6 discusses performance assessment and acceptance criteria, and points the reader towards other standards that provide acceptance criteria for the design or performance assessment.

Step 4: Develop the analytical structural model. Development of the structural analysis model and the nonlinear modeling parameters begins with the high-level decision about the type and level of modeling detail. Models can range from macro-scale plastic hinge type models to micro-scale continuum finite element models. The modeling decisions depend on which elements are expected to undergo inelastic deformations (deformation-controlled components, modeled nonlinearly) and which elements are intended to remain essentially elastic (force-controlled components, modeled elastically). These distinctions depend on the characteristics of the structure and the use of capacity design concepts to control the locations of inelasticity. The selection of model type and modeling parameters for deformation-controlled components may also depend on the extent of inelastic deformations expected. Chapter 3 provides general guidance for developing the nonlinear analysis model, and *Part II Guidelines* provide more specific modeling recommendations for specific types of structural systems. Additional guidance and information on nonlinear model validation and calibration is included in Appendix B. Uncertainties in model parameters and response may be accounted for through use of sensitivity analyses (see Section 6.6).

Step 5: Conduct the analyses. Nonlinear analyses are conducted under appropriate gravity loads, other non-seismic loads, and seismic load effects. Seismic effects can either be modeled through a nonlinear static (pushover) analysis or a nonlinear dynamic (response history) analysis. Chapter 4 provides guidance and supporting references on nonlinear static analysis, which given the current state of practice, should generally be used only as a step in the model testing and validation process. Chapter 5 provides guidance on nonlinear dynamic analysis, including selection and scaling of earthquake ground motions and numerical solution aspects of nonlinear analysis.

Step 6: Evaluate the building seismic performance through acceptance criteria. Evaluation of the seismic performance is usually is accomplished by the following: (1) Comparing the demand parameters to pre-defined acceptance criteria (e.g., drift limits specified in ASCE/SEI 7 or component deformation limits in ASCE/SEI 41); (2) using the demands as input to evaluate damage and loss functions (e.g., FEMA

P-58 component fragility functions); or (3) using the demands to interpret the structural response directly (e.g., excessive deformations or forces that would indicate potential deficiencies in the structure). In all cases, consideration should be given to variability in the calculated demands due to uncertainties in the nonlinear response, further details of which are discussed in Appendix B. Chapter 6 provides a general discussion of this step, including some examples of common acceptance criteria (e.g., Chapter 16 of ASCE/SEI 7-16), prescribed in some of the supporting reference documents and standards.

Step 7: Peer review and documentation. The peer review and documentation requirements will depend on the type of assessment and the requirements of the owner or jurisdiction overseeing the performance assessment. In ASCE/SEI 7-16, the design review and document requirements are outlined in Sections 16.1.2 and 16.5.

2.2 Demand Parameters

Nonlinear modeling decisions are highly influenced by the type of structural demand parameters that need to be evaluated in the structural analysis. Methods for assessment using nonlinear dynamic analyses will generally require analysis under a series of ground motions, from which the demand parameters are calculated. In general, ASCE/SEI 7 and other standards specify that the demand parameters should be evaluated based on mean (arithmetic average) quantities. Given that the statistical variability in demands generally follows a lognormal distribution, the mean values are different (usually larger) than the median (50^{th} percentile) values. Although it is useful to note the observed variability of the demand parameters (dispersion or standard deviation), the calculated variability is usually not relied upon in the acceptance checks since the demands are calculated from limited data sets where the variability in the model parameters and ground motions is generally not well represented. This section further discusses the demand parameters and what structural modeling decisions tangibly affect the predictions for each parameter.

2.2.1 Response Values

Average Peak Story Drift. The average peak drift in each story of the building is a fundamental demand parameter used in nearly all performance assessment methodologies. Several modeling factors affect the peak story (transient) drift, the primary one being the structural stiffness. Commonly used analysis approaches tend to underestimate the building stiffness (and, therefore, overestimate the building story drifts) by only including the primary structural components of the seismic force-resisting system in the model and by underestimating certain contributing factors (e.g., assuming cracked section properties for reinforced concrete components, ignoring composite action in steel buildings with concrete floor slabs, and ignoring or underestimating the effect of finite member dimensions in line-type models). Although such approximations may be reasonable to apply in standard building design approaches, where one is focused on ensuring minimum acceptance criteria that are readily met, the conservative estimates of building stiffness are of greater concern when the calculated story drifts have a major effect on the outcome. For example, when using the methodology documented in FEMA P-58, *Seismic Performance Assessment of Buildings* (FEMA, 2012), to assess building damage, loss, and repair times, it was found that calculation of story drifts based on a bare-frame analysis model with underestimated member stiffness can significantly overestimate damage and losses (ATC, 2017). In such cases, it is recommended to use more realistic component stiffness parameters and to include stiffness provided by the gravity system components and even some of the significant nonstructural components (to the extent permitted by the building code). Additional factors to consider include assumptions regarding damping and modeling of foundations. Section 6.1 discusses acceptance criteria related to story drifts.

Maximum Peak Story Drift. In contrast to the average peak story drift, which is averaged over the peak response of all input ground motions, the maximum peak story drift from any one ground motion is statistically less significant. Thus, while the maximum peak drift should be monitored to gage the reliability of the nonlinear analysis results, care should be taken to not place too much significance in it, since the maximum peak response can be highly dependent on the ground motion selection and scaling techniques (which are not necessarily well constrained in current design practice). The same caution applies to other maximum demand measures (e.g., component deformations or forces) from any one ground motion. This is further discussed in Section 6.6.

Residual Story Drift. Residual story drifts are an important demand measure insofar as they can have major implications on whether a building can be reoccupied and repaired after an earthquake. Compared to peak story drifts, residual drifts are more difficult to accurately estimate, because they are more sensitive to the yield strength, structural degradation, unloading stiffness, and ground motion characteristics (Ruiz and Miranda, 2006). Thus, although it is straightforward to extract residual drifts from an analysis, they are generally less reliable than the peak drifts. It is for this reason that the FEMA P-58 methodology provides a simplified method for predicting residual story drifts, which relies on an estimate of yield story drift and peak story drifts. Where residual drifts are measured directly, one should consider performing additional analyses to assess their sensitivity to cyclic behavior of the components, such as unloading and reloading stiffnesses, hysteretic behavior, such as pinched versus bilinear response, and input ground motion characteristics.

Average Peak Floor Acceleration. Peak floor accelerations are primarily used for evaluating the damage to nonstructural acceleration-sensitive components in buildings. In some cases, the floor acceleration time histories are used to generate floor response spectra, which can provide a more comprehensive characterization of floor response. Floor accelerations can also be used to calculate induced forces in floor diaphragms, structural collectors, and other force-controlled components. The modeling properties most affecting floor accelerations are the building stiffness, the damping assumptions, the level of nonlinearity in the building response, and high frequency ground motion characteristics. In addition, conventional ground motion scaling procedures may not reliably capture the hazard characteristics at short periods (higher frequencies) that can significantly influence the measured accelerations. Section 6.5 discusses acceptance criteria related to floor accelerations.

Deformation. Deformation demands in yielding components are an important measure of performance for structures responding in the inelastic range. Inelastic component deformations may include either displacements or rotations (e.g., plastic hinge rotations, axial strut deformations), generalized strains (e.g., member curvatures), or strains (e.g., axial or compressive longitudinal strains in shear walls). In all cases, but particularly with strain measures, it is important to consider the gage length over which the measurements are averaged and the sensitivity of these measures to the discretization of the analysis model. While component deformations tend to track with story drifts, they can be more sensitive to factors that affect localization of deformations, such as kinematic amplification of deformations due to the geometric topology and post-peak softening response (especially among component actions that act in series with one another, e.g., relative strength and softening of beam or column hinges and joint panel zones around one connection). Therefore, to the extent that the structural analysis model reliably simulates the component behavior under the imposed deformations, the resulting story drifts are more indicative of performance than local component deformations. Nevertheless, it is important to confirm that the deformation demands are within the range of behavior that can be captured reliably in the analysis. Section 6.3.2 discusses acceptance criteria related to deformation-controlled component actions.

Forces. For component actions that are either expected to respond elastically or are capacity designed to remain essentially elastic (e.g., shear in modern shear wall buildings), the component action is typically modeled elastically and the peak force demands for the component are recorded in the nonlinear structural analysis. Note that in this context, the term "force" is intended to include both stresses and stress resultants (forces and moments). To minimize sensitivity to the mathematical discretization of the analysis model (e.g., dependence on element and integration point discretization), stresses and stress-resultants should be integrated over representative regions of structural components, considering the sensitivity of the calculated resultants to the stress gradients and model discretization. Where the demands in force-controlled components are limited by surrounding inelastic components, the forces tend to be less variable than in situations where this is not the case. Section 6.3.1 discusses acceptance criteria related to force-controlled component actions.

Collapse. With the notable exception of FEMA P-695, *Quantification of Building Seismic Performance Factors* (FEMA, 2009), direct estimation of collapse capacity is not the objective of most performance assessment methods. Even so, some have requirements for what should be done if an instability (or excessive drift) is observed for any of the ground motions used in the response-history analysis. Where large drifts (e.g., story drift ratios exceeding 0.05) are encountered, it is important to confirm that the nonlinear structural analysis formulation has been validated for simulating large displacement response and large component deformations. Acceptance criteria that are related to a peak response behavior from one ground motion (rather than a mean response) should be interpreted carefully, since it is less statistically robust than mean (or median) measures. This point is further discussed in Appendix B. Section 6.2 discusses acceptance criteria related to both collapse and other unacceptable responses.

Other Non-Collapse Unacceptable Responses. In addition to the standard response quantities described previously, sometimes it is necessary to address other "unacceptable responses." For example, ASCE/SEI 7-16 places acceptance criteria requirements on the following additional "unacceptable responses":

- Analytical solutions that fail to converge
- Predicted demands on deformation-controlled elements that exceed the valid range of modeling
- Predicted demands on force-controlled elements that exceed the element capacity
- Predicted deformation demands on elements that exceed the deformation limits at which the members are no longer able to support the imposed gravity loads

Where such limits are exceeded in the analysis, they generally need to be compared against acceptance criteria that are established in the basis of design for the project. Section 6.2 discusses acceptance criteria related to unacceptable responses.

2.2.2 Variability of Response

In general, most demand parameters for nonlinear dynamic analysis are evaluated based on mean (or median) values calculated for a suite of analyses. Where the number of input ground motions is relatively small (e.g., 7 or 11 as specified by ASCE/SEI 7 and other standards) and where the analysis model is based only on the mean (or median) modeling parameters (i.e., not considering the variability in the modeling parameters), the calculated dispersion in the results has low statistical significance. This is the reason why most standards rely on assumed (characteristic) values of dispersion in establishing the acceptance criteria. For example, research studies have shown that the dispersion in drift demands of moment frame buildings under analysis of MCE intensity ground motions has a coefficient of variation on the order of 0.4 (Zareian and Krawinkler, 2009). The dispersion varies between demand parameters and tends to increase for higher levels of ground motion intensity.

Where rigorous quantification of uncertainties in the analysis is desired, it requires conducting analyses under multiple ground motions (on the order of 30 or more analyses at each ground motion intensity) and consideration of modeling uncertainties (e.g., variation of the nonlinear modeling parameters). Where both median and dispersion are required, such as in the FEMA P-58 procedures, techniques are often employed to estimate the uncertainties. Background on the treatment of variability is further discussed in Appendix B.

2.3 Overview of the Nonlinear Modeling Approach

Once the goals of the analysis are established and the required demand parameters are identified, the next step in creating the nonlinear structural model is to decide what components of the building to include and how these should be idealized in the analysis model. In general, all components that tangibly affect the building responses (demand parameters) of interest should be included in the structural model, although, with sufficient justification, the model can be simplified. Excluding some components is reasonable provided it does not lead to: (1) a non-conservative assessment of limit states that affect life safety; or (2) unacceptable biases in broader performance evaluations (e.g., biased loss estimates resulting from systematically underestimating stiffness).

Components of the gravity system should typically be included in the structural model, particularly if: (1) the gravity components increase the seismic demands in other force-controlled components; and/or (2) the goal is to perform a realistic performance analysis (as opposed to a minimum safety check). If the gravity framing components are excluded, the possible failure modes of these components should be assessed as a non-simulated failure mode in accordance with Section 2.3.4.

Nonstructural components can have significant impacts on the total building stiffness, particularly at lower seismic hazard intensities and drifts, which can significantly influence the calculated demands and subsequent loss assessment calculations (e.g., using FEMA P-58). However, nonstructural components are rarely accounted for in typical structural models, primarily because their contribution to the collapse safety is not permitted by building codes and because of the challenge in modeling these components.

2.3.1 Classification of Components by Expected Behavior

Once the structural model components have been identified, the next step is to classify each component action as either deformation-controlled or force-controlled. Note that the classification is done for each component action, where there may be two or more actions within each component. Deformation-controlled actions are those that have reliable inelastic deformation capacity with gradual strength decay, whereas force-controlled actions are associated with elements that are intended to remain essentially elastic, due to: (1) concerns for either sudden failure modes and/or inelastic response that cannot be modeled reliably; or (2) a conscious design decision motivated by concerns for structural safety or other reasons. Deformation-controlled actions are represented in the structural model with inelastic elements, and force-controlled actions are modeled with elastic elements. Beyond modeling differences, these distinctions between force- and deformation-controlled actions are also reflected in the structure of the associated acceptance criteria checks (per Chapter 6).

Examples of deformation-controlled actions include flexural response of beams or walls, axial deformations of buckling restrained braces, and shear and/or flexural yielding of coupling beams between walls or links of eccentrically braced frames. Examples of force-controlled actions include shear in reinforced concrete shear walls, axial forces in columns, and shear and flexure in floor diaphragms.

2.3.2 Deformation-Controlled Components

Deformation-controlled component actions must be modeled to reliably capture their expected nonlinear responses under the expected earthquake shaking (e.g., service level, MCE safety checks, collapse). The sophistication of the nonlinear component modeling should relate to the level of nonlinearity expected in the component. In other words, the modeling requirements for components that undergo large inelastic deformations will be more demanding than for components with small deformation demands.

2.3.2.1 Characteristics of Cyclic Strength and Stiffness Degradation

A significant challenge in nonlinear analysis is to simulate the nonlinear response under random cyclic loading that occurs under earthquakes. Figure 2-1 shows an illustration of test data from three identical steel beam-columns that are subjected to three different lateral loading protocols. Under monotonic loading (blue curve) the column reaches its peak strength at a chord rotation of about 0.04, after which the strength gradually degrades under increasing deformations due to combined local and lateral-torsional buckling. When subjected to symmetric cyclic loading (red curve), which is the standard loading protocol applied in tests for earthquake applications, the peak strength is reached at a chord rotation of about 0.02 (i.e., at half the deformation), after which the strength and stiffness degrade rapidly. The difference in response is dramatic, as evidenced by the deformation at which the lateral resistance is lost, at a chord rotation of about 0.06 under symmetric cyclic loading versus over 0.20 (three times larger) under monotonic loading. The green curve in Figure 2-1 is the response under a loading protocol that reflects the deformations a column may undergo in a frame that experiences collapse under an extreme earthquake ground motion. This curve shows how the actual response is likely to lie between the cyclic symmetric and monotonic response. The influence of loading protocol on response will vary depending on the characteristics of the structure. Ideally, the nonlinear analysis model should reliably capture the full range of expected behavior, where the strength and stiffness degradation evolves under random earthquake loading.

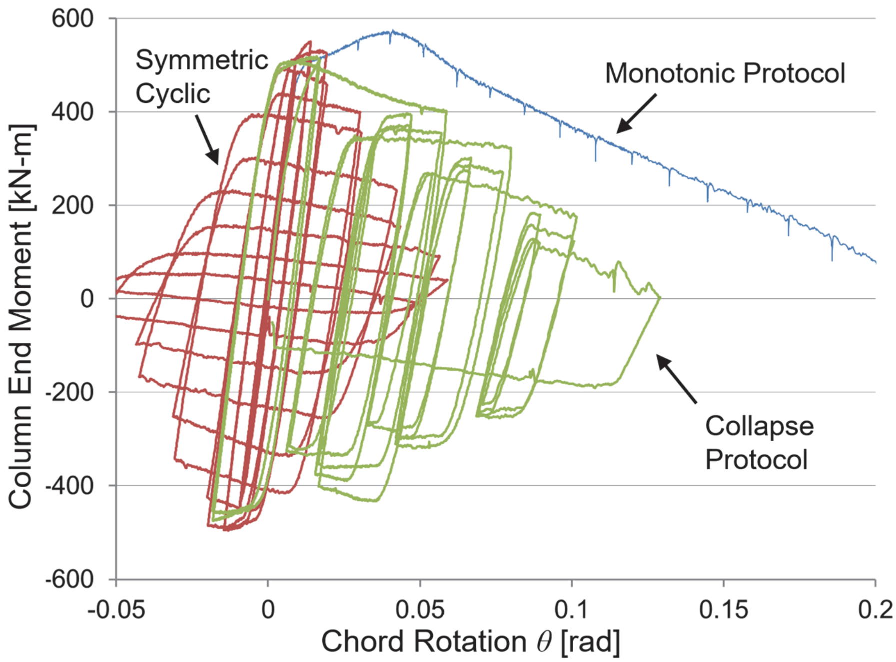


Figure 2-1 Response of steel columns subject to three loading protocols (data from Suzuki and Lignos, 2015).

Figure 2-2 shows the monotonic envelope and cyclic backbone curves for the column tests shown in Figure 2-1. In this case, the monotonic envelope is obtained directly from a monotonic test, although, where monotonic data are not available, it can be inferred from cyclic data and other supporting information. The cyclic backbone curve is obtained from cyclic test data, formed by connecting the peak load points at each level of increasing deformation (PEER/ATC, 2010) and is thereby dependent on the loading protocol that was used in the testing. The strength loss under cyclic loading generally occurs due to a combination of so-called “in-cycle” degradation, characterized by a negative slope to the load-deformation response, and “between-cycle” or “cyclic” softening, where the load drops between cycles, even in cases where the reloading stiffness of the component remains positive. In contrast to the monotonic curve, the cyclic backbone is non-unique and depends on the test loading protocol. As evidenced by the plots in Figure 2-2, the cyclic backbone from a standard symmetric reverse cyclic test is usually a conservative measure of the deformation capacity of structural components.

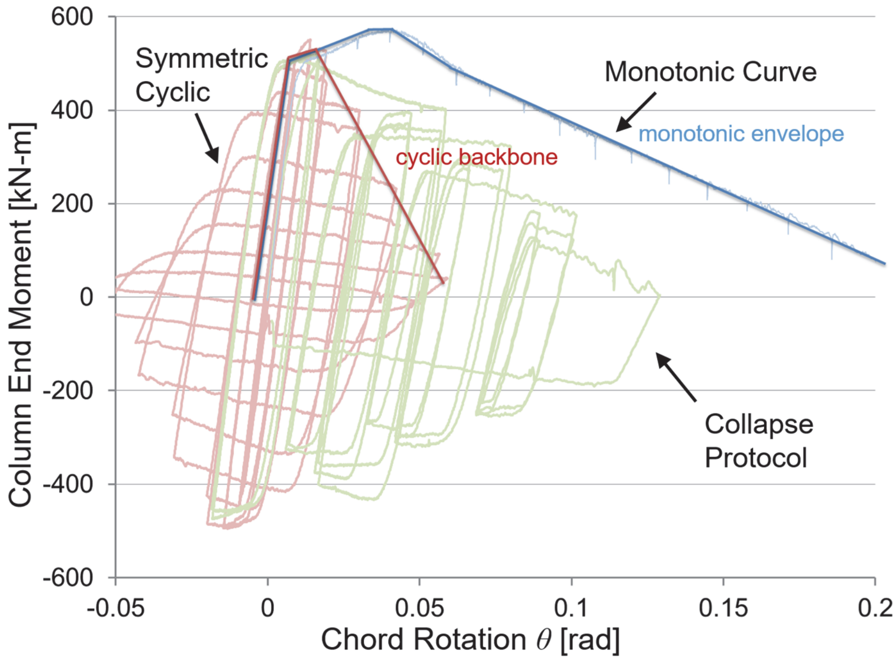


Figure 2-2 Monotonic envelope and cyclic backbone curve superimposed on data from steel column tests (data from Suzuki and Lignos, 2015).

To provide another example of the effects of loading protocols, Figure 2-3 shows the test data of five identical reinforced concrete columns subjected to five different loading protocols (Nojavan et al., 2015). This shows that the responses of the column are highly dependent on the loading protocol and the responses show various mixtures of cyclic versus in-cycle strength degradation. Figure 2-3a shows the results of monotonic loading, Figure 2-3b shows a highly damaging protocol with many cycles, Figures 2-3c and 2-3d show cyclic protocols followed by a monotonic push, and Figure 2-3e shows the results of an expected loading protocol that may cause collapse of a building (and it can be seen that the component response, for the expected loading protocol, is closer to the monotonic responses than the response with many cycles of loading).

To summarize the effects of loading protocol, and particularly the in-cycle versus cyclic degradation modes, Figure 2-4 provides a summary example identifying the locations of the two types of degradation. This is an example of data from a concrete column test that is used to calibrate a nonlinear component model (Haselton et al. 2008, 2016). In this case, cyclic strength deterioration is observed in the cycles before 5% drift and in-cycle strength deterioration in the two cycles that exceed 5%

to 6% drift. The distinction between in-cycle and cyclic deterioration is explained further in several references (Ibarra et al., 2005; Ibarra, 2003; FEMA, 2005), stressing the importance that the in-cycle deterioration having a negative stiffness is what plays an important role in the dynamic instability that can cause the collapse of a structure. In contrast, the cyclic deterioration results in strength loss but does not lead to the same problems with dynamic instability of the structural model. Therefore, it is important that nonlinear models can realistically capture both modes of response up to the level of deformations encountered in the analysis.

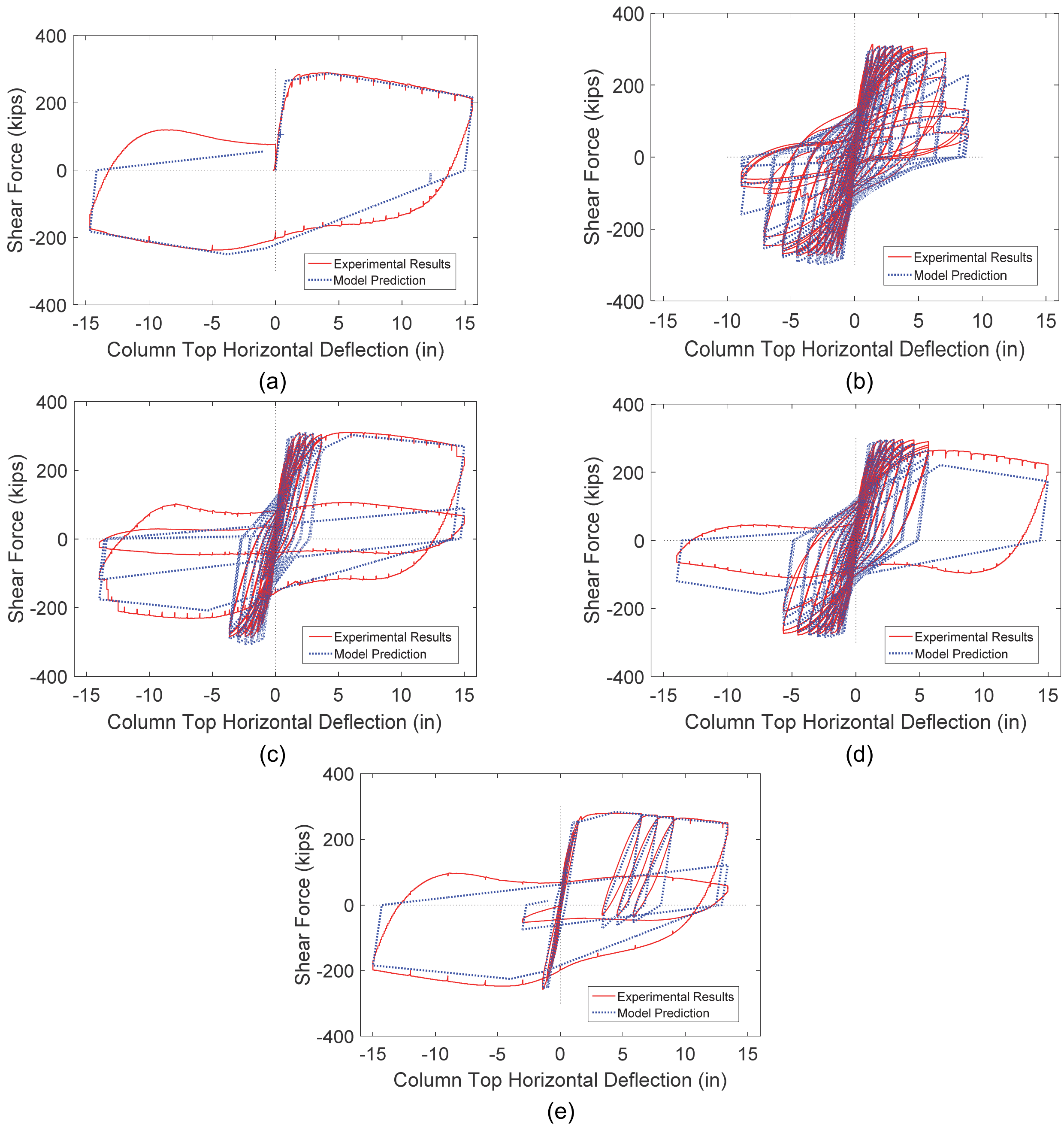


Figure 2-3 Results of five identical full-scale reinforced concrete columns tested under various loading protocols (Nojavan et al., 2015).

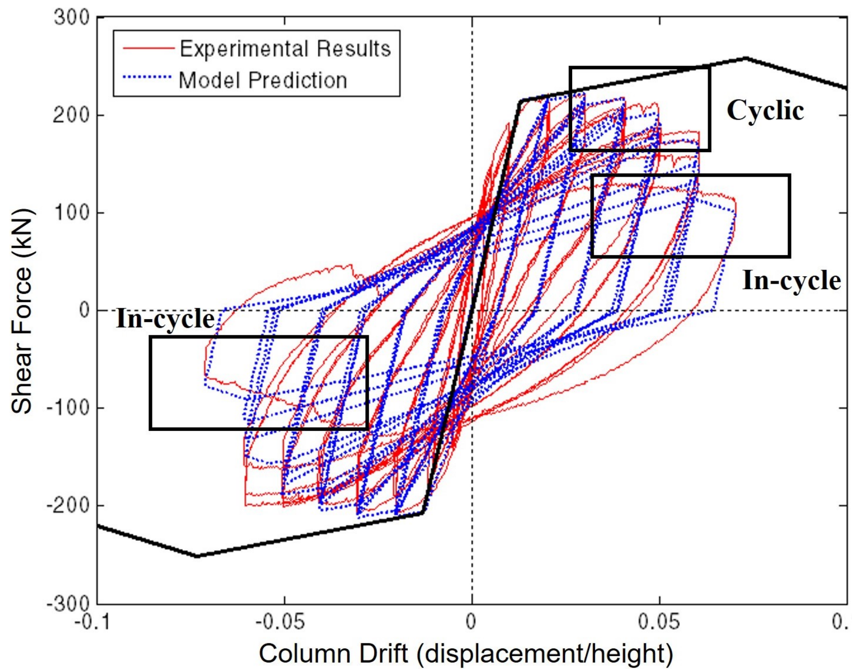


Figure 2-4 Example of test data and calibrated component model, illustrating the differences between cyclic and in-cycle strength deterioration. Experimental test is by Saatcioglu and Grira (1999), specimen BG-6, and the model calibration figure is after Haselton et al. (2008).

2.3.2.2 Relationship of Backbone Curves to ASCE/SEI 41

For nonlinear dynamic analyses, where the cyclic loading is simulated directly, the goal is to develop and calibrate the analysis model to simulate in-cycle and cyclic deterioration, such that it captures the evolving strength degradation under the specific cyclic load history for each earthquake ground motion. However, this is commonly not reflected in contemporary performance assessment methods, such as the 2017 edition of ASCE/SEI 41, which specify generalized load-deformation curves that are calibrated to the cyclic backbone. This approach has its roots in the original emphasis of ASCE/SEI 41 to nonlinear static (pushover) analysis, where cyclic effects are modeled implicitly by calibrating the response to the cyclic backbone curve. The approach is generally conservative and a reasonable approach for nonlinear static analysis. However, for nonlinear dynamic analysis, reliance on the cyclic skeleton curve to calibrate models may: (1) underestimate force demands in force-controlled components; and (2) be overly conservative where the response is dominated by pulse-like loading excursions, which exhibit monotonic softening characteristics (e.g., as evidenced in the test under the collapse loading protocol, shown previously in Figure 2-1).

2.3.2.3 Recommended Component Modeling Approach

To help overcome the limitations of current (e.g., ASCE/SEI 41-13) analysis and calibration approaches that rely exclusively on the cyclic backbone, NIST GCR 17-917-45, *Recommended Modeling Parameters and Acceptance Criteria for Nonlinear Analysis in Support of Seismic Evaluation, Retrofit, and Design* (NIST, 2017a), presents an extension for describing the generalized load-deformation response of components to more realistically represent their response in nonlinear dynamic analysis. As shown in Figure 2-5, the approach is based on specifying two generalized force-deformation curves: the monotonic envelope curve and the cyclic (pre-degraded) backbone curve.

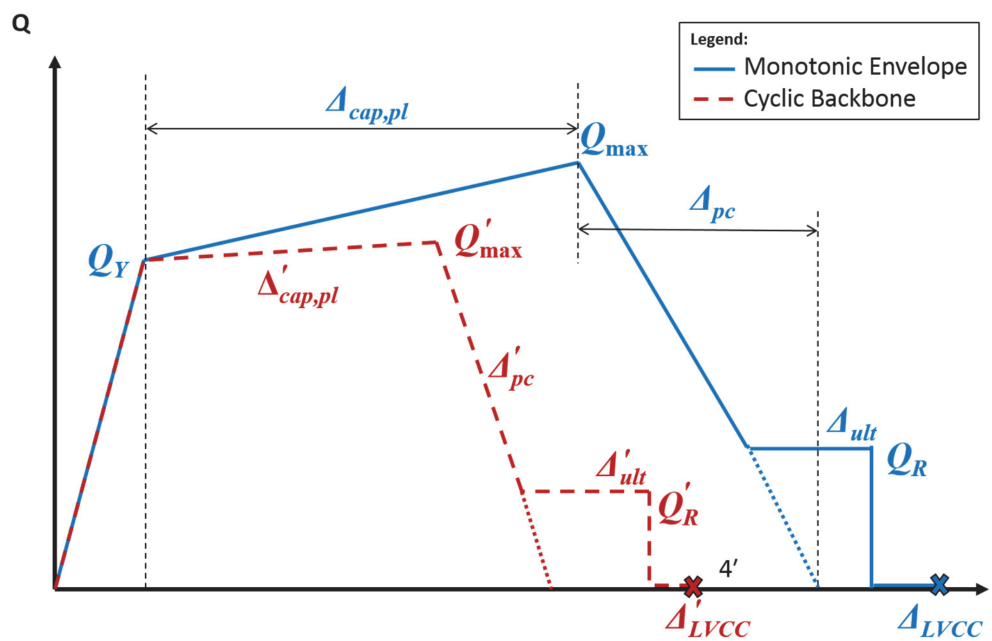


Figure 2-5 Standard cyclic and monotonic backbones with control points (NIST, 2017a).

In the figure, the following notation applies:

Q_y = element yield strength

Q_{max} = element peak strength, monotonic loading

Q'_{max} = element peak strength, cyclic loading

Q_R = element residual strength, monotonic loading

Q'_R = element residual strength, cyclic loading

$\Delta_{cap,pl}$ = plastic deformation, monotonic loading

$\Delta'_{cap,pl}$ = plastic deformation, cyclic loading

Δ_{pc} = effective post-peak deformation, monotonic loading

Δ'_{pc} = effective post-peak deformation, cyclic loading

Δ_{ult} = ultimate deformation, monotonic loading

Δ'_{ult} = ultimate deformation, cyclic loading

Δ_{LVCC} = deformation at loss of vertical load carrying capacity, monotonic loading

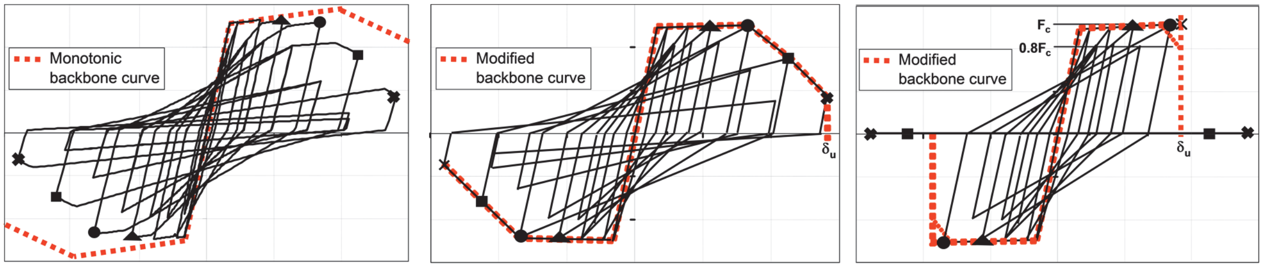
Δ'_{LVCC} = deformation at loss of vertical load carrying capacity, cyclic loading

The cyclic backbone curve is similar to the response curves currently specified in ASCE/SEI 41-13, which should continue to be used with nonlinear static (pushover) analyses. The major change is the introduction of the monotonic envelope curve, which can be used to calibrate cyclic models for nonlinear dynamic (response-history) analysis. For response-history analyses, component models should be capable of simulating both in-cycle and cyclic degradation and calibrated to replicate the range of response between these two response curves, such that the degradation experienced in each analysis will reflect the loading history experienced in that analysis. For concentrated hinge or strut type models, the backbone curves can be used directly in the model definition, whereas for distributed inelastic or continuum models, the backbone curves can be used to check or benchmark their calibration.

2.3.2.4 Alternative Component Modeling Approaches

Although models capable of simulating the full range of response in Figure 2-5 are recommended, model capabilities will depend on what is supported by available software. As a practical matter, less capable models can be used, provided the model calibration and acceptance criteria are consistent with the model capabilities. Figure 2-6 illustrates three alternatives for treating cyclic degradation in analysis and design. These concepts were first introduced in PEER/ATC-72-1, *Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall Build*ings (PEER/ATC, 2010), and PEER TBI, *Guidelines for Performance-Based Seismic Design of Tall Buildings* (PEER, 2010), for nonlinear analysis of tall buildings and have been consolidated from the original four to the following three options:

- **Model Type A.** This is the modeling approach where the full range of strength degradation is simulated directly in the model. This model is preferred because, by simulating degradation effects in the analysis, the acceptance criteria need not necessarily limit the range of behavior permitted in the nonlinear analysis.



Model Type A
Direct Simulation

Model Type B
Degraded Backbone

Model Type C
Elastic Plastic

Figure 2-6 Illustration of the three options for analytical component modeling, showing how cyclic strength deterioration is handled in each case (adapted from PEER/ATC, 2010).

- **Model Type B.** This option applies to models where the backbone response does not degrade under cyclic loading. To account for cyclic degradation, the model uses a fixed backbone curve with a pre-defined amount of cyclic deterioration. This modeling approach will tend to underestimate the seismic resistance of the structure, especially under pulse-like motions. This model may also underestimate the force demands on force-controlled components. This approach is comparable to the one used by the ASCE/SEI 41 provisions.
- **Model Type C.** In this option, the model does not capture in-cycle strength deterioration and it may not capture cyclic degradation. Due to these limitations, the model acceptance criteria are limited to the point at which strength deterioration is expected. Essentially, this approach is equivalent to applying a check for non-simulated deterioration and failure modes that are not otherwise captured in the analysis. When applied with appropriate acceptance criteria, this modeling option will tend to be the most conservative of the three options. Being the least realistic of the three approaches, this modeling option is generally discouraged.

2.3.2.4 Acceptance Criteria

For deformation-controlled components, the approach is to model the inelastic deformation behavior as accurately as possible and then use acceptance criteria to account for either (1) limitations of the analysis model to capture significant modes of degradation and failure that may affect the response and safety of the structure; and (2) controlling the amount of component damage or degradation. Examples of the former are ones that would be applied to Model Type C (described above) or to account for failure modes not captured by the other modeling options (e.g., fracture to steel structures or loss in vertical load carrying capacity, which is not simulated in the nonlinear analysis). Examples of the latter include component acceptance criteria that are tied to global performance targets, such as the Immediate Occupancy (IO) or Life Safety (LS) limits in ASCE/SEI 41-13 or to the onset of component damage and degradation applied in PEER TBI (PEER, 2010) and *An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region*

(LATBSDC, 2011). Whatever the reason, establishment of deformation acceptance criteria is usually based on judgments and assumptions regarding the underlying failure behavior and implied cyclic loading history that depend on a combination of fundamental mechanics of materials, test data, and detailed analysis. The resulting criteria generally introduce significant uncertainty and, in many cases, conservatism into the performance assessment.

2.3.3 Force-Controlled Components

Component actions that are categorized as force-controlled are usually modeled elastically, where the resulting force demands are checked against the strength limits of the component actions. Typical design guidelines will account for uncertainties in the calculated force demands and the component strengths by specifying load factors, which are applied to the average of the maximum force demands calculated from a suite of response history analyses, and/or resistance factors, which are applied to the nominal or expected component strengths. For example, the design requirements of Chapter 16 of ASCE/SEI 7-16 specify load factors that are applied to the mean demands, where the load factor depends on the criticality of the component action based on judgement of the consequence of exceeding the limit state. Alternatively, the 2017 update of *Guidelines for Performance-Based Seismic Design of Tall Buildings* (PEER, 2017) specify a combination of load and resistance factors to account for uncertainties in force demands and resistances. Although it is generally agreed that one should not conclude too much from the statistics from a small suite (e.g., 7 to 11 ground motions) of nonlinear response history analyses, it is generally recommended to check the demands on force-controlled actions from each ground motion to ensure that the nonlinear dynamic analysis results are reliable. This is discussed further in Chapter 6.

2.3.4 Non-Modeled Components

Potential failure modes for all structural components should be checked in the performance assessment, whether or not the components are explicitly modeled in the structural analysis. For components included in the structural model, failure modes can be assessed using the estimated responses and acceptance criteria (discussed previously). On the other hand, if certain components, such as members or connections in the gravity framing system, are not included in the structural model, the possible failure modes of these components must still be checked. Usually, such components are designed to ensure that they can sustain the forces and/or deformations induced by the overall framing system. For example, ASCE/SEI 7-16 requires checks of non-modeled components to ensure that they will not experience loss of gravity load resistance under the calculated deformations. This is discussed further in Chapter 6.

2.4 Types of Component Models

As illustrated for beam-column elements in Figure 2-7, models for nonlinear analysis can range from uniaxial spring or hinge models, to more fundamental fiber-type models, to detailed continuum finite element models. In general, all models are phenomenological in that they rely on empirical calibration to observed behavior at some level of idealization. The concentrated models (toward the left in Figure 2-7) are highly phenomenological in that the underlying functions that describe the structural behavior are based on calibration to overall component behavior. In contrast, the fiber and continuum finite element models (towards the right in Figure 2-7) are calibrated more at the material level, where the kinematics and equilibrium of the components are represented more directly by the model formulation. As such, the latter models are more adaptable to different geometries and loading regimes, however, to the extent possible, they should be validated against test data that represent the governing phenomena in the structural components being modeled.

The choice of model type for a given application involves a balance between reliability, practicality, and computational efficiency, subject to the capabilities of available software and computational resources. The optimal model type depends on many factors, including the structural system and materials, governing modes of behavior, the expected amount of nonlinearity, and the level of detail available for the input and output data. The reliability of the model comes from its ability to capture the critical types of deformation that are of interest to the modeler and control the response.

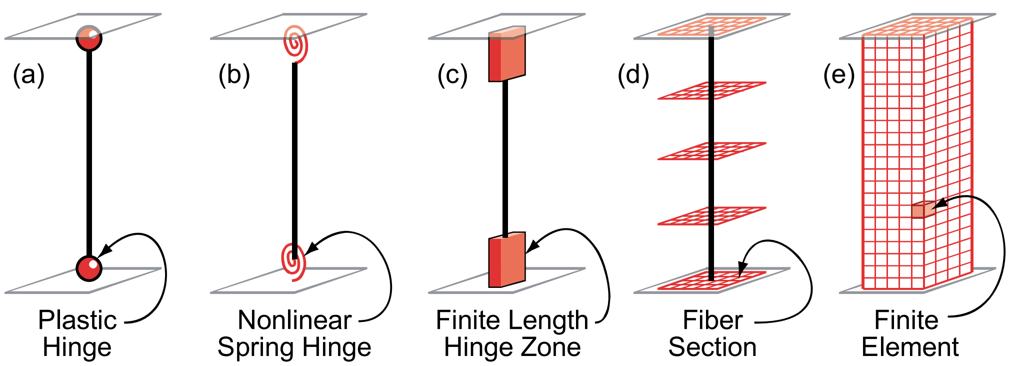


Figure 2-7 Range of structural model types (NIST, 2010).

Hinge and spring models (Figures 2-7a and 2-7b) have the advantage of being computationally efficient by modeling highly nonlinear effects in localized regions of the structure with few degrees of freedom. The models generally employ pre-defined functions to define the nonlinear response of the component. Concentrated spring models are typically implemented to capture single degree of freedom response (e.g., M-θ), but they may include multiaxial response through yield surfaces (e.g., P_x-M_y-M_z interaction) or other means. By capturing complicated behavior with highly idealized models, concentrated hinge models are very versatile, but they are also empirical and limited to modeling phenomena over the range of components and behavior modes for which they have been calibrated.

For beam-columns, fiber-type models provide the capability to numerically integrate material response through the member cross sections at a more fundamental material level. The fiber-type integration through the cross section can be used either in conjunction with a finite length hinge zone (Figure 2-7c) or with model formulations that simulate distributed inelasticity along the member length (Figure 2-7d). Fiber-type models for beams and beam-columns generally invoke kinematic assumptions, such as the Euler-Bernoulli (plane sections remain plane) assumption, to relate uniaxial stresses and strains through the member cross section to stress resultants and generalized strains for the cross section (e.g., M-ϕ or P_x-ε_x). To the extent these assumptions reflect the underlying member response, they offer tremendous benefits; however, one should be mindful of the limitations posed by these kinematic assumptions. For example, the Euler-Bernoulli assumption precludes cross-section warping or distortion due to: (1) shear and torsion effects; (2) local buckling of steel reinforcing bars or steel webs and flanges; (3) slip between components of the section; or (4) material cracking and crushing. Although certain behavior can be incorporated in the fiber material response (e.g., steel materials that are calibrated to simulate reinforcing bar buckling, or concrete models with tension softening), these adjustments require empirical calibration that negate some of the appeal of the fundamental aspects of fiber-type models.

Continuum finite element models (Figure 2-7e) represent the behavior at the most fundamental level and provide the ability to model the complete interaction of three-dimensional behavior, including complex geometries and multi-axial stress and strain states. However, three-dimensional (3D) continuum models are the most computationally intensive, particularly where the numerical mesh refinement is controlled by the smallest dimension of the member (e.g., where finite element meshing through the thickness or depth of a member will dictate the mesh size required along the member length). For this reason, 3D continuum models are typically only used to simulate portions of overall systems. In addition, while continuum models offer the potential for capturing response at very fundamental levels, their practical application is limited by both computational resources and data to calibrate certain localized behavioral effects. For example, 2D shell or 3D continuum finite element models can capture the response of isotropic steel materials fairly well, whereas many unresolved challenges remain for simulating the detailed behavior of reinforced concrete members, considering concrete cracking/dilation and interactions between steel reinforcing bars and concrete (e.g., bond and anchorage).

2.4.1 *Summary of Element Model Types*

Included below is further description of the main types of elements used for nonlinear structural analysis, including some of their advantages and limitations. While each element type is described separately, in practice, it is not uncommon to combine several element types to model building components or systems.

Spring Elements. Spring elements may represent a single degree of freedom in 2D or 3D, translation or rotation, or several coupled or uncoupled degrees of freedom, depending on the application and software capabilities. The most common examples of spring elements include concentrated flexural springs, axial strut springs, joint-panel springs, soil springs, or tension/compression springs. In some implementations, spring properties may be coupled between two or more actions, such as springs representing the axial and bending response of a beam-column hinge. With the exception of springs whose idealized properties represent physical behavior, such as idealized springs that represent tension-only or compression-only behavior (hook or gap springs), most springs are calibrated to empirical data and other behavioral information.

Line Elements. Line elements are typically used to model beams and columns in moment and braced frames, struts in braced frames and trusses, and coupling beams in coupled wall system. Line elements may also be used to model slender walls with regular geometries. Even though they are one dimensional, line elements can represent complex behavior by employing kinematic assumptions, such as the plane sections remain plane (Euler-Bernoulli) assumption in flexural members. Although, as noted previously, the implications of such assumptions may limit applicability of the elements. Response of member cross sections may be handled through uniaxial or multi-axial response (yield) surfaces or through fiber-type integrations. The line element formulations generally involve the assumption of displacement or force interpolation functions along the element length, where the order of the shape function and number of numerical integration points affect the element's ability to simulate distributed inelasticity.

2D Finite Elements. Two-dimensional finite elements are used to model floor diaphragms, walls, webs and flanges of thin-walled members, gusset and connection plates, and other plate-like components. The element formulations are well suited to structural components where the in-plane (field) and out-of-plane (thickness) dimensions are quite different, and where the through thickness strains and stresses are either assumed to be constant or linear. Where stresses and strains are constant through the thickness, the elements may be treated using plane stress or strain assumptions to model in-plane membrane action. Plate and shell elements generally employ the Euler-Bernoulli kinematic assumption to simulate out-of-plane flexural response in addition to in-plane membrane action. Displacement-based shape functions are typically employed to interpolate between the nodes.

3D Continuum Elements. Three-dimensional continuum finite elements, including brick and tetrahedral shaped elements, can be used to model the response that cannot be reliably simulated using line or 2D elements. Aside from being computationally intensive, the main challenges with continuum models are: (1) representation of multi-axial material response, including cracking and fracture; (2) modeling components comprised of two or more materials and the interface between materials; and (3) accurate representation of three-dimensional geometries, including boundary conditions and initial geometric imperfections. 3D elements can be quite effective for modeling steel where the material can be treated as isotropically modeled with well-established cyclic plasticity formulations. The main challenge for steel models is capturing fracture under large inelastic strains. Modeling of reinforced concrete, masonry, and wood is considerably more challenging due to mixing of materials and anisotropic material properties.

2.4.2 Illustrative Moment Frame Models

Two examples of models for moment frame structures are illustrated in Figures 2-8 and 2-9. The steel moment frame, shown in Figure 2-8, has fully-restrained reduced beam section (RBS) connections, where the hinge region is offset from the column face. In this case, the beam and column models consist of elastic line elements with concentrated hinges at each end. The joint panel is shown represented by an offset element, to represent the finite size of the joint panel, which can be modeled with either a flexible or rigid element, depending on the proportions of the panel and the relative strength of the panel and connected beams and columns. In this example, the RBS beam hinges are further offset from the column face using a short elastic element. The beam hinge properties would be calibrated to moment-rotation response of the RBS section. Uniaxial moment-rotation hinges can be used to simulate columns, provided that (1) the axial load due to overturning is small relative to the gravity loads; and (2) the hinges are calibrated for response under the expected axial load. Otherwise, one should either use hinges that directly simulate axial load and moment interaction or make other accommodations to the model. The column base hinges may further be adjusted to account for the base plate response. Elastic rotational springs are provided below the column base to simulate the flexibility of the concrete foundation, which is assumed to be designed to remain essentially elastic.

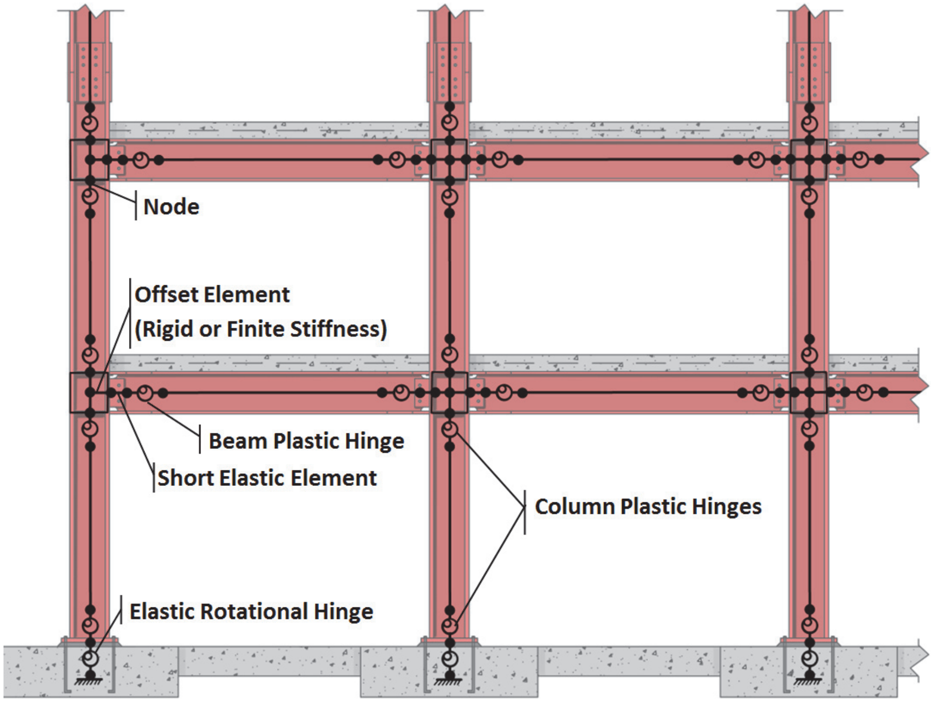


Figure 2-8 Illustrative analysis model for steel moment frame.

The concrete moment frame in Figure 2-9 illustrates an example where the beams and columns are modeled using fiber sections with a distributed-inelasticity element formulation. In this example, the two main benefits of the fiber sections are to simulate the interaction of axial load and moments in the columns and the gradual progression of cracking and yielding through the member cross sections and along their length. Arguably, with steep moment gradients, the beams, and possibly the columns, could be modeled just as effectively using fiber hinge models (Figure 2-7c) with a specified hinge length. In fact, one of the challenges with using distributed fiber models is to reconcile the calculated fiber strain demands with modeling limits and acceptance criteria, which are more readily available in the form of hinge rotations. Similar to the steel frame (Figure 2-8) example, the beam-column joint region is shown modeled with a special joint element to simulate the finite joint size with either finite or rigid joint stiffness. Between the fiber model of the beams and the joint panel, concentrated springs are inserted to simulate yield penetration and bond slip of the beam reinforcement into the joint panel. Alternatively, the yield penetration and bond slip could be implemented with a fiber hinge, where the reinforcing steel material is calibrated to capture this behavior.

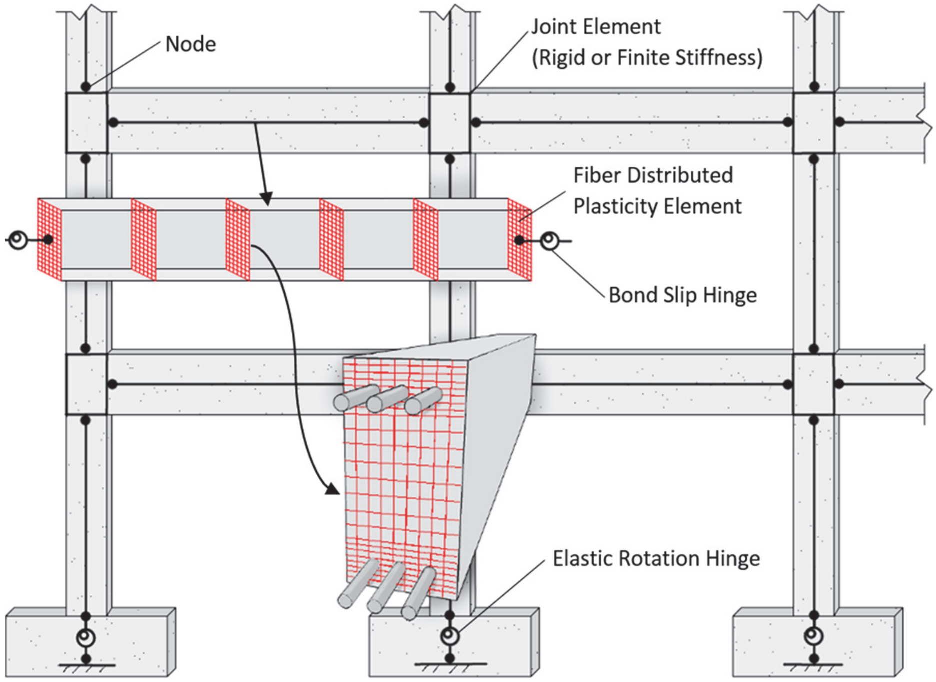


Figure 2-9 Illustrative analysis model for reinforced concrete moment frame.

It should be emphasized that these two examples are intended to illustrate the range of possible options with available modeling technologies. Hinge and fiber-type component models are often combined in the overall frame model, and the most effective model is one that combines the expected behavioral modes of the structure with the analysis software capabilities, the modeling limits, and acceptance criteria.

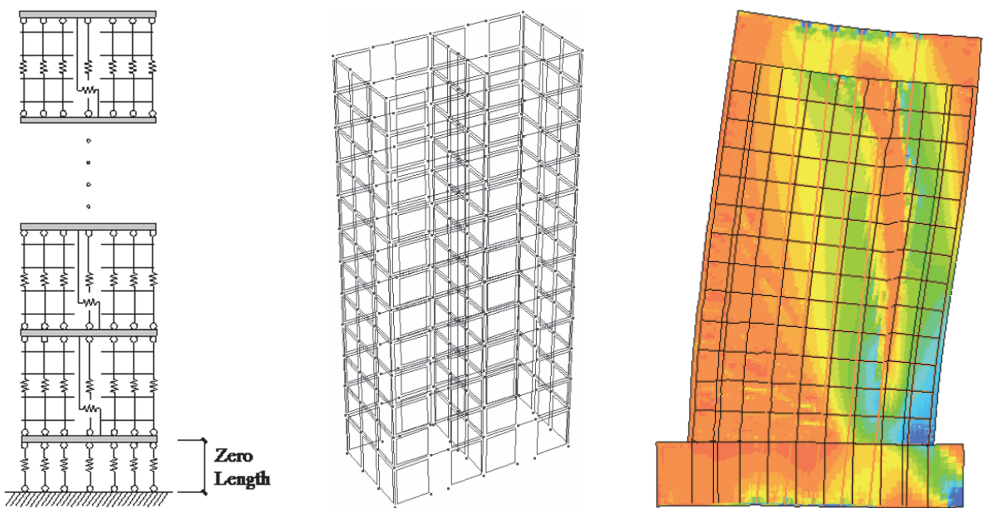
2.4.3 Illustrative Concrete Shear Wall Models

Three alternative strategies for modeling reinforced concrete shear walls are shown in Figure 2-10. Figure 2-10a is an example where flexural behavior is modeled using uniaxial springs combined with kinematic constraints to enforce behavior that is analogous to the plane sections remain plane assumption in beam-column element models. The axial springs are calibrated to simulate the composite behavior of concrete and longitudinal bars. The vertical spacing between kinematic constraint points (i.e., the length of the axial springs) is defined to match the effective hinge length in the slender (flexural) walls. This model also employs a shear spring model that captures the average shear behavior over each segment.

The model shown in Figure 2-10b is an assembly of concrete shear walls that are coupled together to form a box-shaped assembly of coupled wall segments around a typical building services core. In this case, portions of the core wall are modeled by a series of four-node wall panel elements (essentially 2D quadrilateral membrane or plate elements), each of which is similar to the lumped spring models in Figure 2-10a. However, by employing multiple panels through the cross section, Figure

2-10b permits warping of the core wall cross section. The core wall model also includes beam line elements to represent coupling beams between wall panels that are separated by wall openings required in the elevator lobbies of the services core.

The third model type shown in Figure 2-10c is a 2D continuum finite element, where the reinforced concrete wall is discretized into a combination of quadrilateral plane-stress elements (representing the concrete) and vertical strut elements (representing longitudinal steel reinforcement). This example was part of a study to examine the seismic response of concrete shear walls where vertical post-tensioned strands replaced about half of the conventional (mild steel) vertical reinforcement with prestressed high strength steel strands (Ohmura and Deierlein, 2010). The continuum finite element model served as a virtual experiment, providing an effective tool to investigate how the prestressed steel modified the flexural and shear response of the wall.

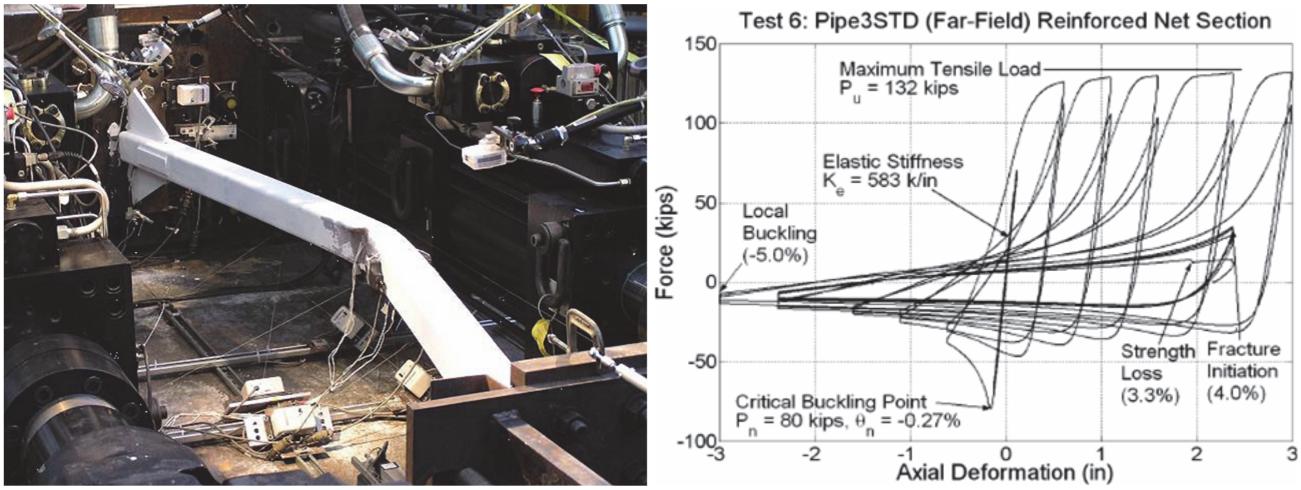


(a) (b) (c)

Figure 2-10 Illustrative models for wall analysis.

2.4.4 Illustrative Braced Frame Models

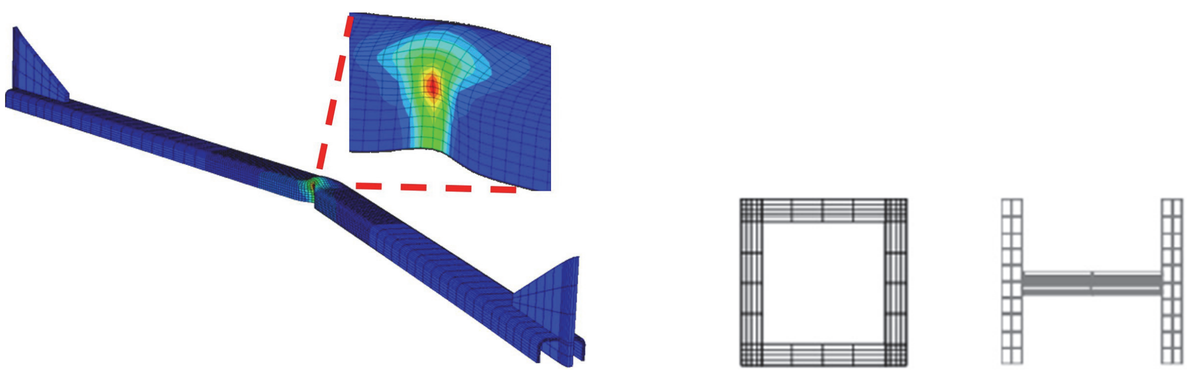
Modeling the seismic response of braced steel frames is particularly challenging, due to the sudden strength and stiffness degradation that occurs after brace buckling (shown in Figure 2-11a). Concentric braces tend to buckle at small drift ratios (often less than about 0.5% story drift), after which the brace resistance drops off rapidly. Figure 2-11b shows the critical buckling point as 0.27%. Mitigating this behavior is the reason why building codes for seismically resistant braced frames usually require braces to be provided in pairs, whereby the degradation in the buckling brace is mitigated by the tension yielding in the paired brace.



(a) (b)

Figure 2-11 Test specimen and data for concentric brace (Fell et al., 2009).

A key component to modeling braced frames is simulating the response of the buckling brace. Shown in Figure 2-12 are two approaches to the problem. One entails the use of continuum finite elements to model the detailed behavior of the hollow structural section (HSS) brace member, including the effects of overall brace buckling followed by local buckling. As illustrated in Figure 2-12a, when properly calibrated (with inelastic material properties and initial geometric imperfections), a continuum finite element model can reliably capture the brace response up to the point of brace fracture, due to localization of high strains due to local buckling.



(a) (b)

Figure 2-12 Illustrative models for buckling brace.

The alternative model in Figure 2-12b shows where the brace is modeled by beam-column fiber-type line elements, which track the interaction of axial compression and bending on the overall column. While such elements cannot simulate directly the local buckling, they have been shown to simulate overall buckling and yielding reasonably accurately. Since spread of plasticity under high axial loads is an important aspect, such models generally require that the member be discretized into at least four to six elements, with a more refined mesh closer to the center of the brace where the yielding is largest. Refer to NIST 17-917-45 for a detailed discussion.

Apart from the brace itself, the bracing connection can have an important effect on both the brace and the overall frame behavior. Figure 2-13 shows an example of a recent study that examined the significance of the brace connection on the overall brace behavior. This example illustrates how rigid end offsets can be combined with a discrete spring to represent the effect of the brace connection on both the brace buckling and the overall frame behavior.

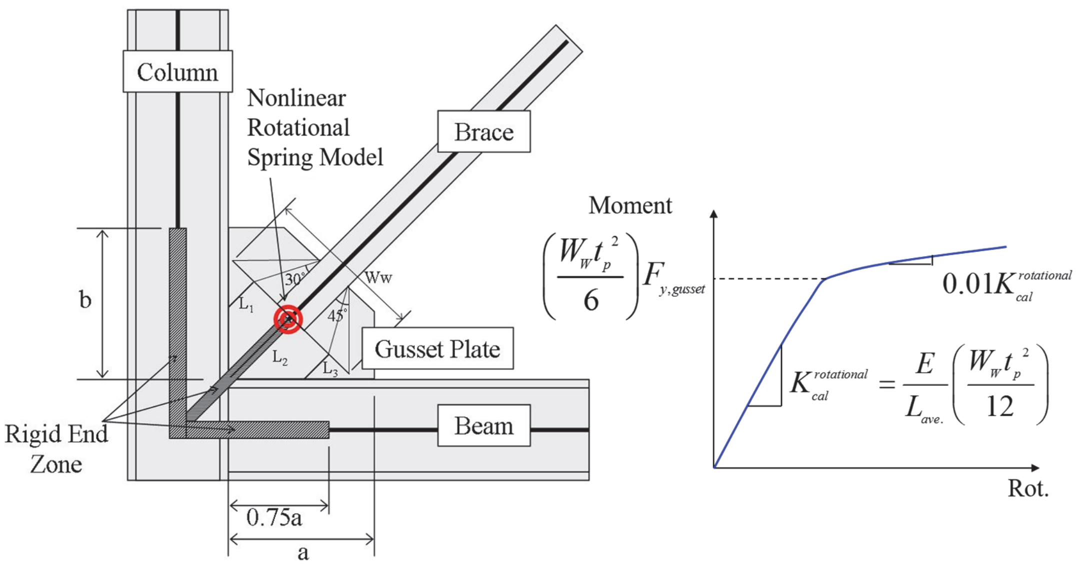
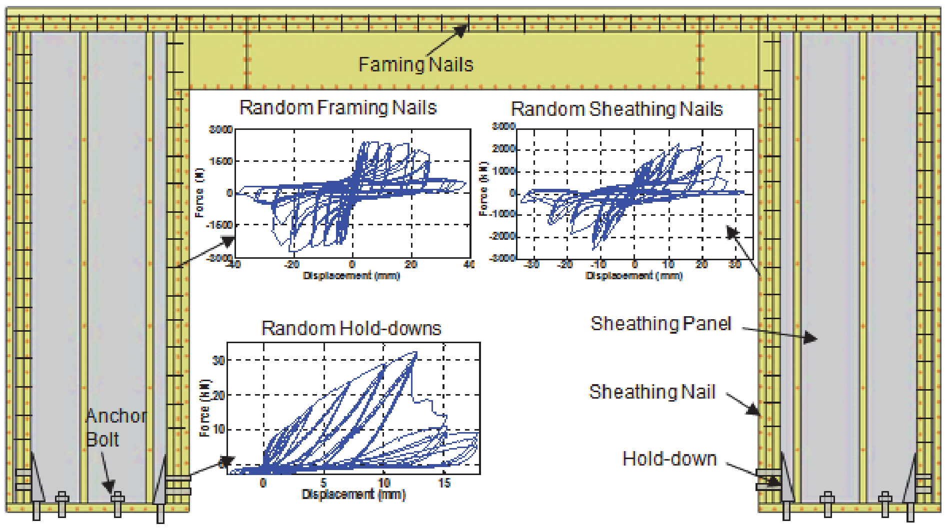


Figure 2-13 Illustrative models for beam-column-brace framing connection (Hsiao et al., 2012).

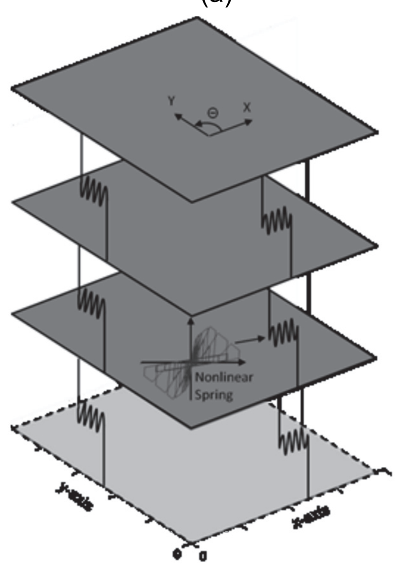
2.4.5 Illustrative Wood-Frame Shear Panel Models

The final illustrative example of nonlinear analysis modeling strategies is for wood-frame buildings shown in Figure 2-14. This illustrates the concept of using detailed assembly models of wall panels (Figure 2-14a) to calibrate overall wall panel response models (shear versus story drift) that are used to analyze the overall building (Figure 2-14b). In the detailed model, the individual wall studs are connected to sheathing panels through nonlinear connector models, representing nails or screws. Discrete shear anchor and hold down bolts are also modeled. These models are based on the assumption that most, if not all, of the nonlinearity in the wall response occurs through nonlinearity in the connectors and anchors, where the wall sheathing and wall studs remain elastic. The connectors and anchors are modeled by nonlinear zero-length springs that are calibrated to connector component tests. The wall studs are modeled as beam-column line elements, and the sheathing is modeled using either 2D elastic finite elements or, in some cases, through kinematic constraint equations.

The shear wall with a large door opening in Figure 2-14a illustrates one example where the detailed wall panel model is useful to calibrate wall spring models which can then be aggregated to analyze overall building response. Shown in Figure 2-14b is an example of a multi-story wood frame structure, where walls connecting the floors are idealized by spring (or strut) models. These models are based on observations that wood frame buildings tend to behave in a story-wise fashion, where drifts localize independently in each story.



(a)



(b)

Figure 2-14 Illustrative model of wood-framed building analysis: (a) detailed shear wall panel (Pang and Shirazi, 2013); (b) aggregated system model (Pang et al., 2012).

2.5 Recommended Procedures for Creating and Calibrating Component Models using Test Data

For the case when standards and guidelines are not available for modeling a component type of interest, the nonlinear component model can be calibrated using test data or advanced analyses. This is discussed in Appendix C.

2.6 Flowchart Outlining Alternative Modeling Types

Figure 2-15 illustrates the different modeling options discussed in this chapter. The flowchart provides a guideline for modeling decisions such as the representation of force- and deformation-controlled actions, the use of fiber and concentrated section discretization, as well as lumped and distributed inelasticity element integration schemes. The flowchart provides a conceptual and idealized decision process. In practice, other considerations may override some of the modeling decisions, with the primary consideration being the availability of implemented modeling options in the structural analysis software, as well as other considerations related to model size, complexity, performance, and the suitability of various models to the specific behavior, which may override some of the flowchart recommendations.

The first decision point outlined in Figure 2-15 is the level of detail of the model, with the primary choice being between a continuum model and a discrete model (i.e., fiber or concentrated hinge). For discrete models (which is typical for design), the next decision is how each component in the model is classified and modeled, either as a force- or deformation-controlled component, as well as which components to model (i.e., decision of if the gravity framing components are included in the model). After this, the model types for each component and the level of detail of the modeling can be decided. All of these modeling decisions are centered on how simplified versus complex the structural model should be and the needed level of complexity of the model depends on aspects such as the expected failure modes and their consequences, the anticipated locations of damage, and the anticipated level of nonlinearity.

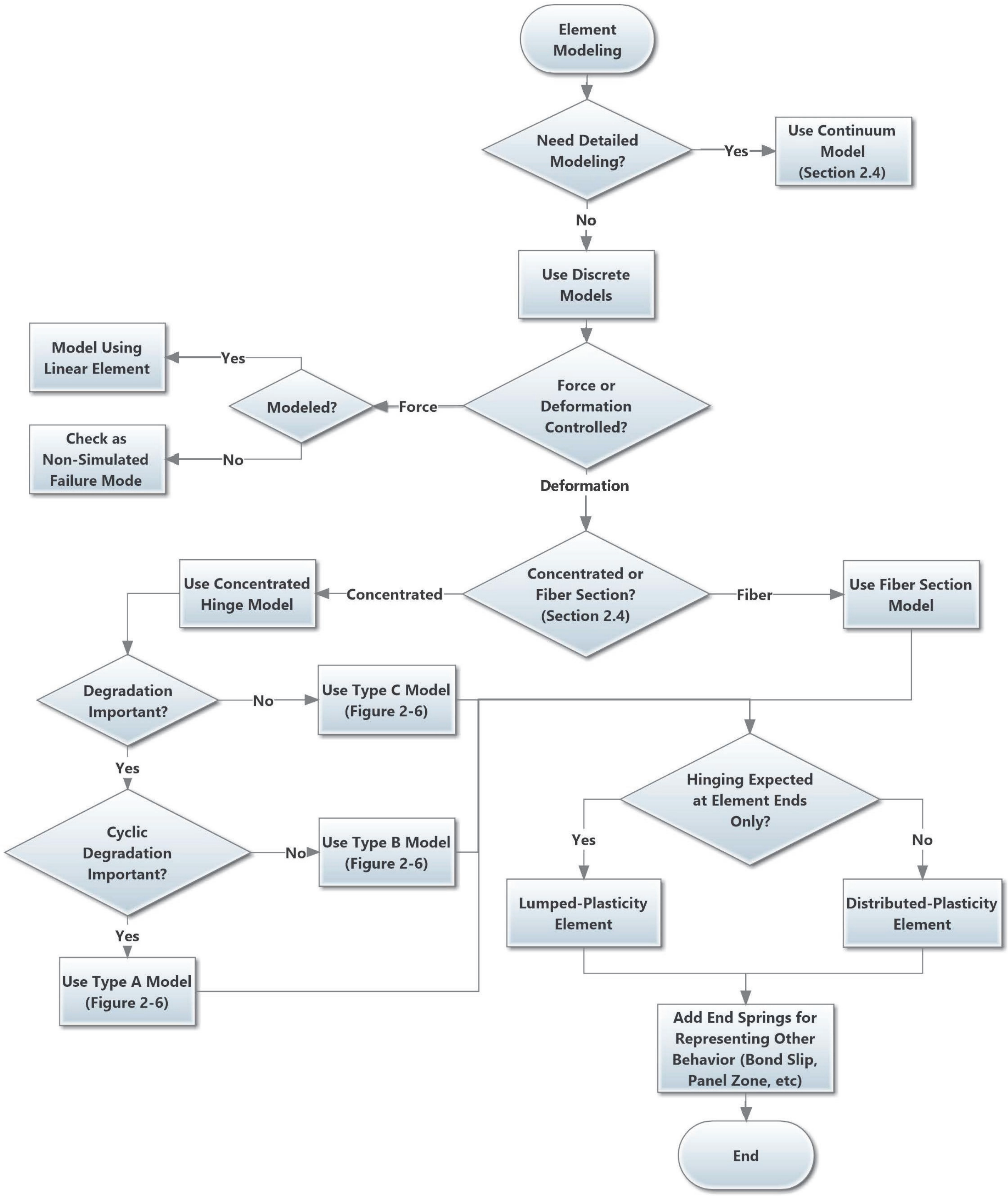


Figure 2-15 Flowchart aid for selecting appropriate nonlinear element and component models.

Chapter 3

General Nonlinear Structural Modeling Requirements

This chapter outlines specific modeling requirements and parameters that generally apply to structural systems of various materials and configurations. These criteria are intended to be accompanied by more detailed, system-specific requirements in Part II *Guidelines*. Section 3.1 provides general considerations and the sections that follow provide detailed guidance on selected topics.

3.1 General Modeling Guidelines

In general, the goal of a performance-based assessment using nonlinear dynamic analysis is to simulate the building response as realistically as possible to obtain an unbiased measure of its performance. This is in contrast to more traditional design approaches where simplified and generally conservative assumptions are made in analysis and design. Simplified assumptions exist in nonlinear analysis as well, but the intent is to make the analysis as unbiased and transparent as practical to achieve the desired results. Included below are some general guidelines and principles for developing realistic models.

- **Three-dimensional models.** Structural analysis models should be three-dimensional so they realistically simulate overall system response, including the spatial distribution of seismic mass and gravity loads and structural resistance to bi-directional and torsional response. A two-dimensional model can be used as a modeling check, but a three-dimensional model is typically necessary for the final design check.
- **Selection of components that significantly affect response.** All components that significantly affect the seismic responses of interest (i.e., those that are used in the acceptance criteria) should be included in the structural model. This is in contrast to more simplified building models that are commonly used for design by linear analysis, which would typically not include components that are not part of the seismic force-resisting system. However, the extent to which one can rely upon components that are not part of the primary lateral force-resisting system depends on whether: (1) the building code permits the contribution of secondary components to meet minimum acceptance criteria of the building code; and (2) the secondary components are designed, detailed, and constructed to ensure that they provide the resistance relied upon in the analysis. Conversely,

in the event that the secondary components can negatively impact building performance (e.g., by creating irregularities that induce torsion or weak stories), then their effect should be evaluated. Components and subsystems that should be considered include: (1) gravity framing (where typical pinned beam-column connections should be modeled as partially restrained rather than using a simplified pinned-end assumption); (2) stair systems or any stiff nonstructural wall systems; and (3) the substructure and foundations.

- **Geometry and finite member sizes.** To adequately capture both system-level response and local plastic deformation demands, the model should include finite joint and member sizes and properly select the locations of the components and plastic hinges.
- **Boundary conditions.** The boundary conditions to foundations and connected structures (where applicable) should be realistically modeled, rather than using highly idealized fixed or pinned assumptions. Column bases should be modeled as partially restrained, to reflect the flexibility of the connection between the column and the foundation, along with flexibility of the foundation itself. Where axial column forces due to overturning effects are large, vertical and rotational column base flexibility should be considered. Foundation modeling is discussed further in Section 3.9.
- **Model sophistication appropriate for level of demand.** The level of required modeling sophistication will depend on the demand parameters that are being calculated and the level of nonlinearity in the structural model. For example, an elastic analysis model is likely to suffice for determining peak floor accelerations for assessing contents damage for a Service Level Earthquake motion, where the maximum story drifts are on the order of 0.5% to 1%. In contrast, to assess collapse safety under Maximum Considered Earthquake motions, a nonlinear component model would be required, which adequately reflects the component response at the levels of plastic deformation calculated in the analyses.

3.2 Seismic Mass

All of the mass that is physically tied to the structure should be included in the analysis model. The mass of superimposed dead loads and certain types of live loads should be included in the seismic mass. Superimposed dead loads includes components such as the building façade, partition walls, ceiling and floor finishes, and mechanical and electrical equipment. Live loads that should be included in the effective mass include densely packed storage loads. Loose furniture and human occupants, which are included in the gravity live load are usually not included in the seismic mass. To reflect this and to comply with the Section 12.7.3 requirements of ASCE/SEI 7-16, *Minimum Design Loads and Associated Criteria for Buildings and*

Other Structures (ASCE, 2017), the seismically effective mass should be computed as the sum of the following three components:

- The full dead load, including all superimposed dead loads
- A minimum of 25% of the live load in areas used for storage
- The actual weight of moveable partitions, with a minimum of 10 lb/ft^2

In general, for most building structures where only horizontal components of ground motion are considered, it is sufficient to include only the horizontal components of mass, which should be defined to capture their realistic spatial distribution over the floor area. In some cases, such as for buildings with long span or cantilevered (flexible) floor or roof framing, it may be important to include the vertical mass, including any significant rotation of the mass about axes parallel to the ground.

3.3 Gravity Loads

The gravity loads should be specified as the expected gravity loads during an earthquake (a single load case). ASCE/SEI 7-16 states that these expected gravity loads shall be taken as $1.0D + 0.2L_0$ for the typical case and $1.0D + 0.4L_0$ when the live loads exceed 100 lb/ft^2, where L_0 is the unreduced design live load. The full gravity loads should be applied to the structural model and then the earthquake ground motions should be imposed with the loads in place. Note that the assumed live load effects in the gravity load combination is different from that in the seismic mass determination, since the two effects are related but involve different phenomena (vertical forces versus horizontal motions).

3.4 Geometric Nonlinearities

In a geometrically linear analysis, small deformations (or rotations) are assumed, which means that the deformed geometry is assumed to be the same as the original geometry for the purpose of computing the stiffness matrix. However, if the structure deforms significantly due to the imposed loads or when significant axial loads are present (e.g., gravity loads), the consideration of nonlinear geometry can be important. In particular, the P-delta effect, defined as the destabilizing force (or moment) exerted by gravity loads on vertical load carrying members due to lateral sidesway, can have a significant effect on the effective lateral stiffness, deformation, and stability of a structure.

There are two types of P-delta effects: P-Δ (capital Greek delta) which consists of the increase in overturning moment on a full frame or story due to the story deformation, Δ, and P-δ (lowercase Greek delta) which causes a magnification of moments within a member due to axial load eccentricity within the member caused by deviation of the deformed shape from the chord of the member. Both may lead to an apparent reduction in lateral stiffness and available flexural capacity of the member. In particular, the effect of P-Δ results in a reduction of lateral stiffness equal to P/L, where P and L are the member (or floor) axial load and height, respectively. For linear structures, this usually results in a corresponding reduction in stiffness and a proportional increase in lateral deformations. For yielding structures, however, this can reduce the post-yield stiffness, which may become negative. A negative post-yield stiffness has been shown to cause an amplification in the lateral dynamic response, similar to in-cycle strength degradation discussed in Section 2.3.2.1.

Structural analysis software generally use “second order” analysis methods to consider nonlinear geometry effects, though the rigor of those second order solution methods in representing nonlinear geometry effects can vary. At the most basic level, most structural analysis software generally support the ability to represent P-Δ effects in a simplified manner, whereby the stiffness matrix is based on the undeformed geometry but includes additional ($\pm P/L$) terms to represent the effect of axial forces on the stiffness. Other software have the ability to consider large deformations, where the geometric stiffness is recomposed at each time step based on the current deformed geometry. Additionally, P-Δ implementations may or may not consider P-δ effects. A more “rigorous” implementation may consider P-δ effects that cause amplification of moments due to deformations within the element. However, this is rarely supported by structure analysis software. Consequently, when P-δ effects are deemed to be important or when buckling of the member needs to be explicitly considered, this can be simulated by considering P-Δ in the analysis and breaking up the member into multiple elements (two to six elements depending on desired accuracy).

In general, it is recommended that the nonlinear analysis incorporate the best available representation of nonlinear geometry, which will depend on the software being used. To avoid numerical difficulties, the best approach is to incrementally include nonlinearities by starting with a linear model, then performing static linear analysis and examining the structure’s mode shape to identify any modeling errors. After the integrity of the model is verified, nonlinear geometry is added (first P-Δ and then large deformation), while debugging and examining the model for potential errors.

In order to accurately model P-Δ effects, the model should include the total gravity load including the gravity loads in the modeled lateral force-resisting elements, as well as the effect of gravity loads in non-modeled gravity members, because the P-Δ effect is proportional to the total load. Where the lateral frames are well distributed, the gravity mass tributary to the frame and the seismic mass that must be resisted for lateral motion are usually similar. However, if the lateral force elements are not highly redundant, the addition of P-Δ effects require special modeling. Traditionally, this has been accomplished by adding a single vertical truss element for each story to the model, a "P-Δ leaning column," further discussed in Section 3.11.1. At each story, all gravity loads that were not applied to the lateral system directly are applied to the P-Δ column. Since the truss elements have no stiffness and are unstable, it is necessary to "slave" the truss nodes at each story back to the story master node (assuming a rigid diaphragm). Although this method may be sufficient in general, in many cases it is important to accurately capture the spatial distribution of P-Δ loads, in addition to their magnitude. Recent research has shown that correct modeling of the location of P-Δ loads is important to accurately capture torsional P-Δ effects in nonlinear response history analysis (Flores et al., 2014).

3.5 Material Properties

Nonlinear models and criteria should be unbiased, implying that they should be chosen to represent median response (central tendency) of the structural system. Typically, this is achieved by basing the model parameters and criteria on expected properties of the materials and components. As noted in NIST GCR 10-917-5, *Nonlinear Structural Analysis for Seismic Design: A Guide for Practicing Engineers* (NIST, 2010), the expected properties are generally reported as mean properties, although given limited data and modest levels of uncertainty, the mean and median properties of the input parameters are generally assumed to be indistinguishable. Where there are concerns about modeling uncertainty, variability in the model parameters can be incorporated in sensitivity analyses, as described in Appendix B.

In the absence of project-specific data, expected properties for modeling of common steel construction materials are given in Table 3-1, which is based on published criteria in ANSI/AISC 341-05, *Seismic Provisions for Structural Steel Buildings* (AISC, 2005), ACI 318-14, *Building Code Requirements for Structural Concrete and Commentary* (ACI, 2014), ASCE/SEI 41-13, *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE, 2014), *An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region* (LATBSDC, 2015), and *Guidelines for Performance-Based Seismic Design of Tall Buildings* (PEER, 2017). These material properties should be used in conjunction with the system-specific *Part II Guidelines* to determine the expected stiffness, strength, and inelastic response of structural components. The expected strength values of Table 3-1 do not account for strain rate effects, which can lead to increases in yield and ultimate strengths during seismic loading.

For concrete material properties, the expected strength of 10,000 psi or larger may not warrant a 1.3 multiplier. Check with the local concrete suppliers to get a refined multiplier. In addition to the expected strength of concrete, the concrete's modulus of elasticity (MOE) is an important parameter to accurately estimate. Codes, standards and guidelines (e.g., ACI and LATBSDC) provide recommendations for calculating the MOE. Where regional equations have been developed based on actual concrete measurements, those equations should be used. Prior to finalizing the MOE used in the nonlinear analysis, it is suggested that data from local concrete suppliers be evaluated to help set the value.

Table 3-1 Expected Steel Material Strengths

Material	Expected Strength
Structural Steel – Hot Rolled Structural Shapes and Bars	
ASTM A36/A36M	$1.5F_y$
ASTM A1043/1043M Gr. 36 (250)	$1.3F_y$
ASTM A992/A992M	$1.1F_y$
ASTM A572/572M Gr. 50 (345) or 55 (380)	$1.1F_y$
ASTM A913/A913M Gr. 50 (345), 60 (415), 65 (450), or 70 (485)	$1.1F_y$
Structural Steel – Hollow Structural Sections (HSS)	
ASTM A500/A500M Gr. B	$1.4F_y$
ASTM A500/A500M Gr. C	$1.3F_y$
ASTM A53/A53M	$1.6F_y$
ASTM A1085/A1085M	$1.2F_y$
Structural Steel – Plates, Strips, and Sheets	
ASTM A36/A36M	$1.3F_y$
ASTM A572/A572M Gr. 42 (290)	$1.3F_y$
ASTM A572/A572M Gr. 50 (345), Gr. 55 (380)	$1.1F_y$
ASTM A588/A588M	$1.1F_y$
Reinforcing Steel	
ASTM A615/A615M Gr. 60 (420)	$1.2F_y$
ASTM A615/A615M Gr. 75 (520) and Gr. 80 (550)	$1.1F_y$
ASTM A706/A706M Gr. 60 (420) and Gr. 80 (550)	$1.2F_y$

Other materials that are not listed here may be used, and their material strengths can be found in the reference documents and in ASCE/SEI 41-13.

3.6 Floor Diaphragms and Collectors

Diaphragms serve multiple roles, including:

- Provide lateral support to vertical elements and cladding
- Resist forces from inclined columns
- Transfer inertial forces to the seismic force-resisting system
- Transfer forces among vertical elements of the seismic force-resisting system
- Resist reactions from retaining walls

Idealizations of the diaphragms included in the structural model should be constructed considering all of their roles. The roles listed above relate to in-plane deformations and forces. Diaphragms also resist gravity loads, demands caused by vertical accelerations, and wind uplift in roofs. Slabs can also affect the response of beams built integrally with them and connections between slabs and columns or walls can be critical.

3.6.1 Diaphragm Idealization

For design purposes, diaphragms are often idealized as flexible, rigid (stiff), or semi-rigid (see Chapter 12 of ASCE/SEI 7-16 for limitations on flexible and rigid diaphragms). However, in three-dimensional computer models, diaphragms must either be modeled with some finite amount of stiffness and, therefore, are usually modeled as either perfectly rigid (with kinematic constraints) or semi-rigid (with some type of finite elements).

Detailed semi-rigid modeling of diaphragms can help assess: (1) force transfer among elements of the lateral force-resisting system; (2) the effects of openings; and (3) effects of irregularities. Idealizing diaphragms as rigid is computationally advantageous; however, where irregularities in the building occur, this assumption can result in both unreasonable diaphragm force transfer and lateral load redistribution among vertical elements of the seismic force-resisting system. Common irregular conditions in which semi-rigid modeling of diaphragms is recommended include buildings with torsional irregularities, reentrant corners, diaphragm discontinuity irregularity, out-of-plane offsets, soft story irregularity, mass irregularity, vertical geometric irregularity, in-plane offset, and/or weak story irregularity.

Ground motions are usually selected and scaled focusing on the fundamental period of the entire building. However, peak diaphragm demands may be controlled by higher vibration modes that are excited by higher frequencies. Ground motion selection should address this issue if accelerations from response history analysis are used to supplement floor diaphragm designs.

3.6.2 Diaphragm Stiffness

In-plane diaphragm stiffness of concrete slab and concrete over metal deck floors is often modeled initially as rigid to generate preliminary results and later modeled as semi-rigid as results of analysis or irregularities warrant (further discussed in Section 3.6.4). Where slab outrigger effects are present (e.g., participation of gravity columns, mobilized by slab flexure, in resisting overturning effects), then the out-of-plane diaphragm stiffness can influence force demands in vertical elements in the seismic force-resisting system. For example, the shear force developed in shear walls or the brace forces in braced frames may increase if the gravity system resists overturning (global flexural) response of the seismic force-resisting system.

For elastic diaphragms, stiffness is calculated using expected material properties and reduced to reflect an effective stiffness (i.e., secant stiffness) based on the expected deformations and cracking. Recommended effective stiffnesses for several levels of demand are listed in Table 3-2 (based on PEER (2017), Section 6.6.3.2.2 of ACI 381-14, and ASCE/SEI 41-13). The range of effective stiffnesses is provided as a guideline to evaluate the effect of stiff and soft diaphragm conditions on force demands in the vertical seismic force-resisting system and foundations.

Table 3-2 Recommended Effective Stiffness Values for Diaphragms

Component	Service Level Earthquake	Design Earthquake	MCE_R
Concrete and Concrete over Metal Deck			
Out-of-Plane [Plate] (%E_c)	35 – 50%	25 – 35%	25– 35%
In-Plane [Membrane]* (%E_c)	30 – 40%	10 – 30%	5 – 30%
Untopped Metal Deck			
Out-of-Plane [Plate]	**	**	**
In-Plane [Membrane]	***	***	***

*: Range represents secant stiffness at shear stresses from $2\sqrt{f'_c}$ to $8\sqrt{f'_c}$

**: Out-of-Plane stiffness is generally negligible and effective values are not reported

***: In-plane stiffness is dependent on connection type and expected damage as discussed in Essa et al. 2003.

Untopped metal deck and wood sheathed floors are often modeled as semi-rigid diaphragms. Diaphragm stiffness for these floor types is obtained from one of the following: (1) a so-called flexibility factor, F, that is calculated per the *Diaphragm Design Manual* (SDI, 2015); (2) calculated through expected diaphragm deformations determined from ANSI/AWC SDPWS-2015, *Special Design Provisions for Wind and Seismic* (AWC, 2015); (3) published data by deck manufacturers which is determined through testing or the *Diaphragm Design Manual*; or (4) diaphragm testing. The flexibility factor, F, is typically specified in units of micro-inch/lb or 10^{-6}in/lb. For untopped metal deck, the flexibility factor can be converted into an equivalent membrane thickness, t_{equiv}, to be used in the analysis, where $t_{equiv} = 10^6/(1000 \times F \times G) = 10^3/(F \times G)$ with the shear modulus, G as specified in the analysis (assuming ksi units) and F as specified by the deck manufacturer (in units of micro-inch/lb). Diaphragm stiffness calculated by a flexibility factor represents the initial stiffness of an untopped metal deck diaphragm. For wood sheathed floors, analysis should confirm that under expected force levels, diaphragm deflections match those calculated from ANSI/AWC SDPWS-2015, or relevant test results.

3.6.3 Diaphragm Behavior

In-plane shear action in diaphragms is typically considered to be a force-controlled action, although depending on the situation some limited inelastic shear deformation is permitted. The distinction is usually made by deeming the in-plane shear action to be either an "ordinary" or "critical" force-controlled action, where the acceptance criteria for "ordinary" allows for some inelastic behavior (e.g., based on average force demands under MCE-level ground motions). In either case, the diaphragms are typically modeled as either rigid or essentially elastic components.

Shear behavior in concrete slab and composite steel deck with concrete infill can be designed to allow some inelastic response provided that diaphragm connections do not limit the diaphragm strength (which it typically does not). Concrete and reinforcing steel transfer loads throughout the diaphragm to vertical members of the lateral force resisting system. Shear capacity in the field of the diaphragm is based on design strength of the concrete or concrete above deck flutes and reinforcing steel, $\phi V_n = \phi(V_c + V_s)$ per ACI 318 Chapter 21. Alternatively, for steel deck with concrete infill, allowable shear capacities from published manufacturer data can be converted to a design shear capacity by increasing allowable capacity by the reported factor of safety and then applying the appropriate ACI 318 ϕ factor. Composite flexural action is permitted per ANSI/AISC 360 in beams that also serve as chords and collectors. Moreover, in the absence of research demonstrating otherwise, the ANSI/AISC 360 commentary states that it is not required to superimpose horizontal shear due to lateral forces with the horizontal shear flow associated with composite beam flexure under gravity loads. Whereas shear studs and other welded attachments are usually prohibited in plastic hinge regions, ANSI/AISC 341 includes an exception for permitting shear connectors within the protected zone for several prequalified connections. Connection capacity of welded connections is calculated per AWS D1.3, *Structural Welding Code-Sheet Steel* (AWS, 2008), and headed stud connectors per ANSI/AISC 360-10, *Specification for Structural Steel Buildings* (AISC, 2010b). It is not recommended that connection capacity govern the behavior of the diaphragm, therefore, supplemental dowels, connections or headed studs should be added to the diaphragm to promote distributed inelastic deformations if the diaphragm strength is exceeded.

Shear behavior in untopped metal deck is generally a "critical" force-controlled action as failure occurs at fastener locations (Rogers and Tremblay, 2003; Essa et al., 2003). Shear capacity may be calculated per SDI DDM04, *Diaphragm Design Manual* (SDI, 2015), obtained from published manufacturer's test reports or testing. In cases where bare metal deck does not provide sufficient strength, concrete fill or in-plane bracing may be added to increase overall diaphragm strength. Additional options for addressing insufficient diaphragm strength are described in the following section.

Shear behavior in plywood diaphragms is typically treated as an “ordinary” force-controlled action. Shear capacity in the field of the diaphragm is based on SDPWS.

3.6.4 Diaphragms at Irregularities

Force transfer caused by horizontal or vertical irregularities may not be concentrated at a single floor (Figure 3-1 shows concentrated force transfer), and may occur over several diaphragm levels (Figure 3-2 shows distributed force transfer). Diaphragms away from transfer levels may be analyzed as rigid if the lateral distribution of forces and force in vertical resisting elements do not significantly change when compared to a baseline model with semi-rigid diaphragms. Additionally, semi-rigid diaphragm modeling is suggested where significant forces are transferred “into” the diaphragm from a vertical member, or line of members. A range of 10-15% is considered a significant change or force transfer in the above cases for typical building design. The purpose of a range is to ensure that analytical diaphragm stiffness assumptions are not protecting vertical lateral load resisting elements or the foundation. As vertical members soften, rigid diaphragms are capable of transferring loads to stiffer members connected to the diaphragm and in the process, decrease force-deformation demands on the softening members.

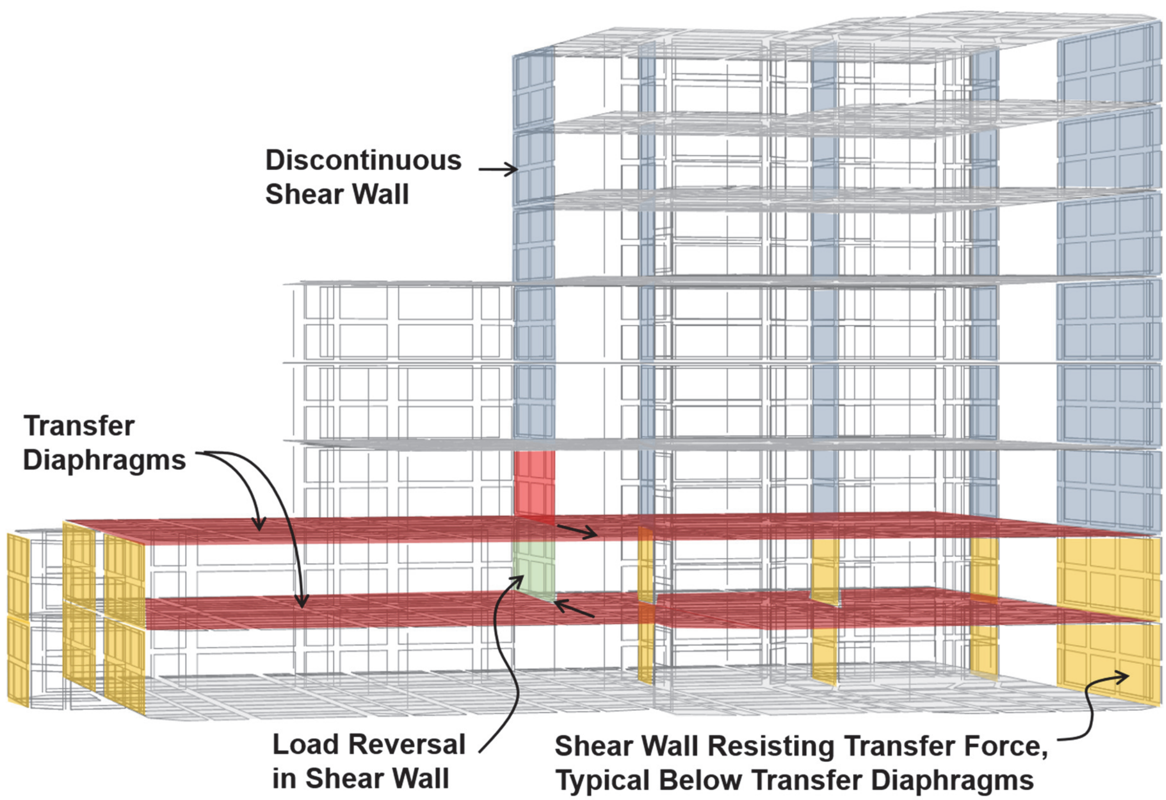


Figure 3-1 Concentrated force transfer out of discontinuous wall in a rigid diaphragm.

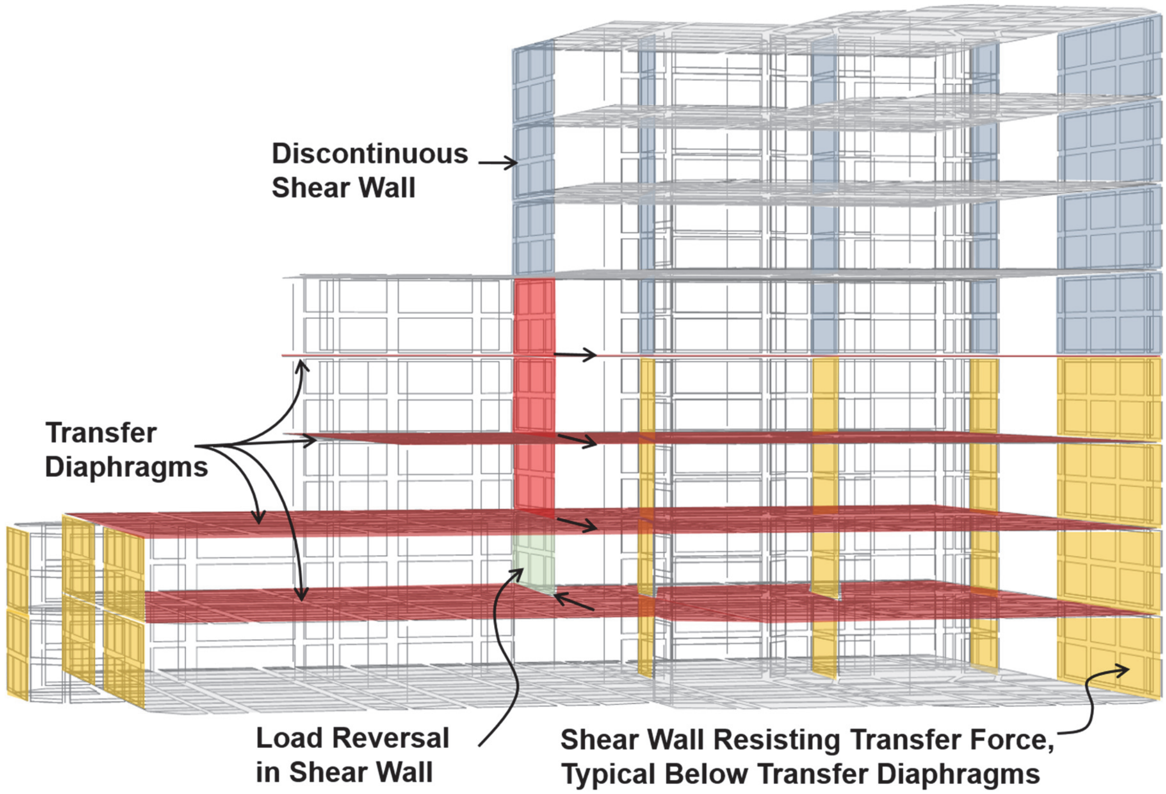


Figure 3-2 Distributed force transfer out of discontinuous wall in a semi-rigid diaphragm.

Typically, at a diaphragm discontinuity irregularity, a bounding analysis is performed using the range of stiffness shown in Table 3-2. Higher effective stiffness values represent the case when loads in lateral force-resisting members above the diaphragm discontinuity can transfer loads through the diaphragm into other seismic force-resisting members below the diaphragm. Lower values of effective stiffness represent the case when the diaphragm is damaged and load transfer into the diaphragm from the seismic force-resisting members above the discontinuous diaphragm is reduced. The severity of this reduction is dependent on the expected cracking in the diaphragm. Seismic force-resisting members must transfer the remaining portion of shear not transferred into the diaphragm to a lower diaphragm or to the foundation.

Discrete truss members may be used to supplement or provide an alternate load path in a diaphragm to mitigate the effect of irregularities. Diaphragms with truss members should be modeled as semi-rigid. A bounding analysis varying the stiffness of the diaphragm should be performed to determine the effect on truss member forces and their connection forces. Depending on the type of irregularity and geometry of the diaphragm, a softer or stiffer diaphragm may result in larger forces in truss members in different areas of a diaphragm. Diaphragm truss members may be designed as either ordinary or special concentric braced frame members.

3.6.5 Diaphragm Force Output and Meshing

For a rigid or semi-rigid diaphragm, it is recommended to estimate force transfer into vertical elements by summing up forces through section cuts of the vertical elements above and below each diaphragm. For semi-rigid diaphragms, if diaphragm shears are to be calculated from the analysis, it is suggested a baseline mesh size of $^1/_{10}$ to $^1/_5$ of the bay or wall length is modeled (Moehle, 2015). Such a mesh may be coarsened if a sensitivity study confirms the difference in baseline diaphragm shear is not significantly different. Section cuts through the diaphragm can be used to determine average stresses for evaluating diaphragm adequacy, rather than using local stresses from individual diaphragm elements.

3.6.6 Chords and Collectors

In some regular structures, collector and chord elements can increase the effective flexural stiffness of the diaphragm and should be included in the analysis. The inclusion of collector and chord elements may also stiffen the structure globally and increase modal mass participation of the fundamental modes. An increase in global stiffness may increase base shear demands for the building as the structural period decreases. For buildings with horizontal irregularities, collector and chord elements should be included in the analysis to tie discontinuous or cantilevered areas of the diaphragm to the main diaphragm (as shown in Figure 3-3). Chord and collector element stiffness should match the stiffness assumed for similar beam-column elements modeled in the analysis.

The chord and collector elements, along with their connections, should be checked according to the acceptance criteria (see Chapter 6) using forces and/or deformations from the nonlinear analysis. Chord and collector reinforcement in concrete diaphragms should be anchored and spliced to develop their full tensile strength or a stress exceeding f_y by a margin consistent with ACI 318. Chapter 4 of AISC *Seismic Design Manual* (AISC, 2010a) provides a detailed connection example for a steel collector beam. It is not uncommon for the nonlinear analysis check for chords and collector elements under MCE-level ground motions to require increases in beam sizes or connections above those determined based on standard (prescriptive) design checks. Generally, chord and collector element actions are classified as force-controlled components, which are modeled as essentially elastic, although in certain cases, they may be treated as deformation-controlled. The next section describes the modeling approach to be taken if the diaphragm is deemed as deformation-controlled and nonlinearity is allowed in the diaphragm.

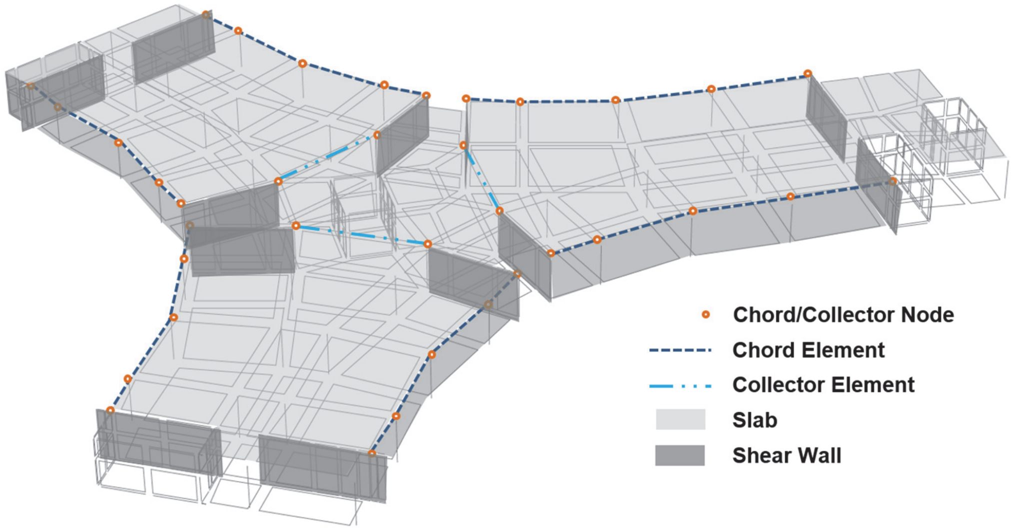


Figure 3-3 Explicit modeling of chord and collector elements.

3.6.7 Nonlinear Diaphragm Modeling

Most diaphragms are proportioned to remain elastic such that inelasticity takes place in vertical elements of the lateral force-resisting system, and, therefore, are considered to be force-controlled components. However, in some structures, such as low-rise buildings with stiff vertical elements and large wood diaphragms or existing buildings, a significant portion of global inelasticity takes place within the seismic force-resisting system. When nonlinear behavior of the diaphragms is expected to have a significant effect on the overall behavior of the building, diaphragm can be deemed as deformation-controlled and the elements can be modeled as inelastic. A force-deformation relationship should be developed based on diaphragm material and connections to vertical elements of the seismic force-resisting system. Chapter 10 of ASCE/SEI 41-17 provides backbones and acceptance criteria for concrete shear walls that can serve as a guide for an appropriate model for a nonlinear concrete diaphragm. A fiber section curvature analysis should justify the backbones for the slab. Chapters 9 and 10 of ASCE/SEI 41-17 provide backbones and acceptance criteria for steel concentrically braced frame members that can serve as a guide for an appropriate model for nonlinear truss elements. Backbones and acceptance criteria for special concentrically-braced frames should consider deformation compatibility with the floor diaphragm. Chapter 9 and Chapter 12 of ASCE/SEI 41-17 provide backbones and acceptance criteria for bare metal deck and wood diaphragms, respectively.

Although generally not recommended, where necessary in evaluating existing buildings, untopped metal deck can be modeled with brittle force-deformation behavior of the bare steel deck fasteners in the analysis. Alternatively, elements with demands exceeding their capacity can be removed from the analysis with the remaining elements demonstrating that they have adequate capacity acting as an alternative load path. Elements in plywood diaphragms with demands exceeding their capacity can also be removed in a similar fashion. Loss of diaphragm connection should not result in a loss of gravity carrying capacity of the floor. Both lateral and gravity framing and connections should also be checked for the additional forces that may arise from an alternative load path resulting from the removal of elements in the analysis.

3.6.8 Diaphragm Design

Diaphragms are generally evaluated and designed to be code-compliant including overstrength factor requirements. Force transfer demands from the analysis are complementary to the code-based design forces and should not be used to justify reductions in code-based forces. For more information, refer to: (1) NIST GCR 11-917-10, *Seismic Design of Composite Steel Deck and Concrete-Filled Diaphragms* (NIST, 2011a); (2) NIST GCR 16-917-42, *Seismic Design of Cast-in-Place Concrete Diaphragms, Chords, and Collectors* (NIST, 2016); (3) NIST GCR 10-917-5, *Nonlinear Structural Analysis for Seismic Design* (NIST, 2010); and (4) *2012 IBC SEAOC Structural/Seismic Design Manual, Volume 4 Examples for Steel-Framed Buildings* (SEAOC, 2013).

3.7 Modeling of Damping

Viscous damping (velocity-dependent force) is commonly used to represent energy dissipation that is not otherwise modeled in the components of the structure. Mathematically, viscous damping effects are the forces associated with the velocity-proportional $[C]\{\dot{x}\}$ term in the governing equation of motion of the system and its components. For this purpose, the representation of energy dissipation by velocity-dependent forces is a mathematical convenience. As such, the determination of viscous damping coefficients is not derivable, but rather, is based on a combination of empirical data from building measurements, tests, and analyses, along with judgment. Moreover, owing to the empirical nature of the assumed damping, the differences between alternative damping implementations (e.g., Rayleigh versus modal damping) is also a matter of judgment, which can depend on the specific aspects of the analysis software and modeling approach.

The purpose of this chapter is to provide a summary of the underlying phenomena involved with damping, a description of the mathematical approaches to model damping and some general guidelines of the range of values for damping. The user, however, is directed to the literature and to the recommendations provided by analysis software developers to determine the best model to employ in the simulation. Because there are numerous variables involved with damping, it is also recommended to perform a sensitivity study to determine the effects of variations of damping on the overall numerical response of the system being evaluated. In general, elastic dynamic response is more sensitive to the damping assigned than inelastic displacement response. This is because at large inelastic deformation levels, the magnitude of the energy characterized by hysteretic damping is significantly larger than that of viscous damping.

There are many sources of damping, such as friction, in a building system. The contribution of the different sources of damping to the total energy dissipation depends on the amplitude of the deformations of the structural system. Studies have shown that, while many factors contribute to the total energy dissipation at low levels of excitation, when the structure is responding in the elastic range at higher levels of excitation, the hysteretic damping from regions of the structure responding inelastically contribute the most energy dissipation in the system. Table 3-3 provides a summary of components in the building system that may dissipate energy, including the structural frame, foundation, and nonstructural components (such as architectural partitions and cladding). These sources of damping are most significant in a service-level analysis, when the structural response is essentially elastic. The table also indicates whether energy dissipation in these components is typically modeled explicitly or as equivalent viscous damping (EVD), each of which are defined as follows:

- **Explicit.** Energy dissipation is represented by hysteretic response of elements that are introduced to represent components of the structure, foundations, partitions, cladding or other elements. Where manufactured viscous dampers (e.g., hydraulic pistons or other devices) are used to dissipate energy, the modeling of these is usually done directly through an explicit damper element in the matrix, where the damping parameters are calibrated to test data or other information about the damping device.
- **Equivalent Viscous Damping (EVD).** Energy dissipation is represented by an equivalent velocity-dependent force that tends to be out-of-phase with the motion of the structure and, thus, to reduce or dampen the motions. Mathematically, EVD is generally represented by terms in the [C] matrix that are derived from Rayleigh (proportional to mass and/or stiffness) damping, modal damping, or other discrete damping terms (e.g., described through fictitious dashpots connecting specific degrees of freedom).

Largely for convenience and due to features available in most computer software, viscous damping is most often modeled using Rayleigh (mass and/or stiffness proportional) damping, modal damping, or a combination of the two. The choice between the two may relate to the specifics of the analysis software used, although there is a growing evidence to support the use of modal damping over Rayleigh damping for nonlinear analysis (e.g., Cruz and Miranda, 2016). However, even where modal damping is used, a small amount of Rayleigh damping tends to improve computational convergence, while providing some damping to vibration modes whose periods elongate with inelastic deformations.

Table 3-3 Sources of Energy Dissipation in Building Systems

Phenomena	How is it Usually Represented
Superstructure	
Yielding of members in lateral force-resisting system	Explicitly modeled although onset of yielding may not be fully captured
Yielding of members in secondary gravity system	Explicitly modeled (like lateral force resisting system) or through EVD
Cracking of floor slab due to beam flexure and differential uplift in tall buildings (with significant column or wall elongation)	EVD or explicitly in the beam or slab models
Beam/column yielding and floor slab cracking in gravity framing connections	EVD or explicit hinge models
Beam-column joint panel yielding	EVD or explicit spring models
Localized yielding and/or bolt slippage or bond slip in connections	EVD or explicit hinge models
Foundation (Substructure)	
Bearing, rotation, and possible uplift of spread footings or mat foundation	EVD or explicit foundation hysteretic springs and dashpots, or both
Axial tension/compression deformations and lateral bearing of piles	EVD or explicit foundation hysteretic springs and dashpots, or both
Lateral bearing and gap opening against footings, mat, pile caps, and/or basement walls	EVD or explicit foundation hysteretic springs and dashpots, or both
Localized reinforcing steel yielding and concrete cracking in foundation components	EVD (unless explicitly modeled)
Boundary interactions between the foundation and soil	EVD (unless explicitly modeled)
Nonstructural Components	
Racking and flexure of exterior cladding	EVD
Racking of full-height partition, stairwell, and elevator walls	EVD
Racking of partial-height partition walls	EVD
Racking of stair systems	EVD
Dynamic interaction with large mechanical equipment	EVD
Interaction with HVAC, mechanical and electrical risers	EVD

Specific considerations related to the specification of Rayleigh Damping (mass and stiffness proportional; $C = \alpha M + \beta K$) are as follows:

- Generally, the two damping constants are selected to specify the percentage of critical damping at two vibration frequencies that encompass the predominant response. When expressed in terms of vibration period, values of T_1 (first-mode) and $0.1T_1$ (third or higher mode period) are commonly used to define the damping coefficients.

- Stiffness proportional damping is usually specified as constant, based on the elastic stiffness; however, variable stiffness proportional damping, based on the tangent or secant stiffness, is also possible for some software programs. Use of tangent stiffness will reduce the damping as the structure yields and softens. Whether this type model is appropriate or not is still to be determined.
- Mass-proportional damping tends to overdamp the low-frequency response. Stiffness-proportional damping tends to overdamp the high-frequency response.
- Where rigid elements are idealized with large stiffness (such as to simulate a "rigid-plastic" condition), it may be necessary to remove the stiffness proportional damping terms associated with these to avoid large unbalanced damping forces. This feature is generally one that needs to be built into the software implementation and is something to check for.

Modal damping can avoid some of the shortcomings of Rayleigh damping by providing a constant, specified value of damping in each desired vibration mode. However, although damping values can be more precisely specified, the specification of viscous damping as a function of elastic vibration modes is still an approximation of the actual energy dissipation mechanisms during inelastic response. Modal damping is typically used to represent the damping in the elastic components and is used to dampen the high-frequency modes.

Given the inherent uncertainties in the appropriate amount of damping and how it is modeled, it is suggested to assess the sensitivity of the calculated demand parameters to the assumed damping parameters. In the absence of more detailed analysis, it is recommended to limit the fraction of critical damping to the following:

$$\zeta_{critical} = \frac{0.36}{\sqrt{H}} \leq 0.05 \tag{3-1}$$

where H is the height of the structure (roof level) above grade in feet (see Figure 3-4). The suggested damping is based on recommendations in PEER/ATC-72-1 and more recent research publications (Cruz and Miranda, 2016; Bernal et al., 2015), which provide evidence from measured building data. However, given that the data supporting Equation 3-1 (and Figure 3-4) are from building response at relatively low story drift ratios (less than about 0.5% story drift ratios), some provisions may allow use of higher critical damping ratios under MCE-level ground motions (e.g., PEER 2017 and LATBSDC 2015 permit damping ratios of 0.025 for tall buildings).

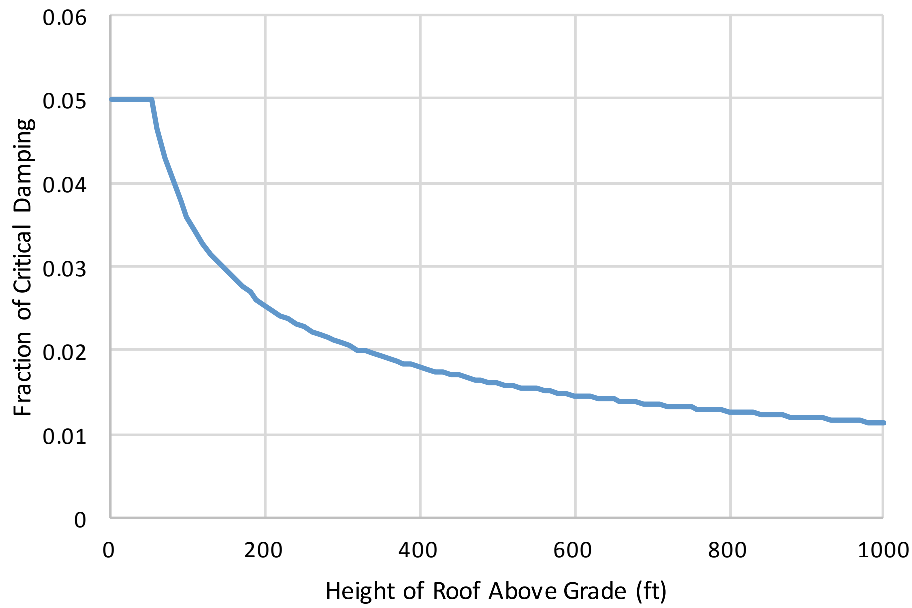


Figure 3-4 Equivalent viscous damping ratio versus building height.

3.8 Torsion

Two types of torsional behavior need to be considered in the analysis model: inherent torsion and accidental torsion.

Inherent torsion is torsion caused by differences in the location of the structure's center of mass and center of rigidity at each level of the building and is automatically included in the three-dimensional analytical model through appropriate distribution of the mass and assignment of element properties. When structural yielding occurs, the center of rigidity changes, resulting in changes to the building response that will be captured in the nonlinear dynamic analysis.

Accidental torsion, on the other hand, is a user-defined offset in the expected location of the mass and is intended to account for uncertainties in mass, stiffness, and strength distribution, to identify buildings with limited torsional resistance and to account for torsional components of ground motion. This offset in the location of the mass is typically defined as a fixed distance in each direction from its expected location. One way to model this offset, or accidental eccentricity, is through the use of a series of masses distributed in the horizontal plane to achieve the prescribed eccentricity.

There are a wide variety of viewpoints in research, guidelines, and standards regarding the inclusion of accidental torsion in nonlinear analysis. ASCE/SEI 7-16 requires the inclusion of accidental torsion for structures with Type 1a or 1b horizontal irregularities as defined in Section 12.3.2.1 of ASCE/SEI 7-16. Accidental torsion can be included by offsetting the center of mass each from its actual location by a distance equal to 5% of the diaphragm dimension of the structure parallel to the direction of mass. The required 5% displacement of the center of mass need not be applied in both orthogonal directions at the same time. When displacing the mass to consider accidental torsion, it is usually done so as to exaggerate any inherent torsion irregularity in the building.

Additional discussion regarding the inclusion of accidental torsion in nonlinear analysis for tall buildings can be found in PEER (2010) and LATBSDC (2015).

3.9 Foundation Modeling

For certain structures, modeling of the subgrade levels and foundation system can affect response of the superstructure depending on the number of subgrade levels, type of foundation system, and subgrade response. The rocking and sliding behavior of foundations can not only affect superstructure response, but should also be evaluated for acceptable behavior. In general, sensitivity to modeling foundations varies substantially depending on both the foundation and superstructure response, and therefore the best way to assess impacts is to perform a sensitivity analysis. The following describes methods to characterize foundation behavior in a nonlinear analysis model.

Determination of appropriate foundation modeling parameters must be developed in conjunction with soil-structure-interaction considerations. Refer to Section 3.10 for additional guidance and discussion on soil-structure-interaction.

3.9.1 General Methods for Modeling Foundation Systems

Given the complexity of foundation and subgrade response, modeling must include some simplifications that still include the representative behavior necessary to envelope superstructure response. For structures with multiple levels of subgrade floors and a transfer diaphragm at grade, it is often necessary to compare a fixed-base model to a model with foundation flexibility to envelope backstay effects.

Foundation systems may be modeled with a combination of vertical springs, horizontal springs, and dashpots to represent foundation stiffness and damping characteristics. Vertical springs representing vertical foundation response are most common in practice and in general can capture foundation deformation and rocking behavior to adequately determine superstructure response. Horizontal springs representing lateral resistance and dashpots representing damping effects on the sides of basement walls or foundation systems can be considered but are often conservatively ignored.

Although it is technically possible to create a model that includes the response of many subgrade layers in conjunction with the foundation system stiffness, it is typically not feasible to model to this level of complexity. Therefore, the use of discrete foundation springs to characterize behavior representative of multiple foundation springs and behavior is a useful simplification in order to create a model that is feasible to implement but still captures the critical superstructure response. Figure 3-5 provides an example of a core wall supported on mat foundation where the nonlinear response of the mat foundation is correlated to the response of a discrete spring at the corner of the core wall.

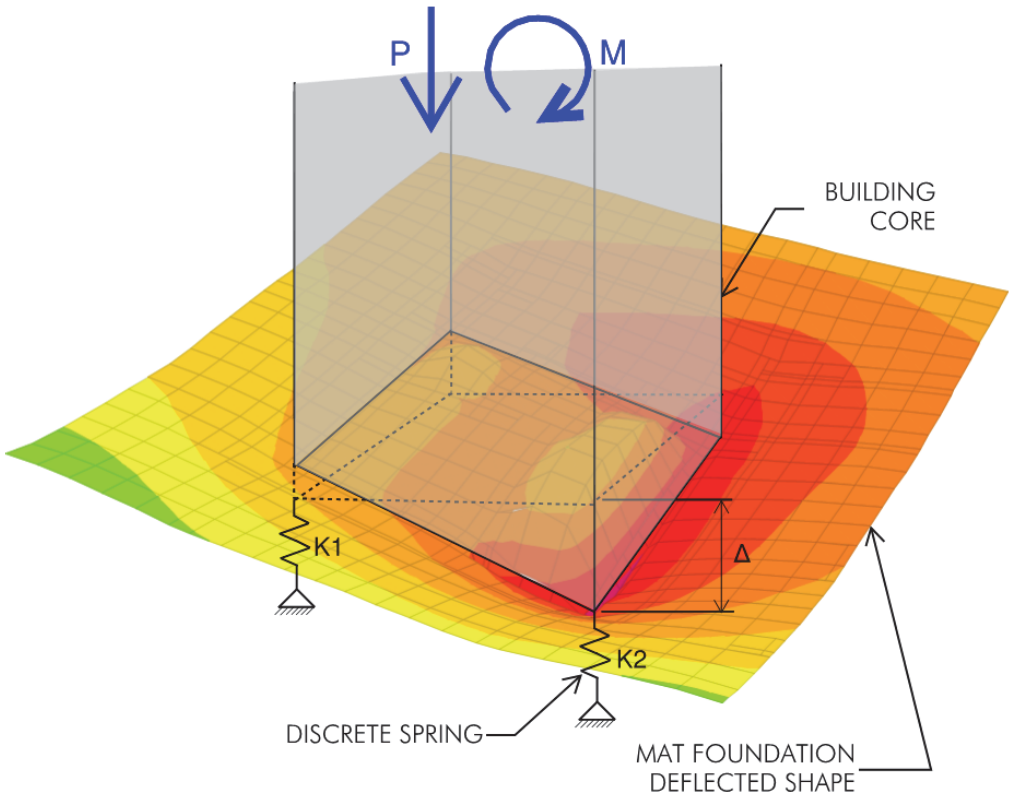


Figure 3-5 Shear wall on mat foundation showing overturning applied with discrete springs at wall corners to represent the mat/subgrade response.

3.9.2 Shallow Foundation Systems

Shallow foundation systems are commonly modeled as discrete springs as described above. To determine representative discrete springs for a shallow foundation system, a separate foundation analysis model should be used. For the example of a mat foundation supporting a shear wall, a separate mat foundation analysis model would be needed to determine the discrete spring values. The mat model should include soil response by using a subgrade modulus determined in conjunction with the geotechnical engineer, no-tension iteration to establish equilibrium with only soil compression, and mat foundation deformations associated with the mat and superstructure flexural and shear stiffness. By applying gravity load and overturning to the mat foundation model consistent with the range of loading anticipated in the nonlinear analysis model, deformation under the key points of the shear wall can be determined and converted to a discrete spring value to be implemented into the nonlinear analysis model.

3.9.3 Deep Foundation Systems

Similar to shallow foundation systems, the response of a deep foundation system can be estimated by representative discrete springs under key corners of the superstructure. A separate foundation analysis model to determine these discrete springs for a deep foundation system would need to incorporate both the vertical and lateral response of the deep foundation elements. This foundation model needs to include the response of the deep foundation element itself, subgrade friction, and subgrade bearing. This response should be determined in conjunction with a geotechnical engineer and with a separate analysis model including appropriate soil springs around foundations. Deformation under the key points of the superstructure can be determined with this model and converted to a discrete spring model to be implemented into the nonlinear analysis model.

3.9.4 Sensitivity Analysis

It should be recognized that determination of foundation and soil response is an estimate based on the limitation of subgrade variances, geotechnical testing, soil response, and other parameters. Section 6.6 recommends sensitivity analysis to be performed for upper- and lower-bound parameter values, to not only validate the analysis model, but also to envelope the response.

3.9.5 Foundation Modeling Implementation

Where discrete springs are used to characterize foundation response, it may be convenient to kinematically constrain the base points of the supported superstructure and then attach the discrete springs as shown in Figure 3-6. This approach then permits the evaluation of foundation response separate from the superstructure response. Provided that the discrete springs are calibrated for deformations consistent with those anticipated in both the superstructure and foundation analysis models, this implementation can provide a reasonably accurate response.

3.9.6 Foundation Acceptance Criteria

For the reasons outlined above, the foundation system is commonly implemented into the model with a system of discrete springs that represent a simplification of overall foundation response. Because of these modeling simplifications, directly evaluating the nonlinear analysis model for acceptable foundation behavior is typically not very accurate. To more accurately evaluate this behavior, enveloped loading results from the nonlinear analysis model should be applied to a more comprehensive foundation analysis model. The response of the more comprehensive foundation analysis model can then be evaluated for overall response and design. The foundation analysis model can be used for design verification of flexure, two-way shear, and one-way shear. In most software tools used for foundation analysis, the finite element analysis results of the foundation analysis model can be integrated into design strips to evaluate flexure and shear on a sectional basis in order to design reinforcement.

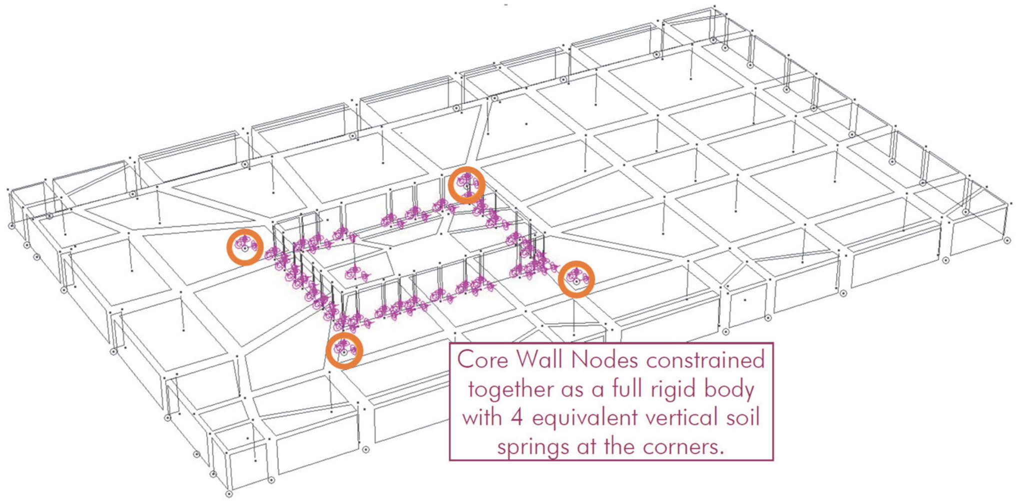


Figure 3-6 Nodes common to core wall constrained as a rigid body.

The foundation analysis model may also be used for evaluation of soil bearing pressure or overall mat deformation, although this response may not be a critical acceptance criterion in which collapse prevention or life safety is the primary performance objective. Additional discussion of foundation design procedures is described in NIST GCR 12-917-22, *Seismic Design of Reinforced Concrete Mat Foundations* (NIST, 2012b).

3.10 Soil-Structure Interaction with Input Ground Motions

Inclusion of the soil and foundation system in the simulation model, as shown in Figure 3-7, increases the system fundamental period (increased flexibility) and damping (radiation damping through the soil medium). In the past, neglecting the foundation system in the model was considered to be a conservative step. This is the case for more flexible structures. For the case of relatively stiff structures in a soft soil medium, however, the increase in fundamental period can lead to an increase in base-shear demands, as shown in Figure 3-8. In such a case, and in cases where an accurate estimate of the system response is desired, a flexible foundation (soil-structure interaction) needs to be considered when creating the simulation model.

NIST GCR 12-917-21, *Soil-Foundation-Structure Interaction for Building Structures* (NIST, 2012a), provides a detailed description on when it is important to incorporate soil-structure interaction (SSI), and how to implement it. The findings of the research in the report indicate that the key parameter used in determining whether SSI needs to be considered is the structure-to-soil-stiffness ratio, defined as:

$$R = h/(V_s T) \tag{3-2}$$

where h is the building height, V_s is the shear-wave velocity of the soil and T is the fixed-base period. h/T can be considered to be a measure of the building flexibility and V_s is a measure of the soil stiffness. The studies show that inertial SSI effects are generally negligible for $R < 0.1$, which occurs in flexible structures, such as moment frames, situated on firm soil. Conversely, SSI effects tend to be significant when $h/(V_sT) > 0.1$, as is the case of stiff structures, such as shear walls or braced-frame buildings, situated in softer soil.

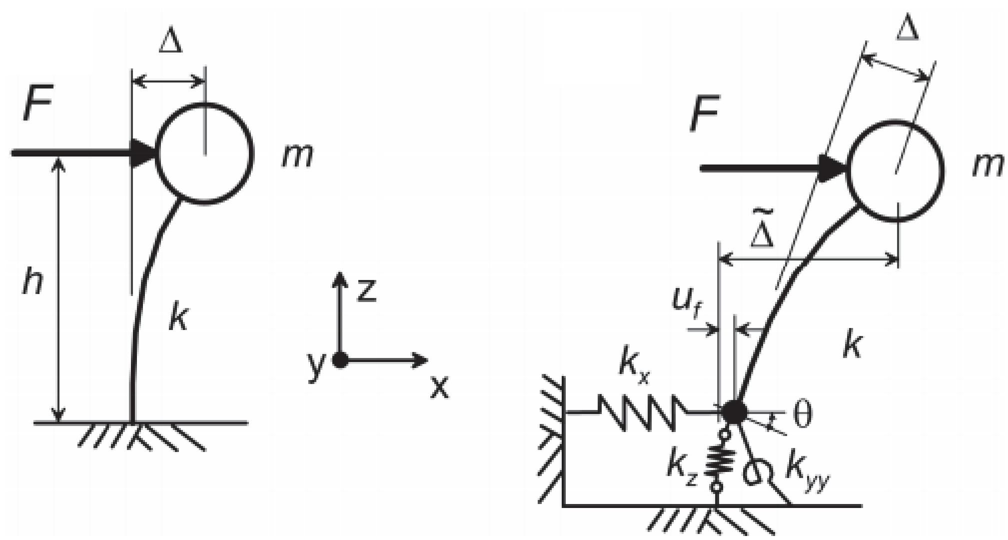


Figure 3-7 Schematic illustration of deflections caused by force applied to (a) fixed-based structure; and (b) structural with flexibility at the base (NIST, 2012a).

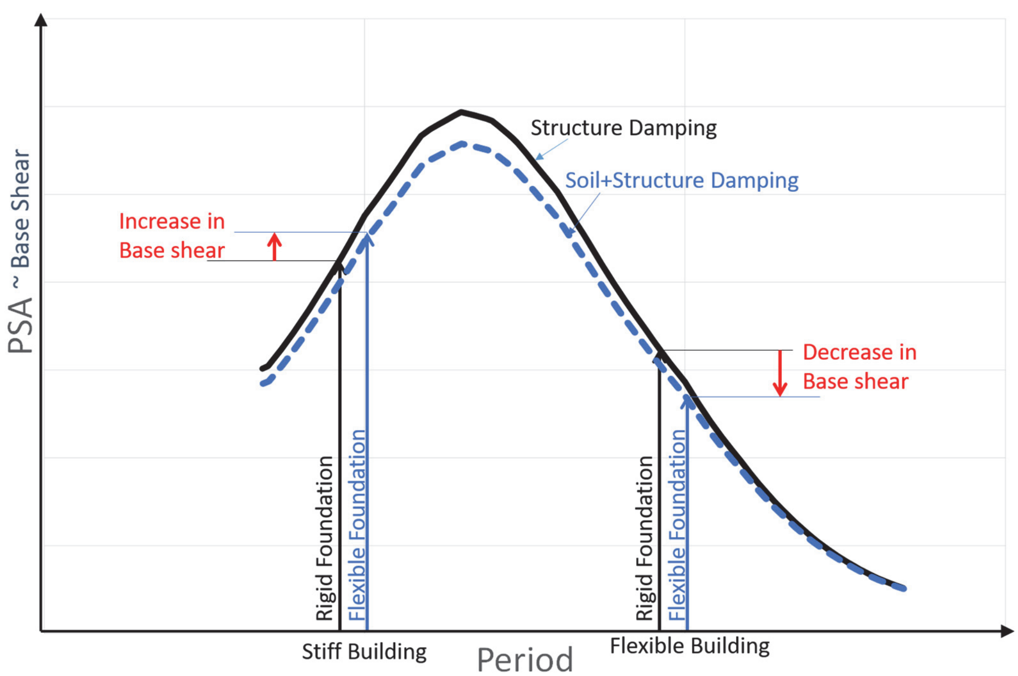


Figure 3-8 Effect of foundation flexibility and damping on seismic forces.

There are two main types of SSI effects: inertial effect and kinematic effects. Inertial effects are those that contribute to the mass, stiffness and damping of the system. These effects are included explicitly in the model by including the foundation structure and soil medium. The foundation structure consists of additional elements representing the basement, the soil medium is typically represented through simple concentrated spring-and-dashpot systems. The NIST GCR 12-917-21 provides guidelines on how to model these inertial-interaction effects, providing equations for both the spring and dashpot elements for both shallow and deep foundations.

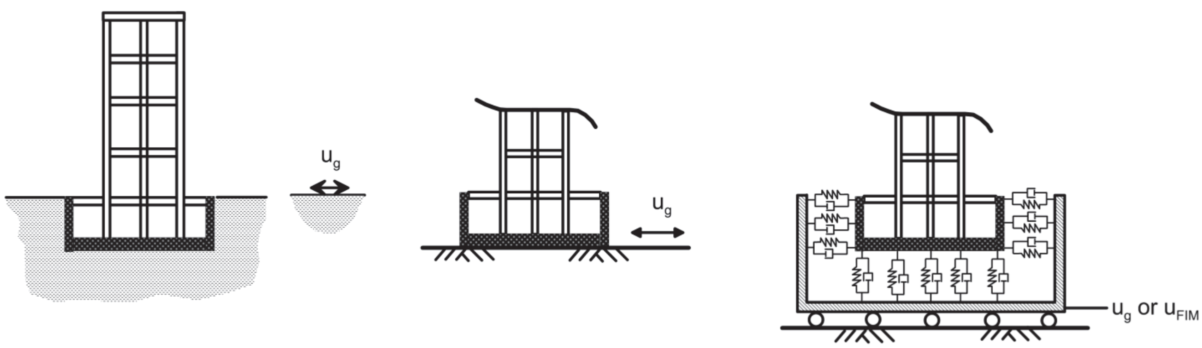
Shallow foundations embedded in soil can be modeled using Beams on Nonlinear Winkler Foundation models, which are described in the NIST report. These models provide vertical and horizontal soil springs distributed on the soil-structure boundary. These springs represent the following mechanisms: the vertical tension and compression strength of the soil, the lateral passive resistance and gapping of embedded shallow footings, and the horizontal frictional resistance along the base of a shallow foundation. The NIST report gives guidelines on the properties and distribution of these soil springs.

Kinematic effects pertain to the relative motion between the soil and the structure, i.e., how the input motion differs from the free-field motion. Related to but distinct from the modeling of foundation behavior is the question of how earthquake ground motions are input to the structural model when the foundation system is modeled explicitly. Most ground motion models and records are for free-field ground-surface conditions. The motions measured at the surface have radiated through the soil from the underlying rock, undergoing amplification due to the soil's mass and flexibility. This amplification increases with the depth of the soil layer, with the maximum amplification at the ground surface.

When the foundation is included explicitly in the model, input motions are applied to the boundaries of the foundation, possibly at various depths. Because the mass, stiffness, and damping (due to radiation and soil inelasticity) of the foundation and soil affect these input motions, using the at-surface free-field records at the foundation depth would misrepresent the seismic excitation to the superstructure being analyzed.

Kinematic effects pertain to the spatial variability of input ground motions over the building plan area (base-slab averaging) and through the soil/foundation depth (embedment effects) of basements. Base-slab averaging occurs when the continuous foundation averages out the incoherent high-frequency waves across the plane of the foundation. The embedment effect is the result of the decrease in soil amplification with depth, mainly affecting high-frequency waves. Both types of kinematic effects can result in a significant reduction in the high-frequency content of the ground motions.

Both inertial and kinematic effects can be modeled directly using a complete model of the structure and surrounding soil (Figure 3-9a), where ground motions are input at a boundary that is sufficiently far enough away from the structure to avoid interference (either at the free-field surface or at the base rock). Because it requires a complete representation of the surrounding soil and the interaction at the boundaries, this model is not feasible in a building analysis. Alternatively, the ground motions can be input directly to the structure assuming a horizontally-fixed boundary (Figure 3-9b, where vertical soild springs could be used but are not shown) or through springs and dashpots assuming full spatial coherence of the ground motions (Figure 3-9c). Where the complete structure is modeled (Figure 3-9a), the kinematic effects can be considered explictly through the input motions to the boudary of the model. Where the simplified approaches of Figures 3-9b or 3-9c are used, kinematic effects (spatial incoherence of ground motions) can be accounted for by modifying the input ground motions to account for base-slab averaging over the building plan and embedment through the substructure depth.



(a) Complete system

(b) Model for service-level earthquake

(c) Model form maximum-considered earthquake

Figure 3-9 Illustration of the method of inputting ground motions into the base of the structural model (NIST, 2012a).

The modification to the input ground motions is prescribed by a modification to the target spectra for the ground motion. Both base-slab averaging and embedment effects reduce the high-frequency content of the motions, thus reducing the target spectrum in the short-period range. Both ASCE/SEI 41-13 and ASCE/SEI 7-16, based on the NIST report, prescribe period-dependent spectrum-reduction factors. The two criteria may differ in the maximum allowed reduction, so the analyst should refer to the reference document on which they are basing their analysis.

In general, it is advisable to input the ground motion through foundation springs and dashpots, as shown in Figure 3-9c. The more simplified (no-soil model of Figure 3-9b) may also be acceptable in some situations, such as for analyses under service loads (SLE-level analysis), where soil effects are expected to be minimal, or in other situations where the soil is stiff relative to the structure.

3.11 Treatment of Gravity System

In the United States, the seismic analysis and design focus on the lateral load-resisting system portion of the building and the portions of the building designated as "gravity-only framing" are not considered in the seismic structural model. Nevertheless, it is understood that the entire structure undergoes and resists earthquake demands and these components that are not modeled can have important effects on the strength and stiffness of the building, as well as on the estimated force demands in the structural components.

At a minimum, the P-delta effects from loads on the gravity system should be accounted for in the nonlinear analysis model (as recommended in Section 3.4). This can be done by either using the "leaning column" approach outlined in Section 3.11.1 or by explicitly modeling the gravity system (per Section 3.11.2). In either case (i.e., even if the gravity system is not modeled), the possible failure modes of the gravity system must be assessed through the use of acceptance criteria for components of the gravity system (per Section 3.11.3 and Section 6.4).

3.11.1 Modeling P-Delta Effects for Loads on the Gravity System

To account for the destabilizing effects of P-delta resulting from loads on the gravity systems, the leaning column approach is typically used. This allows the addition of gravity loads in the structural model, but does not account for any of the lateral stiffness or strength of the gravity system. Figure 3-10 illustrates this approach, were the leaning column is composed of a series of rigid struts connected to one another and the base by frictionless hinges, and to the structure by articulated rigid links. The gravity loads tributary to the gravity framing are applied to this leaning column at each floor.

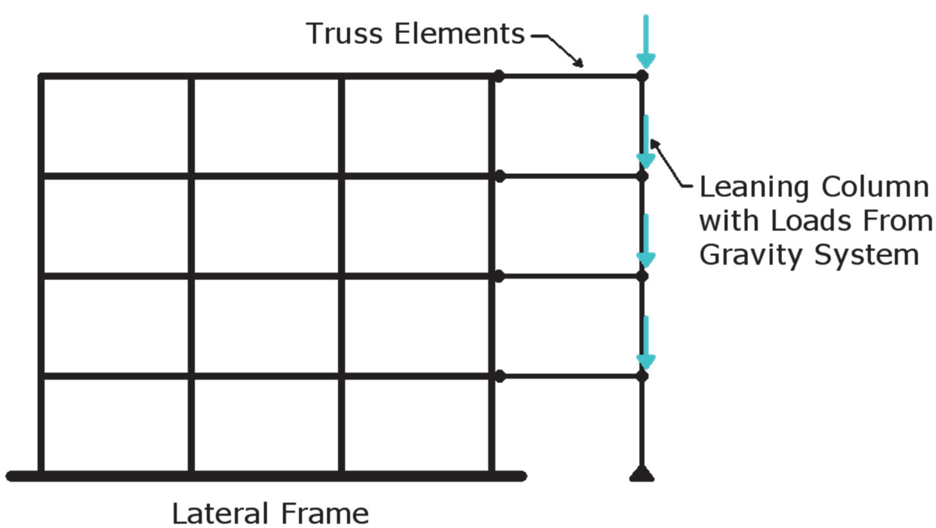


Figure 3-10 Illustration of leaning column.

3.11.2 Explicitly Modeling the Gravity System

Explicitly modeling the gravity system is preferable. This approach will produce a better representation of the building strength, stiffness, and force distributions over height, and allows for direct estimation of force and displacement demands for the gravity framing. This is important for cases where the axial loads in the gravity columns are measurably increased due to slab outriggering effects, and may affect the performance (or design) of the gravity column. Guidance on modeling gravity system components can be found in both Section 3.6 of this Guideline and in the pertinent *Part II Guidelines* for the material type of interest.

3.11.3 Assessing Gravity System Failure Modes

Possible failure modes of the gravity system must be assessed through acceptance criteria even if the system is not explicitly modeled. When the gravity system components are explicitly modeled, then acceptance criteria can be applied directly to the gravity system responses, per Section 6.4, and when the gravity system components are not modeled, then an allowable drift limit must be set for the valid range of modeling.

3.12 Effects of Non-Modeled Building Stiffnesses

Similar to the gravity system commonly being excluded from the structural model, there are other substantial building stiffnesses commonly excluded from the model. Examples of these include non-modeled beam slabs, non-modeled composite action between the slab and the steel beam, assuming all reinforced concrete components are pre-cracked, concrete or masonry partition walls (such as those enclosing stairs and elevators), stairs, cladding systems, and interior partition walls.

Exclusion of these additional building stiffnesses should have little effect on typical performance assessments used for building design, where the focus is assessing the response of just the "lateral system" of the building, and the focus is to ensure structural safety and stability. However, this approach can substantially under-estimate actual building stiffness and, thereby, substantially overestimate building deformations.

In some analysis cases, such as performing a building-specific risk assessment using FEMA P-58 (FEMA, 2012), an accurate estimate of the real drifts is needed (that is not intentionally conservative). The focus of a FEMA P-58 assessment is to assess damage, and associated repair costs and repair times, for both the structural and nonstructural components of the building and overestimation of building drifts can have a large impact on the results of such an analysis. Studies using the FEMA P-58 methodology have shown that excluding these additional stiffnesses beyond the "lateral system" can result in a 50% to 100% over-prediction of building repair cost for some cases (ATC, 2017). Accurate estimates of real building drift require that all of the important stiffnesses in the building be accounted for, not just the bare stiffness of the "lateral system."

The most substantial building stiffnesses that are commonly excluded from the structural model are those that would be characterized as "structural" stiffnesses (e.g., non-modeled beam slabs, non-modeled composite action, concrete or masonry partition walls, etc.). These items can be added into the structural model (e.g., by adjusting effective beam stiffness) if a more accurate reflection of building stiffness is needed to properly achieve the goals of the analysis.

Modeling of "nonstructural" components (e.g., cladding and partitions) is more difficult in the context of a structural model (simply because modeling of such components is not typical), but in new buildings, these components typically have a more modest stiffness contribution as compared with non-modeled structural stiffnesses (FEMA, 2017). However, these contributions may be larger in an older building or a building with unique nonstructural components (e.g., a building with heavy masonry façade).

3.13 Modeling of Residual Drifts

A brief discussion of residual drifts was provided in Section 2.2.1. Accurate modeling of residual drifts is still a topic of research and it should be understood that most analytical approaches have not been refined to attempt accurate estimations of residual drifts and there is simply a great deal of uncertainty in residual drift predictions.

Chapter 4

Nonlinear Static (Pushover) Analysis Procedure

Nonlinear static analysis (static pushover analysis) is intended to approximate the dynamic response under earthquake ground motions through the application of an equivalent static lateral load. When nonlinear methods were first introduced to seismic design practice in the mid-1990s, nonlinear dynamic analysis was not a practical option for design offices, due to the computational demands, model and software limitations, less well-developed methods to characterize input ground motions, and other factors. However, publication of FEMA 273, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (FEMA, 1997), and ATC-40, *Seismic Evaluation and Retrofit of Concrete Buildings* (ATC, 1996), introduced nonlinear static analysis, which offered a major step forward over linear analysis methods. Even though nonlinear static analysis was known to have many limitations (see below), these were accepted at the time. Today, the situation is different, where nonlinear dynamic analysis can be more routinely applied in engineering practice, owing to the tremendous advances in computational modeling, seismic hazard characterization, and other supporting technologies. Thus, the increased availability and practicality of nonlinear dynamic analysis, combined with the inherent limitations of nonlinear static analysis, have dramatically reduced reliance on nonlinear static analysis for design.

In spite of its limitations, nonlinear static analysis continues to serve a valuable role as a companion to nonlinear dynamic analysis. In particular, nonlinear static analysis can be effective for the following: (1) verifying (and debugging) nonlinear analysis models before running nonlinear dynamic analyses; (2) interrogating a structure to understand its nonlinear behavior; and (3) refining seismic designs. In addition, for low-rise structures whose response is dominated by first-mode response (with minimal higher mode effects), nonlinear static analysis may be sufficiently accurate for seismic design of new or retrofitted structures. For guidelines on when nonlinear static analysis may be acceptable, see the limitations on the procedure in Section 4.6 of this *Guideline* and in Section 7.3.2.1 of ASCE/SEI 41-13, *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE, 2014).

4.1 Overview of Procedure

Procedures for nonlinear static seismic analysis are fairly well developed and described in ASCE/SEI 41-13, its predecessor documents (FEMA, 1997; FEMA, 2000; ATC, 1996) and other supporting studies (FEMA, 2005). The overall steps in the procedure include:

1. Creating and calibrating the nonlinear analysis model
2. Defining and applying the static gravity loads
3. Defining an equivalent earthquake lateral load pattern
4. Establishing a target displacement for a specified ground motion intensity
5. Incrementally increasing the equivalent lateral load until the target displacement is reached
6. Assessing the story drifts and other demand parameters (hinge rotations, member forces) against specified acceptance criteria at the prescribed target displacement

Some or all of these steps may be repeated as desired to investigate alternate model parameters (Steps 1 to 6), alternate load patterns (Steps 4-6), or alternate target displacements (Steps 5-6).

4.2 Component Modeling Requirements

Component modeling requirements for nonlinear static analysis are simpler than for nonlinear dynamic analysis in that the component models do not need to capture the cyclic hysteretic response. However, when static pushover analyses are used directly to check the seismic design (as opposed to being used as a preliminary step to a nonlinear dynamic analysis verification), the force-deformation behavior should be calibrated to incorporate cyclic degradation. So, for example, this implies calibrating the concentrated hinge component models to the cyclic skeleton curve, rather than the monotonic backbone curve. The force-deformation response curves in the current (and previous) editions of ASCE/SEI 41 are calibrated to the cyclic skeleton, but this may not be the case for other models. In particular, most fiber-type and continuum finite element models are formulated to simulate cyclic degradation effects directly, in which case monotonic analyses run using these models would not represent cyclic degradation and may give unconservative results. When nonlinear static analysis is not used for design, but is instead used to help verify a nonlinear dynamic analysis model, then it is less important that the static analysis be calibrated to the cyclic skeleton response. This is because the goals in verifying a nonlinear dynamic analysis model are more related to confirming the general trends in the yielding response.

4.3 Equivalent Static Load

Definition of the equivalent static load poses one of the major challenges and limitations of nonlinear static analysis. In concept, the load pattern should be such that the resulting displaced shape will approximate that of the actual structure under dynamic loading. In practice, this is very difficult to achieve except for low-rise buildings with regular geometry and structural systems, whose inelastic deformed shape is similar to that of the elastic fundamental vibration mode.

Section 7.4.3.2.3 of ASCE/SEI 41-13 recommends a lateral load pattern that is proportional to the story masses and the fundamental model shape. Sometimes, alternate load patterns, such as a uniform load pattern up the height of the building, are used to help bracket the response; however, as noted in the commentary to ASCE/SEI 41-13, studies of alternative load patterns, include multi-mode and adaptive methods, to improve the reliability of the method to account for inelastic higher mode effects have proved inconclusive (e.g., FEMA, 2005). This is one of the major reasons why nonlinear dynamic analysis is now preferred over static nonlinear analysis.

4.4 Target Displacement

Historically, there have been two major competing methods to determine the target displacement as a function of the ground motion intensity (usually $S_a(T_1)$) and the fundamental period of vibration, T_1. The so-called "coefficient method" is the predominate approach used in ASCE/SEI 41 and its predecessor documents, and the "capacity spectrum method" was featured in ATC-40 and other documents. The coefficient method is arguably easier to use, whereas the capacity spectrum method has the virtue of providing a graphical interpretation of response. However, both methods involve a number of simplifying assumptions and parameters that have been empirically calibrated to data from nonlinear dynamic analyses. Among the two, the coefficient method of ASCE/SEI 41 is recommended, as it incorporates recent studies (FEMA, 2005) to improve its calibration for cyclic degradation effects.

4.5 Interpretation of Structural Response

As noted above, interpretation of results from the nonlinear static analysis model depends on its intended application. For cases where the analysis is deemed to provide a reliable basis of checking the structural performance, results of the analysis can be used to evaluate appropriate acceptance or limit state criteria. This would include, for example, the way in which nonlinear static analysis is applied for use in ASCE/SEI 41-13, which specifies component acceptance criteria for specific performance targets (IO, LS, CP) for deformation- and force-controlled components. Although the procedures for checking component acceptance criteria are straightforward, the interpretation of the response is not. Questions invariably arise as to the relationship of specific component level checks to the overall system response. For example, to what extent does exceeding the CP limit state in one or more components impact the collapse safety of a large indeterminate structure? Apart from the inherent limitations of static analysis itself, these ambiguities in relating the local component response to the overall response are another significant limitation that affects both nonlinear static and dynamic analysis methods.

Although there are no formal acceptance criteria to check when nonlinear static analysis is used to help verify the nonlinear dynamic analysis model and understand the inelastic behavior, it is useful to go through some systematic checks to formalize the process. The following are suggestions for specific things to look for in the nonlinear static analysis results:

- Plot the story drift ratios under various target displacements to identify and examine any concentrations of inelastic response in the structure, including building twist.
- Compare the roof drift versus base shear response and check whether the base shear at the onset of significant yielding and the ultimate load are within expectation relative to the design base shear.
- Plot the inelastic force-deformation response of selected deformation-controlled components to confirm that the response is as expected.
- Check the force demands in selected force-controlled elements to confirm how they compare to the expected member strengths.
- Examine the estimates of target displacements to use in checking the displacement demands from the nonlinear dynamic analysis.
- Run the pushover analysis with and without geometric nonlinear (P-delta) effects to confirm how significantly they affect the response in the elastic and inelastic range.
- Push the structure beyond the target displacement to help confirm that the structure continues to behave in a stable fashion, and therefore, is less sensitive to variabilities in response due to uncertainties in ground motions and other effects.

4.6 Limitations of Nonlinear Static Analysis

Major limitations of nonlinear static analysis stem from its inherent inability to capture dynamic response, including the following:

- Multi-mode dynamic behavior, which is especially important for taller buildings (> 4 stories) and/or buildings with geometric or structural irregularities,
- Cyclic behavior and degradation of materials and components,

- Rate-dependent effects, which are particularly important for systems that employ viscous dampers,
- Behavior of seismically isolated or rocking systems, and
- Unique characteristics of earthquake ground motions, including spectral shape, near-fault pulse, and duration effects.

In addition, nonlinear static analysis methods are not well-suited to assessing the implications of strength degradation response on the overall structural system, which can ultimately lead to concentration and accumulation of drifts, leading to structural instability.

Chapter 5

Nonlinear Response-History Analysis Procedure

Nonlinear response-history analysis is a dynamic analysis where the structural model is subjected to one or more ground motions that are selected and scaled according to the goals of the analysis. Nonlinear response-history analysis overcomes most limitations of nonlinear static (pushover) analysis (Chapter 4) due to its ability to accurately represent multi-mode behavior and other time or history dependent behaviors such as rate-dependent effects, path-dependent cyclic behavior, and a realistic modeling of ground motion characteristics (through proper ground motion selection) such as spectral shape, long duration and near-fault pulse effects. The topics covered in this chapter include the following: (1) characterization of the earthquake ground motions; (2) numerical modeling considerations in performing the time-step integration; and (3) interpretation and reporting of results.

5.1 Characterization of Earthquake Ground Motions

Unlike nonlinear static analysis, nonlinear response-history analysis offers a more accurate representation of the temporal response of the structure to the input ground motions. In order to obtain meaningful results, it is important that the input grounds motions reflect as accurately as possible the characteristics of the seismic hazard at the site. This section summarizes how to characterize earthquake ground motions when the goal is prediction of mean structural response for a specified level of ground motion demand. Selection and scaling of ground motions is a broad topic and this section does not cover all possible aspects related to ground motions. The following decisions have to be made to characterize ground motions:

- **Type of assessment method.** The typical type of assessment method is termed an “intensity-based” assessment (FEMA, 2012), which is based on analyzing the response of the building for a specified level of ground motion (e.g., MCE_R, DE, or SLE). Two other types of methods are described in FEMA P-58 as “scenario-based” and “time-based assessments” (FEMA, 2012); these are not covered in this *Guidelines* document and the user is referred to FEMA P-58 for ground motion guidelines for those alternative assessment methods.
- **Target spectra.** ASCE/SEI 7-16 allows two types of target spectra for an intensity-based assessment, a basic uniform-hazard (or uniform risk) spectrum and site-specific scenario spectra (usually two or more scenarios). The user is

referred to Sections 16.2.2 and C16.2.2 of ASCE/SEI 7-16 for the development of the target spectra. The use of multiple scenario spectra generally increases the analysis effort by a factor of 2 or more, but often results in a reduction in unnecessary conservatism in seismic demands imposed by the ground motions, since each of the scenario spectra will be enveloped by the target spectrum.

- **Number of ground motions.** ASCE/SEI 7-16 requires 11 motions for predicting the mean building response under MCE_R-level ground motions, per the considered scenario. This is the number that is recommended for most cases, but fewer motions could be used at lower ground motion intensities where there is less nonlinearity in the structure or for selected design or modeling sensitivity analyses. If accurate prediction of response variability is required, in addition to the mean response estimate, then thirty or more motions might be necessary and the selection and scaling approach would need to be modified to explicitly account for ground motion variability. For such cases, the user is referred to the Conditional Spectrum approach outlined in NIST GCR 11-917-15, *Selecting and Scaling Earthquake Ground Motions for Performing Response-History Analyses* (NIST, 2011b).
- **Components of ground motion.** In most cases, two horizontal components of ground motion are used and no vertical motion is included in the structural analysis. However, if the building is sensitive to vertical motions (e.g., long spans, large cantilever elements or transfer girders, exceptionally high vertical ground motions in the near-field), a vertical component of motion should be included, or alternatively a study should be performed to verify that the effect is not significant. Refer to Section C16.3.1 of ASCE/SEI 7-16 for more guidance.
- **Ground motion selection.** The ground motions should be selected based on magnitude, distance, tectonic setting, site soil conditions, and spectral shape, consistent with the controlling seismic source and intensity characteristics at the site for the considered design spectrum, in accordance with Sections 16.2.2 and C16.2.2 of ASCE/SEI 7-16. For near-fault sites, the proportion of ground motions with near fault and directivity effects should represent the probability that the target hazard will exhibit these effects. Although recorded ground motions are usually preferred, simulated ground motion records may be used when there is an insufficient number of recorded ground motions.
- **Ground motion intensity measure.** The ground motion intensity is usually defined as an elastic acceleration response spectrum, S_a, which is typically obtained through a seismic hazard analysis. This response spectrum can be expressed as the geometric mean S_a (a type of average S_a value for all horizontal orientations) or maximum direction S_a (the largest S_a in any horizontal orientation). It is critical to use a consistent definition of S_a for both the target spectrum and the spectra computed for the ground motion records.

- **Ground motion scaling.** Ground motions should be scaled such that the spectra match the target spectrum over the period range important for the response of the building. This can be accomplished using the scaling procedure in Section 16.2.4 of ASCE/SEI 7-16 or other procedures specifically intended for performance-assessment, where the goal is to determine expected performance.
- **Spectral matching.** Spectral matching is a viable approach to develop ground motions for nonlinear analysis when the goal is to predict mean building response. Because spectral matching can cause significant modification of the record signal frequency and energy content, care must be taken to ensure that important motion characteristics are retained after the matching process has been completed. This is especially important for near-source motions with pulses. The reader is referred to Section C16.2.4.3 of ASCE/SEI 7-16 for further information on the use of spectral matching and the 10% penalty applied to spectral matching in ASCE/SEI 7-16. One potential disadvantage of tight spectral matching is that it can reduce or eliminate variability in the ground motion suite. This can lead to unexpected or unconservative results if the analysis intends to consider individual record structural response in addition to the mean response, which is the reason for the 10% penalty in ASCE/SEI 7-16.
- **Application of motions to structural model.** Consistent with Section 16.2.5.1 of ASCE/SEI 7-16, for near-fault sites, ground motions should be applied in the fault-normal and fault-parallel orientations. For sites that are not characterized as near-fault, these specific orientations do not need to be retained and motions can be applied arbitrarily to the building, which means that the average of the components applied in one direction should be comparable to the other direction. In either case, multiple orientations of the same records are generally deemed to be unnecessary, although some codes still require that in certain cases (e.g., California Office of Statewide Health Planning and Development and ASCE/SEI 41-13); if the analyst is interested in performing additional analyses, it might be more useful to use additional ground motion records rather than run multiple orientations using the same records.
- **Modifications for soil-structure interaction effects.** Although the simplest analytical approach is to apply free-field surface motions, this can be significantly conservative depending on the type and depth of the foundation, since the effective input ground motions at the level of the foundation can be significantly reduced compared to surface motions. Hence, ignoring soil-structure interaction is typically conservative for typical buildings, but it is recommended to be considered for buildings with significant embedment (e.g., buildings with multiple basements). Alternatively, the input motion can be modified to account for kinematic interaction effects (termed a "foundation input motion"), which can result in a reduction in the target response spectrum, mainly

in the high-frequency range. This might lead to a modification of the ground motion selection approach, and will usually require that the model explicitly account for the soil-structure interaction through the modeling of nonlinear soil springs. Additionally, inertial effects can be accounted for by including foundation components in the structural model. More detailed guidance on treatment of kinematic interaction effects can be found in the NIST GCR 12-917-21 report (NIST, 2012a) and in Chapter 19 of ASCE/SEI 7-16.

5.2 Numerical Modeling Considerations

Nonlinear response-history analysis requires discretization in the time domain (by modeling the time duration of a ground motion and the associated structural response by dividing the analysis into a series of discrete time steps, sequentially executed), as well as the spatial domain (through use of a finite element mesh to approximate the geometry and associated properties of the structure). In addition to customary errors that may be accrued due to the spatial discretization (such as, in a static analysis, having an inaccurate force distribution in the structure due to using a mesh that is too coarse), errors may accrue during nonlinear response-history analysis due to temporal discretization. Depending on the nature of the temporal errors in particular, it may be difficult to discern that the errors exist, especially if it is difficult to approximate or estimate the anticipated response through some other approximate calculations. Thus, it is important to be aware of the potential causes of both spatial and temporal error in nonlinear response-history analysis.

5.2.1 Spatial Errors

Spatial errors in creating the associated finite element mesh of the structure are comparable to errors that may occur in executing static analyses of the structure, with the possibility of additional errors due to dynamic wave propagation. Correspondingly, the solutions to ensure an accurate spatial representation of the structure are comparable to those of static analysis, although it should ensure that the spatial error is assessed within the context of understanding the likely dynamic response of the structure. Refinement of the mesh is important, for example, to ensure that the significant modes of vibration are modeled accurately. How this refinement impacts the model accuracy may depend in part on the level of modeling that is selected, whether it be concentrated-hinge type models, distributed plasticity fiber-type models, or continuum finite element models. Concentrated plasticity and fiber models, for example, can typically be coarse models for dynamic analysis as they are for static analysis, with one to three elements per member often adequate to capture the dominant modes of deformation. If continuum models are used, mesh refinement may often be comparable to that used in static analysis unless it is known that dynamic loading will excite localized response.

In particular, the distribution of degrees-of-freedom should be sufficient to enable all low frequency (high period) modes to be captured accurately. Unlike frame elements, the stiffness of continuum elements (such as plate elements used to model shear walls) is more dependent on element size, which requires additional calibration and verification.

If a higher level of resolution of the structural response is desired, such as being able to capture the buckling of an individual column or brace, a finer mesh should be used. Typically, localized response such as flexural buckling of a beam-column will be a mode of response with a frequency of vibration that is several orders of magnitude larger than the frequency of the global sidesway of the structure or other related dominant modes of vibration. Note that with an overly refined mesh, the model may require substantially shorter time steps to capture high-frequency response, which may make explicit integration a better choice than implicit integration in this case. Alternatively, the local buckling behavior can be investigated separately in a sub-model, and a generalized representation of the behavior incorporated back in the global model.

5.2.2 Temporal Errors

Temporal errors accrue largely due to having a time step that is too large. Two categories of dynamic time history analysis may be used for nonlinear response-history analysis: implicit analysis and explicit analysis. Bathe (2014), Chopra (2011), Cook et al. (2001), Zienkiewicz and Taylor (2013), Hughes (2000), Crisfield (1996), and others provide excellent summaries of the common methods for time discretization. The use of the implicit analysis method is more common in practice and is supported by the majority of structural analysis software, while explicit analysis is less commonly used.

5.2.2.1 Explicit Integration

The time-step size in a nonlinear time-history analysis must be selected, whether by the user or through an automated procedure, to ensure both stability and accuracy of the analysis. Explicit analysis methods are conditionally stable, meaning that the time-step size must be smaller than the smallest period of vibration of the structure (a structure with N degrees-of-freedom has N modes of vibration due to the spatial discretization). For seismic analysis of buildings, this time-step size is often several orders of magnitude smaller than the time-step size needed to assess the response of the structure, resulting in a large number of time steps. Explicit analysis algorithms, however, typically will not require solution of a simultaneous set of equations and thus require little computational time per step, and since they also do not require forming or solving a stiffness matrix, they require substantially less memory and disk space, which can be important for very large models. Explicit analysis is especially appropriate for structures dominated by high frequency modes, or for structures subjected to loading with very short duration or very high frequency response, where wave propagation dominates the response, such as blast loading. Since the number of explicit time steps required to integrate earthquake ground motions can be very large, explicit analysis is not typically used for seismic analysis of building structures. However, there are situations where explicit methods may perform better than implicit methods, such as in analyses involving collapse or near-collapse where the structure goes highly nonlinear and experiences significant stiffness and/or strength degradation due to material or geometric nonlinearities (which can lead to convergence problems with implicit methods).

It is usually common to use low-order elements (e.g., elements using linear displacement integration as opposed to higher quadratics) with explicit analysis, since those elements tend to reduce numerical noise in the solution. In contrast, implicit analysis methods can use both low- and high-order elements.

5.2.2.2 Implicit Integration

Implicit analysis methods may be either conditionally or unconditionally stable, depending on the approach. For nonlinear response-history analysis, a time integration algorithm that is unconditionally stable, such as the Newmark-β constant acceleration method (Bathe, 2014) that is generally supported by the majority of structural analysis software, is highly recommended, as the stability of the algorithm is not dependent on the time-step size. This enables the time-step size to be selected based solely on the required accuracy of the analysis. For these methods, the time-step size should be selected to be on the order of $^{1}/_{20}$ to $^{1}/_{10}$ of the period of vibration of the dominant mode of vibration with the smallest period of vibration. Selection of the dominant modes of vibration should include modes with substantial modal mass participation factors.

The time-step size should typically be smaller than (or equal to) the time step used to discretize the ground motion itself (ground motions are commonly available with a discretization between 0.005 and 0.02 seconds).

If errors accrue in unconditionally stable implicit time integration approaches, the analysis will not become unstable and thus the response will typically not grow exponentially to the point where it is clear the response is inaccurate. Rather the errors will accrue more gradually, often making it difficult to discern that the results are inaccurate. A simple and effective way to determine if the time-step size in implicit analysis is sufficiently accurate, is to perform a simple sensitivity study of the time-step size. For example, if the analysis is rerun with the time-step size cut in half and the results are comparable within a tolerance, then the time-step size should be adequate.

5.2.3 *Convergence*

It is common to conduct nonlinear iterations within each time step in a nonlinear analysis. Each time step should typically converge within a relatively small number of iterations (1-20) if the time-step size is chosen correctly. To minimize spurious numerical issues that cause issues with convergence, it is common to assign small masses (i.e., on the order of 0.005% of the primary mass values to all active translational and rotational degrees-of-freedom in the mesh). Convergence is often assessed by comparing normalized values of key deformation, force values, or work (energy) values. A tolerance for this convergence check is commonly set to a value of 0.001 to 0.0001 of the quantity being assessed. Some software programs also have strategies for addressing convergence through automatic selection and adaptation of the time-step size. For example, if a time step does not converge, the software may automatically and recursively rerun the step with the time step cut in half, and some of those methods may perform automated accuracy checks that are used to determine if the size of the time step should be reduced, increased, or maintained. The benefit of such methods is that they can be used to control the accuracy of the solution, and also speed the analysis by avoiding the use of unnecessarily small time steps. It is advisable to set limits on the maximum number of iterations per time step (e.g., 20 to 100), as well as a minimum acceptable step size in order to avoid excessive iterations in ill-conditioned and unstable models.

It is common to encounter convergence problems while performing nonlinear response-history analysis, and it usually requires multiple iterations of model review and modification to finally achieve convergence. Even models that have been successfully analyzed linearly, can still encounter difficulties in nonlinear analysis due to a number of reasons including errors in element discretization and connectivity, material hysteretic models, negative stiffness due to P-Δ or large deformations, numerical errors, or in some cases actual dynamic instability and collapse. Unfortunately, it is often difficult to distinguish between convergence problems and instability/collapse, and the two cases are often treated similarly.

While an exhaustive listing of all possible sources of convergence problems is not possible, the following are some common causes of convergence problems:

- **Nonlinear iteration algorithm.** Different iterative algorithms have different trade-offs of accuracy, stability, and speed. Multiple algorithms should be attempted when possible. Some software can adaptively switch between different solution algorithms.
- **Time-step size and convergence tolerance.** As discussed above, reducing the analysis time step or adopting automatic time-stepping methods can help improve convergence. In some cases, reducing or increasing the convergence tolerance can also be helpful.

- **Element connectivity and releases.** Incorrect element releases causing zero stiffness along one or more degrees-of-freedom can cause failure of the analysis, even if no issues are encountered during linear analysis. Examples include torsionally unrestrained frame elements or membrane elements supporting out-of-plane loads. A careful examination the structure's deformation under different mode shapes can be helpful in detecting such modeling errors.

- **Mesh discretization and element size.** Changes in element sizes should be gradual and all element degrees-of-freedom should be appropriately transitioned and connected. Sudden changes in element sizes can cause large dynamic fluctuations in stresses across elements which can cause unrealistic localized yielding and convergence problems.

- **Material hysteretic modeling.** Sudden slope changes in material nonlinear backbone curves, especially in the presence of steep negative stiffnesses or sudden stiffening, can lead to convergence problems (e.g., backbone curves that exhibit sudden steep negative slopes), though some software can experience convergence problems even with more gradual strength loss. The analyst should assess if the models are realistic or over-conservative and consider making some adjustments. This usually involves engineering judgment to balance numerical convergence and accuracy requirements.

- **Insufficient validation of nonlinear components.** A significant source of difficult-to-track errors is the incorrect use of nonlinear components without sufficient validation and proper understanding of their internal behavior. The need for verification and calibration is especially important for complex component models. See Appendix C for additional discussion on validation.

- **Undamped response.** While most nonlinear models usually have sufficient viscous and hysteretic damping, care should be taken to ensure that all degrees-of-freedom and modes of vibration have at least a small amount of damping to avoid spurious undamped oscillations. This can be achieved by prescribing a small amount of damping (e.g., Rayleigh damping), even when other types of damping, such as modal damping, are used in the model, and ensuring that each degree-of-freedom has at least a small mass associated with it.

- **Geometric nonlinearity.** Geometric nonlinearities can amplify other convergence problems or cause new ones. Element connectivity and releases should be verified in the presence of geometric nonlinearity. Some elements may not be intended to have geometric nonlinearity (e.g., soil springs) and should be explicitly specified to have linear geometry in those cases.

5.3 Complete the Analysis and Interpret the Structural Response Results

After constructing the model and performing basic quality control checks, nonlinear response-history analysis can be performed under the ground motion suite of records. Recommended quality control checks include checking the elastic modes of the structure, verifying the total mass and gravity loads, verifying that the model is well behaved under other types of loading such as static gravity loading, lateral static linear and nonlinear pushover analysis, and linear response spectrum analysis. The nonlinear analysis solution strategy can be checked by performing preliminary sensitivity analysis to verify the accuracy and suitability of the selected solution algorithms and time steps. For example, the solution time-step and convergence tolerances may be varied for one of the ground motions, and the analysis results, such as deformations, accelerations, forces, and residual displacements, are checked to make sure that no further refinement of the time-step or convergence criteria are required. It is also advisable to verify that the damping in the model by observing the decay of the building displacement (e.g., roof displacement) at the end of the record. Once the dynamic analysis is completed, global force and energy balance checks can also be performed.

The structural response due to the different ground motions typically exhibits significant variability, even when the ground motions are selected or spectrally matched to the target response spectrum. Structural results, such as story drift, column axial forces or beam plastic rotations, may vary substantially between ground motion records. In lieu of selecting the maximum response among all ground motions, which can be overly conservative, it is customary to compare the mean (or some other percentile) of the response to acceptance criteria. This generally requires that all analyses reach convergence to allow the computation of the mean. As more ground motions are used, there is increased likelihood of having at least one ground motion that causes collapse or non-convergence, and it might be reasonable to admit at least one such case. For example, Section 16.4 of ASCE/SEI 7-16 allows one "unacceptable response" out of 11 ground motion analyses, in which cases the mean response is estimated as 120% of the median value of the successful analyses, but not less than the mean of the ten converged analyses. Although deformation controlled actions are generally checked using the mean response, a higher limit is recommended for force-controlled actions. Section 16.4.2 of ASCE/SEI 7-16 requires force-controlled elements to be checked under amplified demands that can range from 1.0 to 2.0 times the mean demand, depending on the importance of the element and the likelihood that its failure will lead to local or global collapse. A more detailed discussion of acceptance criteria is presented in Chapter 6.

While the evaluation of structural response is generally focused on interpreting mean results from the different ground motion analyses, it is still important to examine individual analysis results in order to ensure that each of the runs has concluded successfully without convergence problems or serious structural failures. A close inspection of each analysis can pinpoint weak spots in the design that can benefit from capacity design or other types of improvement. These weaknesses may not present themselves in the statistical mean results. While the analyst might be tempted to fully automate the interpretation of the results and their statistics, it is recommended that some time is allocated to performing these individual checks. This serves as a quality control check and also helps achieve a better understanding of the structural behavior.

Chapter 6

Performance Assessment and Acceptance Criteria

Nonlinear dynamic analysis can be used for a variety of purposes, ranging from a comprehensive performance assessment (e.g., as is done in FEMA P-58) to a more routine application of evaluating conformance with pre-defined building code acceptance criteria (e.g., as is done with Chapter 16 of ASCE/SEI 7-16 or recent *Tall Building Guidelines* including PEER (2017) and LATBSDC (2015)). For all types of performance assessment, acceptance criteria are needed to gauge whether the building meets the intended performance goals. This *Guidelines* document focuses on requirements for nonlinear modeling, and it does not specify what acceptance criteria should be imposed to judge performance. The expectation is that this *Guidelines* document will be used in conjunction with a design standard or other guidelines that specify the required acceptance criteria. Even so, since acceptance criteria are a fundamental aspect of the performance assessment process, and to the extent that the definition of these criteria affect aspects of the analysis model (e.g., calculation of demand parameters that are consistent with the acceptance criteria), this chapter summarizes various acceptance criteria that are used with some of the major performance-based guidelines and design standards in use today.

For building code applications, such as ASCE/SEI 7-16, which focus on structural safety, the acceptance criteria typically address limits on story drifts, component deformations, and component forces. The criteria may also address instances where the analysis results are deemed invalid, either because of excessive deformations beyond the reliable range of modeling or problems with numerical convergence or other issues. For more comprehensive performance assessments, such as FEMA P-58, the acceptance criteria may be implicit in the global performance descriptors, rather than specific limits on structural demand parameters. For example, the criteria may be specified in terms of limits on component damage states, limits on repair costs or repair times, and limits on fatality and injury rates.

It is conceptually useful to break acceptance criteria up into two types of criteria: (a) fundamental acceptance criteria that are independent nonlinear modeling attributes; and (b) model simplification acceptance criteria that are required because of limitations in the nonlinear structural model. Examples of fundamental acceptance criteria include story drift limits or limits of number of ground motions that cause structural collapse (as judged by excessive story drifts or other criteria). Examples of model simplification acceptance criteria would be checking the force demands on elastically-modeled components, which are deemed to be force-controlled, and checking the deformation demands of inelastically-modeled components to ensure that they are within the valid range of modeling. Fundamental acceptance criteria are always necessary, whereas model simplification acceptance criteria can sometimes be avoided through associated improvements to the structural modeling.

Definition of acceptance criteria involves consideration of uncertainties in the calculated demands and the capacities, where the specified criteria are intended to provide a reasonable level of confidence that the performance requirements will be met. As such, consideration of these uncertainties are considered either qualitatively or quantitatively in most specified acceptance criteria. The following sections include some discussion of how uncertainties are incorporated in existing acceptance criteria, and Appendix B provides further discussion on treatment of uncertainties in the development of acceptance criteria.

6.1 Story Drift Demands

Story drifts provide an overall measure of the seismic deformation demands on the building and an effective way to limit the structural response to a range where it can be reliably evaluated by nonlinear analysis. Therefore, story drifts are a common fundamental acceptance criterion used in many standards and guidelines. The most common story drift measurement is a simple calculation of the relative displacement between two stories, normalized by the story height (also called story drift ratio). For evaluating building damage in buildings where significant vertical deflections (e.g., due to flexural elongations of tall shear walls) exist, it may be necessary to modify the basic drift calculation to consider "racking drifts" (see Section 6.7).

6.1.1 Story Drift Limits in Building Codes

For design using nonlinear dynamic analysis, most design specifications impose limits on the peak transient drifts, and some documents also specify limits on residual drifts (see Appendix A). Although these limits have some relationship to helping ensure that the building performs safely, most of the limits are empirical and have been calibrated to existing limits for approximate code-based design procedures. These limits generally pertain to the story drift and do not distinguish between story drifts and racking deformations. The story drift limits are typically applied to the maximum resultant in the difference in drifts (e.g., considering the resultant of deformations in two orthogonal directions) at any point in the building floor plan. The drift demands are usually evaluated based on the average (mean value) of the peak responses from the suite of input ground motions.

Peak Transient Drift. The ASCE/SEI 7-16 criteria require that the mean story drift ratios for the suite of eleven MCE_R-level ground motions be less than two times the linear-design limits provided in Table 12.12-1 of ASCE/SEI 7-16. The ratio of two is intended to achieve parity with criteria for conventional design and is based on the product of 1.5 and 1.33, where 1.5 is the ratio of MCE_R to design basis ground motions and 1.33 is an approximation to a typical ratio of R/C_d. For typical building systems with standard occupancy, this results in a limiting story drift ratio (SDR) of 0.04 (4%). The *Tall Building Guidelines* limit the peak mean SDR under MCE ground motions to 0.03 (3%), which is based on a more conservative extrapolation of the traditional design level limits to the MCE intensity, considering only the 1.5 factor. At least one study (Gokkaya et al., 2016) has suggested that the tall building story drift limit of 0.03 may provide a more reliable limit to mitigate collapse risk, as compared to the more lenient ASCE/SEI 7-16 requirement. The *Tall Building Guidelines* also limit the maximum drift from any MCE ground motion to 0.045 (4.5%), and mean drifts under service level ground motions to 0.005 (0.5%). ASCE/SEI 41 provisions do not specify any drift limits, relying instead on component acceptance criteria to evaluate performance.

Residual Drift. Residual drifts are referenced in some standards, primarily when the performance goal includes limiting building downtime and losses, associated with closure and demolition. When residual drifts are used in the acceptance criteria, care should be taken in the nonlinear structural modeling to ensure that the residual drift is calculated reliably. In particular, compared to the peak drift, the calculated residual drift is more variable and more sensitive to the yield strength, pinching and unloading, and other response attributes in the cyclic hysteretic models as well as to ground motion characteristics. FEMA P-58 presents a simplified procedure for estimating residual drifts, based on the peak drifts (FEMA, 2012).

The *Tall Building Guidelines* are among the few standards that address residual drift, limiting the mean residual story drift under MCE ground motions to 1%. Most other standards, including ASCE/SEI 7-16 do not specify any limits on residual drifts.

6.1.2 Story Drift Limits in Performance-Based Design

Whether or not peak or residual drifts are limited by design standards, drifts are one of the primary demand parameters used to evaluate building performance. For example, the performance-assessment procedures of FEMA P-58 rely almost exclusively on peak story drifts, in addition to peak floor acceleration, to assess earthquake damage and its consequences. In contrast to minimum building code requirements that are only concerned with the maximum story drift ratio at any location in the building, loss assessment requires a more complete description of peak transient drifts up the height of the building, including information on correlation of peak drifts up the building height. Therefore, for performance assessment studies, the peak drifts up the building height should be reported separately for each ground motion, from which relevant statistics can be extracted for performance assessment.

Apart from their role in formal loss assessment calculations, graphical presentations (plots) of drift values up the building height are also informative to understand the structural behavior.

6.2 Collapses and Other Unacceptable Responses

Situations may arise where the results of nonlinear dynamic analyses exhibit dynamic instability, do not converge properly, or where a component response exceeds the valid range of modeling. In the example of Chapter 16 of ASCE/SEI 7-16, there are four types of such "unacceptable responses" that are included in the acceptance criteria.

The first type of unacceptable response is excessive drift, which suggests that the ground motion demand exceeds the building's collapse capacity, and is considered a fundamental acceptance criterion (i.e., one that does not depend on any model simplifications.

The remaining three types of unacceptable responses are characterized as model simplification acceptance criteria, the triggering of which can be avoided through improvements to the structural model. These include the following:

1. Non-convergence during the nonlinear analysis, resulting from causes such as: (1) incomplete iterations to achieve equilibrium tolerance; (2) ill-conditioning in the system of equations; or (3) other numerical issues that prevent the computer run from finishing. This can be avoided by identifying and resolving the specific cause of the numerical problem. Methods of resolution will depend upon the specific situation, including such means as: (1) revising structural model input parameters (member properties, masses, damping, etc.); (2) adjusting time steps, equilibrium tolerances, solution strategies; (3) revising input ground motions; or (4) fixing problems (bugs) in the software.
2. Calculated responses that exceed the valid range of modeling for either deformation-controlled components or other limits in the analysis formulation (e.g., large displacements or rotations that may not be reliably captured in the kinematic formulation of the model). This can be avoided by ensuring that all deformation-controlled components are reliably modeled for the level of deformations that they experience in the analysis and that other aspects of the analysis are reliable.
3. Force demands are exceeded in a force-controlled component; this can be avoided by modeling the component inelastically as a deformation-controlled component action. Or deformation demands of non-modeled components exceed the deformation at which they can sustain vertical load carrying ability. These non-modeled deformation-controlled components could be added to the structural model, but excessive deformations in such components would likely still trigger

the “valid range model modeling” criterion under Item #2 above (since current models do not typically capture vertical collapse behavior).

The “valid range of modeling” criterion (Item #2 above) is triggered in many cases because structural modeling techniques cannot capture all of the important deterioration and failure modes of the building. This acceptance criterion makes it possible to account for these possible failure modes in the overall performance assessment. Two examples of this are as follows:

- Deformation-controlled components that are included in the structural model. Even when a deformation-controlled component is explicitly modeled, there may still be failure modes that the model cannot adequately capture (e.g., reinforcing bar buckling and fracture in hinge regions of concrete members, post-buckling fracture of a steel reduced beam section). In such cases, a deformation limit should be determined, up to which the component model is deemed to be accurate. If the predicted deformation exceeds this limit in the analysis, then the non-simulated failure is said to have occurred and the response of that analysis should be deemed to be unacceptable.
- Deformation-controlled components that are not in the structural model. There are also often failure modes in elements that are not represented in the structural model (e.g., failures in a gravity-system component that is not in the structural model). In such cases, a drift limit should be identified up to which gravity system failure is not expected (with proper accounting for uncertainties, per Section 6.3.2 or Appendix B). If the drift exceeds the limit, then the gravity system is said to have failed and the response of that analysis should be deemed to be unacceptable.

In Chapter 16 of ASCE 7-16, for analysis under MCE_R ground motions, unacceptable responses (of any type) are limited to a maximum of one in eleven ground motions for Risk Categories I-II and none (out of eleven) are permitted for Risk Categories III-IV.

6.3 Element-Level Acceptance Criteria

For assessment of building safety, element-level acceptance criteria all fall into the category of model simplification acceptance criteria and are primarily intended to provide checks against deterioration or failure modes that are not captured by the nonlinear analysis. These may involve criteria: (1) for maximum forces in elements that are not intended or modeled to respond inelastically; or (2) for maximum deformations in elements that are modeled to respond inelastically, but for which the certain modes of degradation are not captured in the model. A secondary role of element-level criteria could be to enforce requirements to limit damage to the structure, so as to provide a higher level of performance beyond collapse safety. This is the intent, for example, in the service level checks that are specified in the design guidelines for *Tall Building Guidelines*.

The element-level acceptance criteria are typically established to distinguish between the following types of limiting actions in the components:

- **Force-controlled actions.** Any element actions modeled with linear elastic properties are deemed to be force-controlled. This is typically done for cases of brittle components, where inelastic deformation capacity cannot be assured, but can also be used in the case where a ductile component is modeled elastically for purposes of simplifying the modeling.
- **Deformation-controlled actions.** Actions for which inelastic deformation capacity is achievable, with the strength deterioration either being captured by the analysis or otherwise limited by criteria to control the maximum permitted deformations.

In addition to the distinction between force- and deformation-controlled actions, the acceptance criteria may be distinguished by the consequences of component failure on the structural system. For example, ASCE/SEI 7-16 distinguishes criteria between element actions that are deemed critical, ordinary, or non-critical, as follows. Implicit to each of these categories is a reliability target for the specified criteria, i.e., limits on the probability of exceeding the limit state criteria under the specified earthquake ground motion intensity.

- **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.

In reporting component demands, it is important to establish and document how the demand quantities are determined from the analyses. For example, forces that are reported for force-controlled actions are generally associated with either an implied or explicit integration of stresses over a specified region, which depending on the circumstances may be sensitive to modeling and integration assumptions. In addition, certain force (stress) quantities may exhibit high-frequency temporal variations, which may affect how the results are interpreted. Similarly, the deformations (or generalized strains) that are reported for deformation-controlled actions have an implicit gage length (e.g., for axial strain in a shear wall), the definition of which should be consistent with that of the deformation acceptance criteria.

The following subsections provide examples of general requirements applied to component acceptance criteria from ASCE/SEI 7-16 and the *Tall Building Guidelines*. Also included is some discussion of how the component demands more generally relate to structural performance and evaluating the reliability of the nonlinear analysis.

6.3.1 Force-Controlled Actions

Acceptance criteria for force-controlled actions typically involve comparing the component force demands to the member strength. The ratio between the force demand is dictated by the criticality of the component and the expected reliability target.

The following are examples of suggested categorization of force-controlled actions from the Section C16.4.2.1 ASCE/SEI 7-16 where a complete list is provided:

- Critical force-controlled components:
    - Axial compression forces in columns due to combined gravity and overturning forces
    - Combined axial force, bending moments, and shear in column splices
    - Shear in concrete walls with limited ability to redistribute to adjacent wall panels
    - Punching shear in slabs without shear reinforcement
    - Tension in brace and beam connections of braced frames
    - Shear in concrete columns and beams
- Ordinary force-controlled components:
    - Axial tension forces in columns due to overturning forces
    - Shear in steel beams and columns
    - Column base connections
    - Shear in individual panels of concrete walls that can redistribute to adjacent panels
    - Shear and chord forces in diaphragms
- Non-critical force-controlled components: Any component where the failure would not result in either collapse or substantive loss of the seismic resistance of the structure (none listed here).

Associated with each category of component is a demand factor with an implied limit on the probability of the actual demand exceeding the component strength, based on assumed level of variability in the demands and capacities. Section 16.4.2.1 of ASCE/SEI 7-16 provides the following specified load factors for the force-controlled member checks:

$$\gamma I_e(Q_u - Q_{ns}) + Q_{ns} \leq Q_e \qquad (6\text{-}1)$$

where:

γ = based on the criticality of the component (2.0 for critical, 1.5 for ordinary, and 1.0 for non-critical)

I_e = the importance factor as prescribed from Section 11.5.1 of ASCE/SEI 7-16

Q_u = the mean of the peak force demand on the component

Q_{ns} = the mean force demand caused by loads other than seismic

Q_e = the mean expected strength of the component

For critical force-controlled components with I_e equal to 1.0, the load factor in Equation 6-1 is intended to provide an acceptably low probability (<10%) that the force demand will exceed the member strength. This assumes that the variability in force demand and force capacity have lognormal distributions, where each has a dispersion of about 0.4. For ordinary force-controlled components, the probability of exceedance is about 25%, and for non-critical components it is about 50%. These probabilities are smaller for larger importance factors I_e or where the dispersions are less (e.g., by employing capacity design to have well-controlled yielding mechanisms and/or where efforts are made to improve quality assurance in design and construction).

The *Tall Building Guidelines* have similar requirements to ASCE/SEI 7-16, although there are some differences in the consequence categories, the load factors and the use of resistance factors.

The requirements described above pertain to safety limit states, which are typically calculated under MCE_R-level ground motions. Some guidelines, such as the *Tall Building Guidelines*, also impose force-based checks of all members under less intense (more frequent) ground motions to evaluate serviceability conditions. Such checks are usually applied uniformly across all members, without distinctions of critical, ordinary, non-critical, and usually without load or resistance factors.

In most force-controlled component checks, calculation of the component strengths is usually done with the component design equations from standard building code provisions. Expected strengths are usually determined by using expected material strengths in the nominal strength equations, although sometimes additional factors are applied to obtain more realistic mean values of component strengths. Further details regarding expected component strengths for components of specific systems are included in *Part II Guidelines*.

6.3.2 Deformation-Controlled Actions

Criteria for deformation-controlled components are more challenging to develop than criteria for force-controlled components, for the following reasons:

- Standard building code provisions do not typically address deformation limits, so consensus guidelines with consistently established deformation limits are not generally available. Those limits that are available, such as in ASCE/SEI 41, tend to be conservative due to built-in assumptions regarding behavior, such as cyclic loading effects.
- The requirements for deformation-controlled components are less absolute, in the sense that the limits may be related to the capabilities and limitations of the specific nonlinear analysis models that one is using (e.g., see the discussion related to modeling of degradation in Chapter 2).
- The dependence of component strength degradation on cyclic loading response makes it difficult to establish a single criterion that is equally reliable for all loading histories.
- Unless the deformation criteria can be determined based on theory and mechanics, then quantification of the criteria may be limited by the available range of test and/or detailed analysis data.

Most standards generally impose performance limits on component deformations, such as limiting deformations to the point that the components can no longer resist the required gravity loads. In addition, standards may impose limits, based on the range of available test data to validate the performance.

Similar to the criteria for force-controlled components, the requirements for deformation-controlled components include allowances to account for the variability in the deformation demands and capacities. For consistency, it is generally preferable to begin with an estimate of the median value of a limit and then impose a factor (either to amplify the demand or to reduce the limit) to account for uncertainties in the behavior.

The following are examples of how deformation-controlled actions would be categorized based on the consequence of failure of the components (from Section C16.4.2.2 of ASCE/SEI 7-16)

- Critical deformation-controlled components:

- Plastic hinge rotations in beams and columns leading to significant strength/stiffness degradation
- Deformations of non-ductile connections between gravity beams or slabs to supporting columns or walls
- Axial deformations in steel braces
- Axial strains in longitudinal shear wall reinforcement

- Ordinary deformation -controlled components:
  - Deformations of ductile connections between gravity beams or slabs to supporting columns or walls.
- Non-critical deformation -controlled components: Any component where the failure would not result in either collapse or substantive loss of the seismic resistance of the structure.

For evaluation of designs under MCE_R ground motions, the Section 16.4.2.2 of ASCE/SEI 7-16 specifies that deformation criteria should be determined either from the collapse prevention (CP) limits of ASCE/SEI 41, or alternatively, by mean values determined from tests. In the case of the latter, ASCE/SEI 7-16 further specifies a capacity reduction factor, ϕ_s, that should be applied to the mean deformation limits. Thus, according to ASCE/SEI 7-16, the deformation capacity should be checked as follows:

$$Q_u < \phi_s Q_{ne} \text{ or } Q_{CP} \qquad (6\text{-}2)$$

where:

Q_u = the mean deformation demand from analysis

Q_{ne} = the mean deformation capacity from tests

Q_{CP} = the CP deformation limit from ASCE/SEI 41

ϕ_s = $(0.3/I_e)$ for critical components, $(0.5/I_e)$ for ordinary components, and $(1/I_e)$ for non-critical components

I_e = the importance factor, depending on the ASCE/SEI 7 Risk Category

These criteria in ASCE/SEI 7-16 are based on helping to ensure a less than 10% probability of collapse under MCE_R ground motions, recognizing that the reliability analysis is based on a number of simplifying assumptions. The ϕ_s values are based on assumed lognormal distributions of deformation demands and capacities, where the assumed dispersions are 0.46 for deformation demands and 0.66 for deformation capacities. These are larger than the corresponding dispersions for force-controlled components, owing to the larger uncertainties in estimating the values. For critical components, the specified ϕ_s corresponds to a maximum probability of exceedance of about 10%, with larger exceedance values implied for ordinary and non-critical components.

6.4 Components of the Gravity System

When applying the acceptance criteria to evaluate performance, all components of the gravity system should be checked just like the components of the "lateral system." Ideally, the gravity system would be included directly in the structural model, enabling straightforward checking of the criteria.

If the gravity system components are not explicitly included in the structural model, the failure modes in the gravity system should still be considered in the acceptance criteria checks. This would typically be done by deeming the gravity system components (e.g., slab-column connections) as deformation-controlled components and then imposing an associated acceptance criterion on the drift response experienced by the gravity system. If components of the gravity system are deemed to be force-controlled, these should be included in the structural model and the forces should be checked, unless the demands can be estimated and the analyst can demonstrate that such components will not be overloaded when subjected to ground motions.

6.5 Acceptance Criteria for Performance Metrics beyond Safety of the Structural System

Discussion in the previous sections have focused primarily on the most common checks with pre-defined criteria, which are primarily intended to help ensure adequate building collapse safety (e.g., MCE_R checks in Chapter 16 of ASCE/SEI 7-16 or *Tall Building Guidelines*). Nonlinear dynamic analysis can also be used for completing a more comprehensive performance assessment, including evaluation of damage, economic losses and the associated repair time due to earthquakes. For example, the FEMA P-58 method uses the structural responses (peak transient and residual story drifts and peak floor accelerations) to estimate damage to structural and nonstructural components and then relate that damage to the final performance (or resiliency) metrics. For performance assessments using FEMA P-58 (or similar approaches), the structural acceptance criteria are assessed indirectly, depending on allowable thresholds for loss (e.g., repair cost below 5%, reoccupancy time below 7 days, red tagging probability below 5%). To the extent that the damage and losses depend on the building drifts and deformation or force demands in structural components, in concept, one can limit the losses by imposing specific acceptance criteria. However, at present, there are no well-defined limits, with the possible exception of the serviceability limit state acceptance criteria imposed by the *Tall Building Guidelines*.

6.6 Treatment of Modeling Assumptions through Sensitivity Analysis

The acceptance criteria in this chapter have implicitly assumed that it is sufficient to only assess the mean building response, from which variability about the mean is considered through assumed dispersion in demands and capacities. However, there may be some specific aspects of the structural model where a modeling assumption or decision has large influence on some of the structural responses of interest, one whose influence may have a disproportionate effect on critical demand parameters. Two examples of this include:

- Assumed stiffness for concrete transfer diaphragms and walls, including those at the ground level and subgrade levels (where the back-stay effect is predominant) and any other diaphragm that transfers a substantial amount of load.
- Assumed stiffness and/or damping properties for soil, which can influence both the input ground motions and the flexibility of the support foundations.

In such cases where a specific modeling assumption has a large impact on predicted responses, then it is prudent to perform an upper- and lower-bound sensitivity analysis to directly assess the impact of the modeling assumption and create an estimated range of possible structural responses. Typically, the acceptance criteria would then be enforced for both the upper- and lower-bound cases, to ensure that the building performance is acceptable for the range of possible modeling parameters. This cannot be codified for all type of models and systems, but requirements for this should be established as part of the project-specific criteria for analysis and design.

6.7 Additional Details on Story Drift Calculations

Although the calculation of story drift is generally straightforward, it may sometimes be necessary to also consider "story racking shear demands" that can arise where there is significant differences in vertical deformations across the floor plan area. This often occurs due to flexural deformations in tall shear walls, axial deformations in columns that resist large overturning forces, or rocking of foundations. Depending on the situation, these vertical displacements may increase or decrease the deformations that are imposed on structural and/or nonstructural components.

Shown in Figure 6-1 is an idealized plan view of a building and an elevation view of the racking effects due to lateral drift and vertical floor movement. For illustration purposes, the building is considered to be a tall building with a concrete shear wall surrounded by a gravity framing system; but the concepts described here can generally apply to most systems. The core is intentionally located off-center to highlight situations where the building may undergo both translation and twist. Where twist is significant, the drift should be monitored at multiple points around the floor plan.

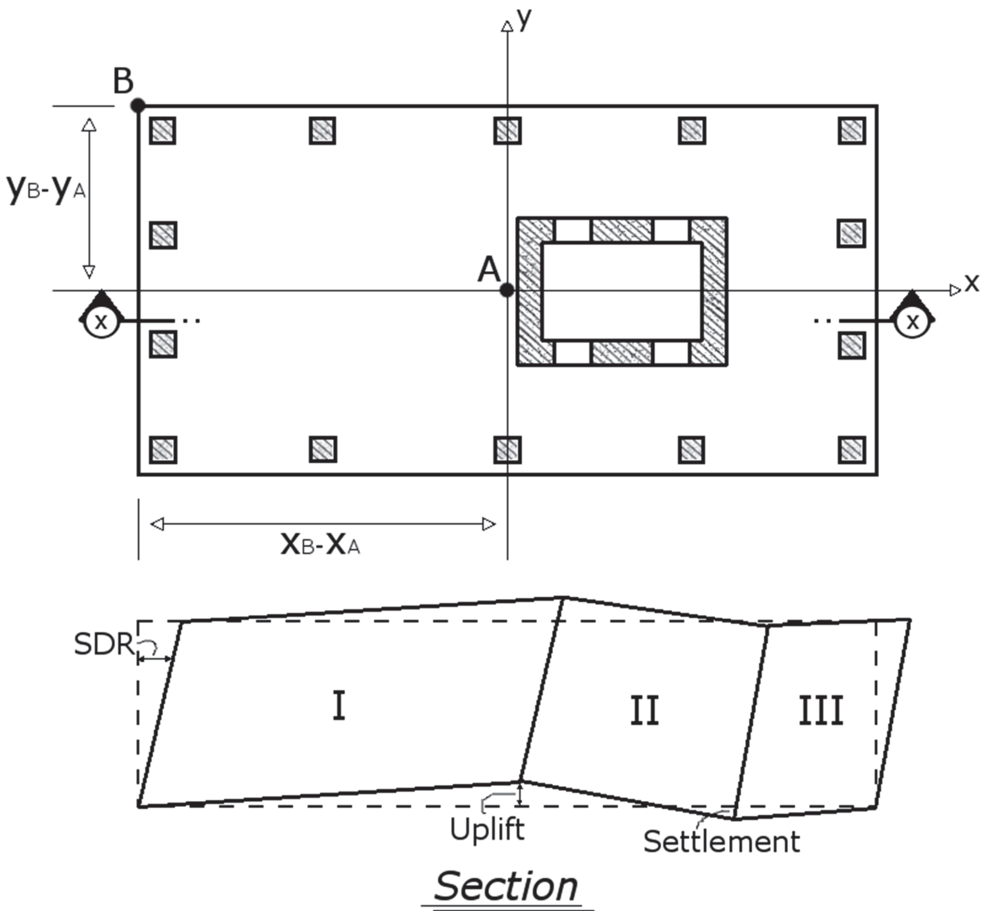


Figure 6-1 Building floor plan and section to illustrate calculation of building drift and amplification of racking drift due to vertical floor deflections.

The drift (and story drift ratio, SDR) at any point in plan can be reported in terms of a drift component in each orthogonal direction, SDR_x and SDR_y, along with a resultant drift, SDR_r, given by the following:

$$SDR_x = (\Delta x_{i+1} - \Delta x_i)/h_i \tag{6-3a}$$

$$SDR_y = (\Delta y_{i+1} - \Delta y_i)/h_i \tag{6-3b}$$

$$SDR_r = \sqrt{SDR_x^2 + SDR_y^2} \tag{6-3c}$$

The average story drift in each orthogonal direction would be the average of SDR_x and SDR_y values for representative nodes on the building floor plan.

Assuming that the floor diaphragm is fairly rigid, the story rotation angle (SRA) can be calculated as:

$$SRA_k = \frac{h_i\left(SDR_{r,B} - SDR_{r,A}\right)}{\sqrt{\left(X_B - X_A\right)^2 + \left(Y_B - Y_A\right)^2}} \tag{6-4}$$

where X and Y refer to the plan dimensions between points A and B in plan (Figure 6-1).

The racking story shear deformation, shown in Figure 6-1, is illustrated in the story cross-section where the shear distortion varies across the story due to the combined effects of the imposed story drift and the relative vertical floor deflections. In this example, the vertical deflections are associated with uplift on the tension side and settlement on the compression of the concrete shear wall. As illustrated in Figure 6-1, the floor uplift has the effect of increasing the story shear racking distortion of Panel I, decreasing it in Panel II, and slightly increasing it in Panel III. In this example, partition walls attached between two floors in Panels I and III would experience larger shear deformations (and damage) than would be implied by the overall story drift ratio. On the other hand, partitions in Panel II, would experience less deformation. The vertical deflections can also increase the deformations demands in the slab sections that connect shear walls to gravity columns. Conversely, the vertical deformations associated with flexural deformations may tend to reduce the structural damage, relative to what might be inferred by the observed drift (e.g., consider that flexure-dominated shear walls may have large story drifts in upper stories where there is little deformation of the wall itself). The specific effects will depend on the building configuration and the location of vertical members (walls, columns) that are resisting axial stress/strains due to overturning.

The racking ratio, RR, may be calculated for individual "story panels" that extend the story height and are defined by four corner points. An idealized panel of story height h_i and length l_k is shown in Figure 6-2. Assuming negligible axial deformations in the panel, the racking drift ratio in each direction (k equals x, y, or r) can be calculated by the following:

$$\mathrm{RR}_k = \mathrm{SDR}_k - (\Delta z_a - \Delta z_c)/l_k \tag{6-5}$$

In cases with minimal axial deformations ($\Delta_z = 0$), RR_k is equal to the SDR_k. On the other hand, when the vertical rotation angle is equal to the story drift ratio, then RR_k becomes equal to zero. In many cases, however, the vertical deformations will increase the RR_k to some value greater than SDR_k.

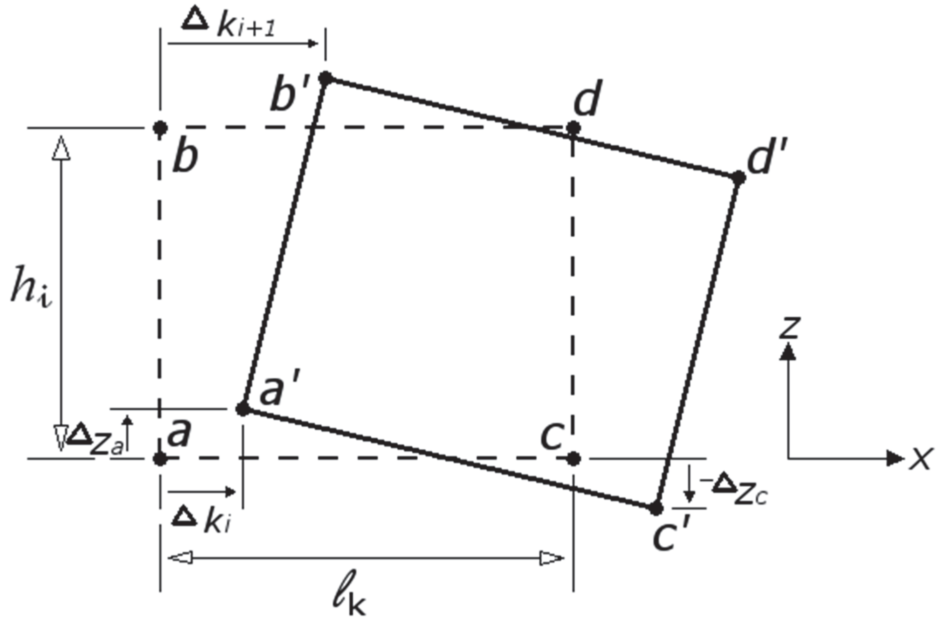


Figure 6-2 Schematic elevation view of building story panel for calculation of story drift ratio (SDR) and racking ratio (RR).

Appendix A

Overview of Methods for Response-History Analysis

Table A-1 provides a summary of contemporary methods for response-history analysis. These are codes or guidelines that specify how design should be done using response-history analysis (e.g., rules on ground motions, acceptance criteria), but they intentionally do not provide detailed instructions on how the nonlinear modeling should be done (they only provide high level requirements). The purpose of this *Guideline* is to complement these other documents by providing detailed instructions building a nonlinear structural model. Accordingly, it is expected that this *Guideline* typically be used in concert with one of the code or guideline documents listed in Table A-1.

Table A-1 is intended for illustration and should not be used as a replacement for requirements specified by these governing document.

Table A-1 Overview of Contemporary Response-History Analysis Guidelines (after Haselton et al. 2017)

Components of the Response-History Analysis	ASCE/SEI 7-16 (2017)	ASCE/SEI 41-13 (2014)	LATBSDC (2015)	PEER TBI (2010)	FEMA P-58 (2012)
Goals and Ground Motion Levels					
Explicit Goals of Assessment	P[C] < 10% for MCE_R; 1% in 50 year collapse risk (in most regions)	Target performance level for each level of seismic hazard (e.g., "Collapse Prevention in BSE-2")	Well-defined NL behavior, func. for service motion, low P[C|MCE]	P[C] < 10% at MCE, low resid. drift, low cladding failure risk	Estimates of loss, repair times, and casualties
Ground Motion Intensity Measure	S_{a_maxDir}	S_{a_maxDir}	S_{a_maxDir}, per ASCE7-10, $S_{a\_geoMean}$ for service level	$S_{a\_geoMean}$ or S_{a_maxDir}, ASCE7-05/10	Typically $S_{a\_geoMean}$
Ground Motion Level	MCE_R	MCE_R, 2/3 MCE_R, 5% or 20% in 50-yr	MCE, service-level	MCE, service-level	From service-level to high-level
Target Spectrum					
General approach	UHS or multiple CMS, risk adjust	UHS with or without risk adjustment	UHS or multiple CMS, risk adjust	UHS or multiple CMS, risk adjust	Any are allowable
Ground Motion Selection					
Number of motions	≥11 pairs	Varies	≥7 pairs	≥7 pairs	≥7 pairs
Scaling/Modification of Motions to Match Target Spectrum					
Specifics for far-field sites	Match to target, enforce 90% floor	SRSS is above target, per ASCE7-10	None	None	-
Specifics for near-fault sites	Same as far-field component	Average of FN is above target	None, per ASCE 7-10	None	General, also check velocities
Period range for matching	$T_{MIN} - 2.0T$, $T_{MIN} = 0.2T$ or 90% mass	$0.2T - 1.5T$	$0.2T$ (or $0.1T$) – $1.5T$, ASCE7-10	Not specified	$0.2T$ - $2.0T$

Table A-1 Overview of Contemporary Response-History Analysis Guidelines (after Haselton et al. 2017; continued)

Components of the Response-History Analysis	ASCE/SEI 7-16 (2017)	ASCE/SEI 41-13 (2014)	LATBSDC (2015)	PEER TBI (2010)	FEMA P-58 (2012)
Application of Ground Motions to Structural Model					
Far-field sites	Arbitrarily orient motions; no need for multiple orientations of GMs	Just apply horizontal motions together; no rules for orientation	Orient motions randomly; no multiple orientations of GMs	"Apply along principle directions" (but no GM rotation)	Not specified
Near-fault sites	Apply in FN and FP directions; no need for multiple orientations of GMs	Apply FN/FP if site < 5km from fault	Apply in FN and FP directions	Apply in FN & FP directions if directivity dominates	Not specified
Vertical Ground Motions	Included in rare cases	Include for specific cases	Not considered, per ASCE7-10	Included in rare cases	Not specified
Response Metrics and Acceptance Criteria (typically at MCE or 2/3 MCE)					
Peak story drifts	μ < 2*limit; no max check	No limit	$\mu < 0.03$	$\mu < 0.03$, max < 0.045	Mean and variability
Residual story drifts	No limit	No limit	$\mu < 0.01\kappa_{i,max} < 0.015\kappa_i$	$\mu < 0.01$, max < 0.015	Mean and variability
Deformation-controlled actions	μ < limit, where limit is ASCE 41 CP / I_e, alternative limits allowed	μ < limit	μ < limit	No limit (except within reliable analysis range)	Used only for advanced fragilities
Force-controlled actions (critical, no well-defined mech.)	$2.0\mu I_e \leq F_{n,e}$	$\mu < F_{n,lower\text{-}bound}$	$1.5\mu \leq \kappa_i F_{n,e}$	$1.5\mu \leq \phi F_{n,e}$	Used only for advanced fragilities
Force-controlled actions (critical well-def. mech., or ordinary)	$1.5\mu I_e \leq F_{n,e}$		$1.5\mu \leq \kappa_i F_{n,e}$	$\max(\mu + 1.3\sigma, 1.2\mu) \leq \phi F_{n,e}$	
Force-controlled actions (non-critical)	$1.0\mu I_e \leq F_{n,e}$		$\mu \leq \kappa_i \phi F_{n,e}$	$\mu \leq F_{n,e}$	
Treatment of collapse or unacceptable response cases	Not more than 1/11 motions have unacceptable response	Not discussed, average drift limits suggest collapses are not allowed	Not discussed, average drift limits suggest col. not allowed	Collapses are not allowed	Collapses are specifically treated
Peak floor accelerations	Used for nonstructural component design (typically per Chapter 13 simplified calculations)	None	None	None	Mean and variability

Appendix B
Consideration of Uncertainties

B.1 Overview of Uncertainties in the Performance Assessment Process

In contrast to conventional prescriptive seismic design where the underlying uncertainties in ground motion hazard and structural response are suppressed, when assessing seismic performance through use of nonlinear response-history analysis, the uncertainties are more obvious and transparent. Much of the uncertainty is inherent to the seismic design and response of buildings, regardless of the type of analysis that one performs, such as in the variability in the earthquake hazard, structural properties, and structural response. On the other hand, some uncertainties are introduced by the structural analysis models and procedures themselves, including the extent to which the analysis process faithfully represents the ground motion hazard and structural behavior.

Comparisons of analysis and laboratory tests consistently show that even for well-controlled laboratory tests, the response quantities determined by nonlinear dynamic structural analysis often differ from experimental measurements by 30% to 40% (e.g., the study described in Ohsaki, 2014) with large differences in response predictions between different analysts. Similarly, the variability in structural response between different ground motion records (i.e., record-to-record variability) is also large. Accordingly, the structural response predictions cannot be treated deterministically and these large modeling uncertainties must be appropriately handled in the performance assessment (or design) process.

Figure B-1 provides an overview of the various uncertainties involved in a complete performance assessment process (based on the FEMA P-58 methodology; FEMA, 2012) and provides commentary to the right regarding how each step is typically handled in most assessment processes (e.g., ASCE/SEI 7-16). This shows that uncertainties are typically handled only in an approximate manner in common performance assessment approaches.

B.2 Rigorous Evaluation of Uncertainties

The ideal approach would be to handle uncertainties rigorously by quantifying the uncertainties at each step of the assessment process (per Figure B-1) and to propagate these uncertainties throughout the assessment process. This would provide a full distribution (mean and variability) of the final desired performance metric (e.g., repair cost, onset of safety issue) and then performance acceptability could be judged based on an acceptance criterion with a desire level of prediction confidence. While this is preferable, this is a formative task and is currently still very much in the research realm.

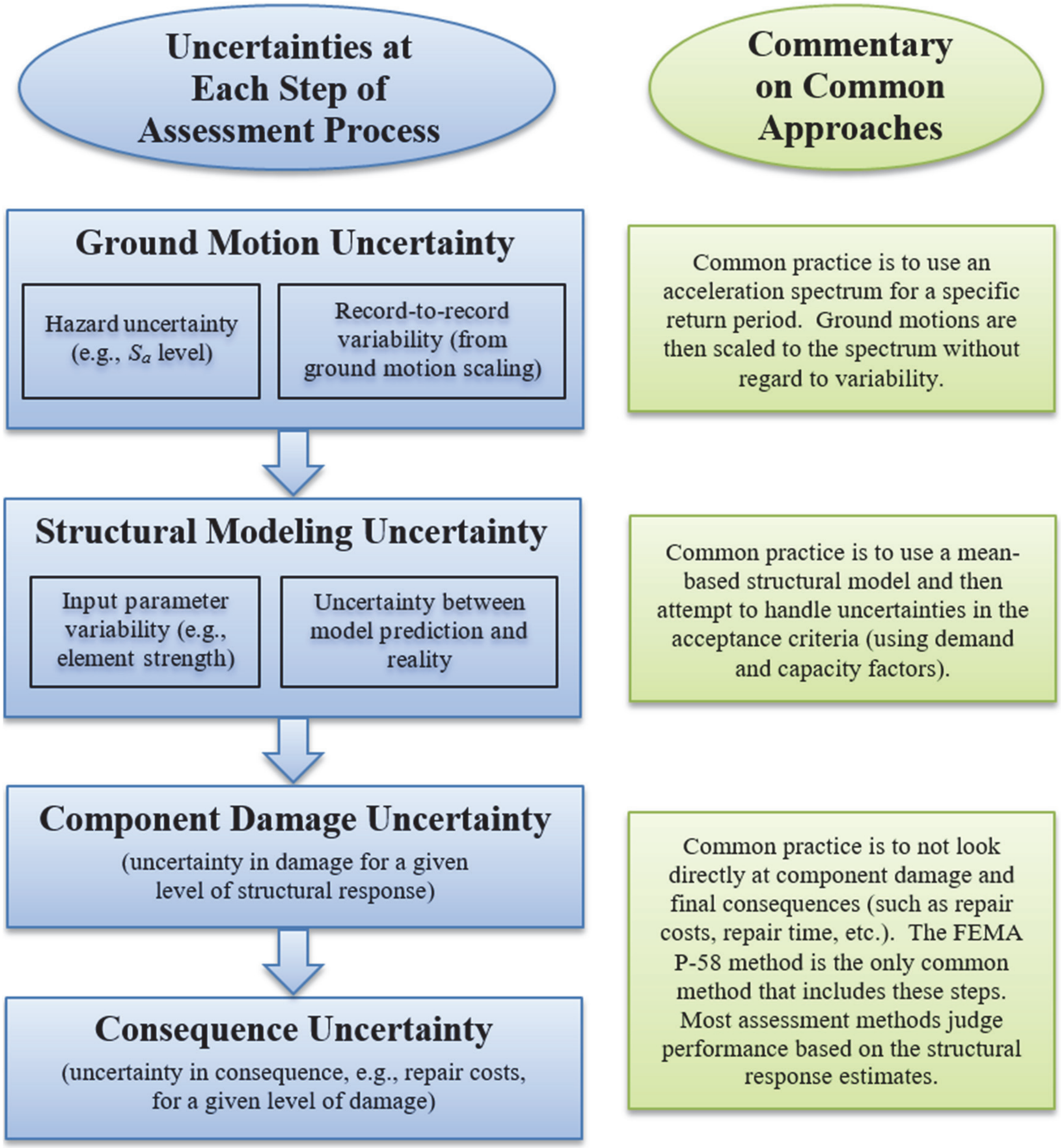


Figure B-1 Overview of the uncertainties at each step of the performance assessment process.

The only common performance assessment methodology that handles uncertainties in this manner is the FEMA P-58 methodology, in which uncertainties are tracked at each step of the performance assessment and Monte Carlo simulation is used to propagate the uncertainties. Yet, even in this method, the amount of variability in each step is often estimated, based on a combination of available data and judgment.

Although conceptually straightforward, this process is not used in most common performance assessment methods because the following necessary inputs are challenging to quantify: (1) variability in the uncertain parameters at each step of the process; and (2) correlations between the various modeling parameters (which is not well understood and which has large effect on the final variability in the final performance metric. Additionally, such an approach requires establishing appropriate reliability targets for various performance limits and hazard levels.

B.3 Typical Approach to Treatment of Uncertainties and Appropriate Interpretation of Structural Response Predictions

The typical performance assessment procedures (e.g., ASCE/SEI 7-16) used in engineering practice focus on estimating the mean structural responses and typically involves steps like the following:

- Create a mean-based structural model (i.e., one where the structural modeling parameters are calibrated to mean values).
- Select one or more intensity-based target spectra to represent the ground motion hazard. Usually these are based on a mean spectrum from a probabilistic seismic hazard analysis with a specified return period or a spectrum from a deterministic earthquake scenario.
- Select either 7 or 11 ground motions that are either scaled or spectrally matched to a target spectrum (note that 7 or 11 records is the minimum for determining a reasonable median or mean value of response; a suite of 30 or more motions would be required to provide a meaningful estimate of variability, i.e., standard deviation, about the median or mean).
- Scale (or match) the ground motions to meet or exceed the target spectrum; in this scaling (or matching) process, typically only the mean spectrum is controlled and variability between ground motion spectra is not explicitly controlled.

The approach outlined above is geared toward estimating mean response quantities. As such, one is cautioned against trying to make inferences about the variability in response or large response outliers. Observations of responses that are much larger than the mean response (or, conversely, not a large response) are statistically insignificant because the performance assessment process does not attempt to handle the uncertainties at several important steps of the assessment process (as listed above). The observance or non-observance of such large responses will depend heavily on how the ground motions were selected and scaled (or spectrally matched) to meet the target spectrum. To rigorously evaluate uncertainties in response would require more analyses to consider variability in both the ground motions and structural modeling parameters.

The above rationale also applies to the interpretation of individual analyses that may cause excessive drifts or other unacceptable response cases (e.g., non-convergence). Although it is useful to explore reasons for outliers in the set of analyses, from a statistical standpoint, one should be careful to not read more into the limited statistics from a small set of analyses (e.g., 7 or 11 sets of analyses).

When using performance assessment procedures as specified in current design standards (ASCE/SEI 7-16, PEER (2017), LATBSDC (2015)), the mean structural responses should be used as the basis for evaluating the performance of the building and acceptance criteria should be based on mean responses. Any acceptance criterion that is based on a peak response should be used with caution and should only be a secondary check of acceptable performance (as is done with unacceptable responses in ASCE/SEI 7-16). Otherwise, where it is appropriate to consider more extreme values, it is statistically more reliable to use standardized multipliers on the mean demands or mean capacities, which are described in the next section.

B.4 Consideration of Uncertainties when Establishing the Acceptance Criteria

B.4.1 Overview

When the performance assessment process is structured based on mean building responses, then the primary mechanisms to account for modeling and other uncertainties are: (1) “demand factors” applied to the median responses; (2) “capacity factors” applied to the acceptance criteria; and/or (3) some other factor of safety included in the acceptance criteria. This is necessary to ensure that the building meets the intended performance goals, accounting for all important sources of uncertainty, with an adequate level of reliability.

Figure B-2 shows a conceptual example of the distributions of demand and capacity of a structural component. The acceptance criteria, including factors to separate the demand and acceptance criteria, must be structured to achieved an appropriate probability of failure of such component (or system), considering the uncertainties in both demand and capacity. In this example, the mean capacity is required to be an appropriate factor larger than the mean demand, such that the probability of sudden failure of force-controlled component actions is less than 10%.

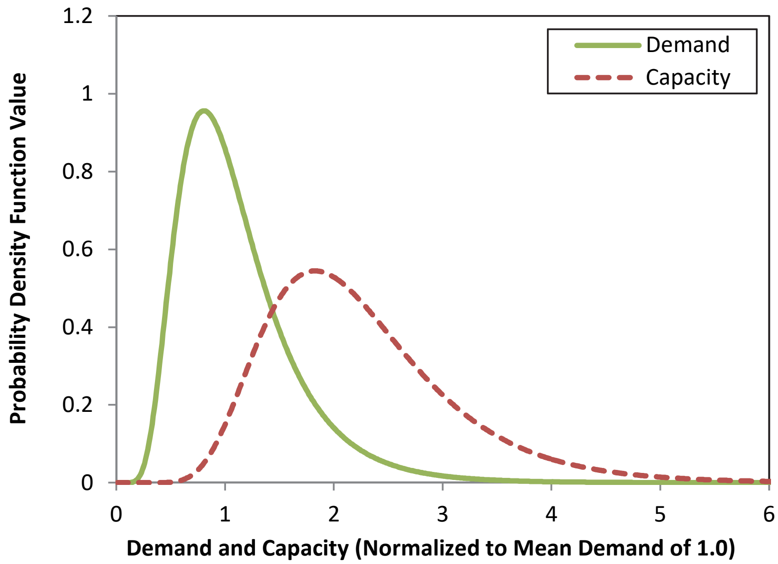


Figure B-2 Illustration of lognormal distributions for component capacity and component demand (normalized to a mean demand of 1.0); the mean component capacity is calibrated to achieve P[Failure|MCE$_R$] = 10% for a force-controlled component.

B.4.2 ASCE/SEI 7-16 Chapter 16 Examples for Force-Controlled and Deformation-Controlled Components

An example of the process of creating acceptance criteria that appropriately account for various contributing uncertainties can be found in the Section C16.4 of ASCE/SEI 7-16.

Tables B-1 and B-2 show the uncertainties in demand and capacity used for forces in non-ductile force-controlled components, and Tables B-3 and B-4 show the uncertainties in demand and capacity used for deformations in ductile deformation-controlled components. In each of these tables, the reported dispersion (equal to the standard deviation of the natural logarithm of the data, assuming a lognormal distribution) is analogous to a coefficient of variation (i.e., standard deviation divided by the mean). As described in Section C16.4 of ASCE/SEI 7-16, the demand and capacity factors specified in Chapter 16 of ASCE/SEI 7-16 were then calibrated based on a target reliability, considering the expected performance (based on building importance group), consequences of failure (based on the type of component) and the assumed dispersion in demands and capacities (Table B-1 through Table B-4).

When other acceptance criteria are created, they should be created similarly, such that the acceptance criteria account for the important uncertainties and ensure that the building performance goals are achieved with the appropriate level of reliability.

Table B-1 Variability and Uncertainty Values for the Component Force Demand

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

Table B-2 Variability and Uncertainty Values for the Component Force Capacity

Capacity Dispersion (β_C)	Variabilities and Uncertainties in the Final As-Built Force Capacity of the Component
0.30	Typical variability in strength prediction equation (from available data)
0.10	Typical uncertainty in strength prediction equation (extrapolation beyond available data)
0.20	Uncertainty in as-built strength due to construction quality and possible errors
0.37	$\beta_{C\text{-TOTAL}}$

Table B-3 Variability and Uncertainty Values for the Component Deformation 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 in estimating mean deformation demands from structural model (modeling uncertainty)
0.13	Variability in estimating mean deformation demands from only 11 ground motions
0.46	$\beta_{D\text{-TOTAL}}$

Table B-4 Variability and Uncertainty Values for the Component Deformation 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 deformation capacity (extrapolation beyond data)
0.20	Uncertainty in as-built deformation capacity due to construction quality and errors
0.66	$\beta_{C\text{-TOTAL}}$

B.4.3 Acceptance Criteria for Other Structural Responses

Similar to the above example for force- and deformation-controlled components, acceptance criteria could also be set for other structural response parameters of other performance metrics. This could be done for other structural response metrics, such as the following:

- Peak story drifts (to limit non-structural damage)
- Residual story drifts (to limit chance of building being torn down after earthquake)
- Peak floor accelerations (to limit damage to mechanical, electrical, and plumbing)
- Collapse ground motion level (to establish appropriate level of safety)

A similar process would need to be used where the uncertainties in the responses are quantifies, the reliability target is set for the response type, and the acceptance criterion is calibrated accordingly.

Alternatively, acceptance criteria could instead be based on more complete performance metrics (i.e., the final step in Figure B-1), such as the following:

- Building repair costs
- Building recovery time (downtime)
- Occupant fatalities and injuries

This is the approach used in the FEMA P-58 performance assessment methodology. Such acceptance criteria could be similarly developed by accounting for the uncertainties in all the steps of the performance assessment process (i.e., all steps shown in Figure B-1).

Appendix C

Advanced Calibration of Nonlinear Component Models

Parameters to construct numerical models of structural components can be obtained from these *Guidelines* or similar sources. The parameters recommended here are based on test data and engineering judgment. In the case of components not considered in these or similar guidelines, modeling parameters must be estimated (or calibrated) from test data. This chapter provides guidance on what to do in this situation.

Ideally, the calibration of modeling parameters should be based exclusively on test data. There may be instances in which test data may need to be extrapolated to other cases. Detailed finite element analyses (FEA) can be an option to extrapolate test data. See Chapter 6 of *Part II Guidelines* for information on advantages and limitations of detailed FEA.

Whether calibration is done using test data, extrapolations obtained with detailed analyses, or a combination of the two, it is important that the loading and boundary conditions in the tests and analyses represent the conditions of the component being idealized as explained in the next two sections.

C.1 Characteristics of the Supporting Tests or Analyses

It is important that the loading and boundary conditions in the tests represent the conditions of the component being idealized. The same is true for material properties (related to both strength and quality). Properties that may appear to be irrelevant at first sight (such as aggregate type and size in reinforced concrete elements, and the shapes of stress-strain curves) may be critical (Daluga, 2015). The directions in which specimens are loaded can be important also.

In the case of reinforced concrete elements, experimental results from small specimens should be treated with care because quantities related to bond, shear, and the tensile strength of concrete cannot be projected directly from small to large elements.

The governing building code may limit the size and other properties of specimens to be used to justify design assumptions. In the case of steel structures, ANSI/AISC 341, *Seismic Provisions for Structural Steel Buildings*, include detailed requirements for tests to be used to show that a connection type has the required toughness.

C.2 Loading Protocols for Calibration of Strength and Stiffness Deterioration

The loading protocol of the tests used for calibration should include cycles with peak displacements or rotations not smaller than peak deformations estimated in preliminary analyses.

Despite their apparent conservatism, tests with displacement cycles of increasing amplitude applied in at least two opposing directions along one or more axes are preferred over "monotonic" tests.

Section 2.3 illustrated that differences in loading protocol can affect test results. Although it is seldom possible, it would be ideal to have data from tests in which multiple loading protocols are used for nominally identical test specimens. Depending on the properties of the expected demands, the following are options to be considered in selecting displacement protocols for a future test program. An example of such a testing program is also summarized in Section 4.2.3.6 of *Part IIb Guidelines*.

- **Monotonic protocol.** The best component modeling approaches discussed in Section 2.3 require information on monotonic response. Inferring monotonic response from cyclic test results is difficult. Data from monotonic tests would help circumvent this difficulty.
- **Various symmetric cyclic loading protocols.** Tests have shown that repetitions of displacement cycles of constant amplitude can affect response. Large numbers of repetitions can cause decay at smaller displacements. Large numbers of repetitions can also initiate fracture (in a phenomenon often referred to as low-cycle fatigue). Having data from tests with different numbers of repetitions would help the engineer understand to what extent this effect may be critical.
- If the response is expected to be dominated by a single excursion in a single direction, data from protocols with a few cycles followed by a "monotonic push" may be more relevant.
- Protocols in which displacement cycles are applied in two or more different directions can be more demanding than tests in which displacements are applied along a single direction.

The amplitudes of the applied cycles should encompass the expected ranges of deformation.

Calibration of parameters should concentrate on reproducing three main properties: initial stiffness, strength, and deformation capacity. Other properties controlling the shape of the hysteretic loops are not as critical especially for structures with relatively long periods (Shimazaki and Sozen, 1984). Defining deformation capacity is not simple. In fact, different authors refer to it with different names. In these *Guidelines*, the term "capping rotation" is used to refer to the deformation level at which strength starts decaying. In general, drift capacity is quantified as the displacement or rotation at which the peak point (at Q_u) in Figure C-1 is reached. Nevertheless, the peak point is not always as clear in a measured load-deformation curve as it is in Figure C-1. And there is uncertainty about the repeatability of test results and their sensitivity to the chosen load protocol. It is common to estimate displacement capacity as the maximum deformation reached before a "strength loss" of 20% or more is measured on a "backbone" curve enveloping the measured load-displacement cycles. ASCE/SEI 41-13 (ASCE, 2014) recommends obtaining this backbone by joining with straight lines "the intersection of the first cycle curve for the *i*-th deformation step with the second cycle curve of the $(i\text{-}1)^{th}$ deformation step." Where more than one set of test data is available and all tests are deemed to be equally representative of the expected conditions, the simulation parameters should generally be calibrated to match the median results and provide an unbiased estimate of response. It is also useful to record and report the variability between multiple tests, either through a statistical dispersion parameter (e.g., coefficient of variation) when sufficient data are available, or at least the range of the parameters for small data sets.

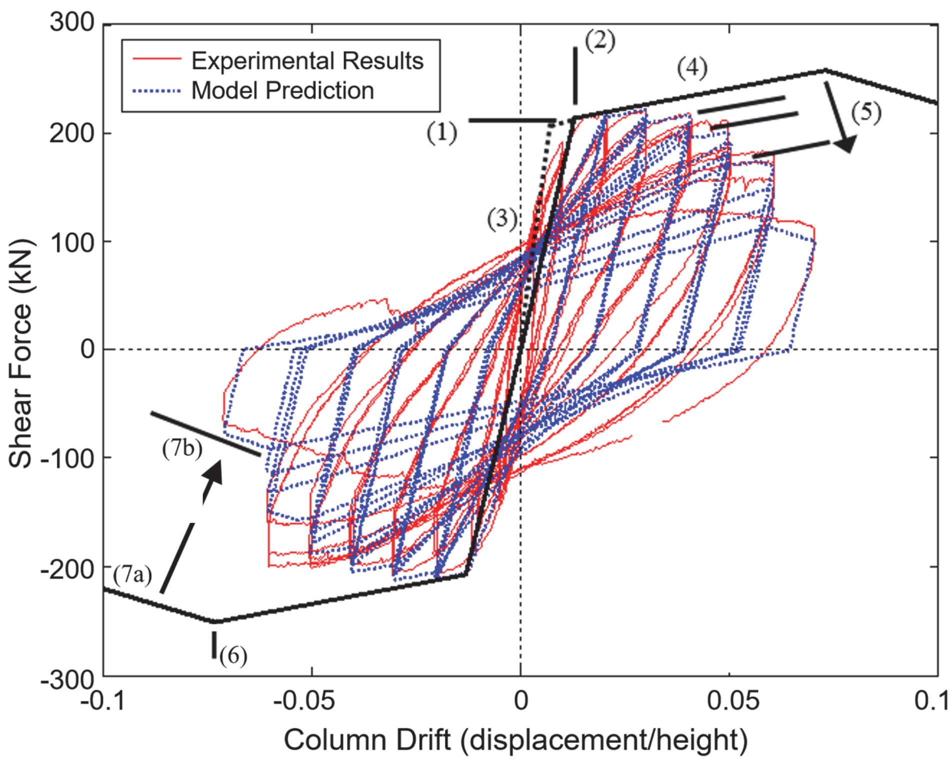


Figure C-1 Calibrated cyclic component response for Saatcioglu and Grira (1999) specimen BG-6, which illustrates the steps in the calibration procedure. The black backbone shows the monotonic loading curve for the calibrated model. Figure is from Haselton et al. (2016) (1 kN force = 0.2258 kips).

C.3 Example

Ideally, the monotonic response curve should be calibrated first using monotonic test data, and then used together with cyclic test data to calibrate the hysteretic and degradation model parameters. However, the vast majority of available test data have employed symmetric cyclic loading, with few if any monotonic tests. In such cases, a joint calibration of the monotonic backbone and cyclic deterioration parameters is required, where the monotonic curve is inferred from the cyclic data. Figure C-1 provides an example of a calibration procedure for the case where cyclic symmetric loading is used to jointly calibrate the model parameters for both the monotonic curve and the hysteretic degradation. In doing any model calibration, it is important that the available data and calibration approach are not missing behavioral and failure modes that may be different under different loading histories.

Figure C-1 illustrates a series of steps in a typical calibration process when using cyclic test data. First, the yield force (Point 1) and displacement (Point 2) values are estimated by visually fitting a secant yield stiffness and post-yield hardening stiffness (Line 4), paying particular attention to the point where there is a significant observed change in lateral stiffness. The initial stiffness, K_e, defined by the linear response up to Points 1 and 2, is then used to back-calculate an effective secant yield stiffness EI_y for the member, based on the length and boundary conditions in the test specimen. Note that the dashed Line 3 could be used to define an effective stiffness that may be more appropriate for service level response, where Line 3 could be defined with a slope that goes through the measured displacement at 40% of the yield force. These four calibration steps are used to determine parameters that define the component strength and stiffness behavior prior to degradation.

Next, the cyclic strength degradation parameters are calibrated. In this example, the primary calibration criterion was to match the average deterioration observed over the full displacement history, with slightly more emphasis on matching the deterioration rate of the more damaging cycles at large deformations. This calibration was based on the observed strength deterioration in the hardening portion of the backbone curve prior to the peak point (Lines 5 in Figure C-1) and this same strength deterioration rate is assumed for the post-capping portion of the curve.

The final step (Point 6 and Line 7) of the calibration involves determining the monotonic capping rotation, $\theta_{cap,pl}$, and the post-capping rotation capacity, θ_{pc}. Point 6 shows the monotonic capping rotation point and Line 7a shows how the component would behave post-capping if subjected to a monotonic load. Line 7b is similar to Line 7a, but shows how the component behaves post-capping when subjected to the displayed cyclic loading protocol (i.e., the post-capping behavior after the component has been cyclically deteriorated). In test data where post-peak negative stiffness is observed, model parameters for the capping and post-capping rotations can be determined by visual fit to the data. However, in many tests, the maximum applied deformations are not large enough for a capping point to be observed (i.e., negative tangent stiffness are not observed in the tests). In such cases, a lower-bound estimate of the capping rotation, $\theta_{cap,pl}$, should be calibrated and the resulting component model should not be trusted much beyond tested levels of deformation.

Test data may not always be “corrected” for P-delta. In that case, if the model is calibrated with such data, P-delta may be “double counted” when second order geometric effects are included in the analysis. Care should be taken that component test data used for modeling do not already include a P-delta.

For a few other examples, please also see Nojavan et al. (2015 and 2016), Haselton et al. (2008 and 2016), and Suzuki and Lignos (2015).

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Project Participants

National Institute of Standards and Technology

Steven L. McCabe
Engineering Laboratory (MS8604)
National Institute of Standards and Technology
100 Bureau Drive
Gaithersburg, Maryland 20899

Matthew S. Speicher
Engineering Laboratory (MS8604)
National Institute of Standards and Technology
100 Bureau Drive
Gaithersburg, Maryland 20899

Jay Harris
Engineering Laboratory (MS8604)
National Institute of Standards and Technology
100 Bureau Drive
Gaithersburg, Maryland 20899

Kevin K.F. Wong
Engineering Laboratory (MS8604)
National Institute of Standards and Technology
100 Bureau Drive
Gaithersburg, Maryland 20899

Siamak Sattar
Engineering Laboratory (MS8604)
National Institute of Standards and Technology
100 Bureau Drive
Gaithersburg, Maryland 20899

Applied Technology Council

Jon A. Heintz (Program Manager)
Applied Technology Council
201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065

Veronica Cedillos (Associate Project Manager)
Applied Technology Council
201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065

Ayse Hortacsu (Associate Program Manager)
Applied Technology Council
201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065

Program Committee on Seismic Engineering

Jon A. Heintz (Chair)
Applied Technology Council
201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065

James R. Harris
J.R. Harris & Company
1775 Sherman Street, Suite 1525
Denver, Colorado 80203

Michael Cochran
Thornton Tomasetti
4551 Glencoe Avenue, Suite 350
Marina del Rey, California 90292

James Jirsa
Department of Civil, Architectural and Environmental Engineering
University of Texas at Austin
301 E. Dean Keeton Street, Stop C1700
Austin, Texas 78712

Roberto Leon
Department of Civil and Environmental Engineering
Virginia Tech
102-D Patton Hall
Blacksburg, Virginia 24061

Stephen Mahin
Department of Civil and Environmental Engineering
University of California, Berkeley
721 Davis Hall
Berkeley, California 94720

James O. Malley
Degenkolb Engineers
375 Beale Street, Suite 500
San Francisco, California 94105

Donald Scott
PCS Structural Solutions
811 First Avenue, Suite 620
Seattle, Washington 98104

Andrew Whittaker
Department of Civil, Structural and Environmental Engineering
University at Buffalo
230 Ketter Hall
Buffalo, New York 14260

Project Technical Committee

Gregory G. Deierlein (Project Co-Director)
Dept. of Civil and Environmental Engineering
Blume Earthquake Engineering Center
MC 3037
Stanford, California 94305

Curt Haselton (Project Co-Director)
Haselton Baker Risk Group, LLC
120 W 2nd Street, Suite 3
Chico, California 95928

Stephen T. Bono
Simpson Gumpertz & Heger, Inc.
100 Pine Street, Suite 1600
San Francisco, California 94111

Wassim Ghannoum
Dept. of Civil and Environmental Engineering
The University of Texas at San Antonio
Building: BSE Room 1.328
One UTSA Circle
San Antonio, Texas 78249

Mahmoud Hachem
Degenkolb Engineers
1300 Clay St. Suite 900
Oakland, CA 94612

John D. Hooper
Magnusson Klemencic Associates
1301 Fifth Avenue, Suite 3200
Seattle, Washington 98101

James O. Malley
Degenkolb Engineers
375 Beale Street, Suite 500
San Francisco, California 94105

Silvia Mazzoni
325 Davis Hall
University of California
Berkeley, California 94720

Santiago Pujol
Bowen Laboratory
Purdue University
1040 S. River Rd.
W. Lafayette Indiana 47907

Chia-Ming Uang
Dept. of Structural Engineering
University of California, San Diego
409 University Center
La Jolla, California 90293

Project Review Panel

Tony Ghodsi
Englekirk Institutional
888 S. Figueroa Street, 18th Floor
Los Angeles, California 90017

Jerome F. Hajjar
Dept. of Civil and Environmental Engineering
400 Snell Engineering Center
360 Huntington Avenue
Northeastern University
Boston, Massachusetts 02115

Yuli Huang
Arup
560 Mission Street, Suite 700
San Francisco California 94105

Michael Mehrain
Mehrain Naeim International, Inc.
100 Spectrum Center Drive, Suite 900
Irvine, California 92618

Farzad Naeim
Farzad Naeim, Inc.
100 Spectrum Center Drive, Suite 900
Irvine, California 92618

Charles Roeder
University of Washington
Dept. of Civil Engineering
Seattle, Washington 98195

Thomas Sabol
Englekirk Institutional
888 South Figueroa Street, 18th Floor
Los Angeles, California 90017

Mark Saunders
Rutherford + CHekene
375 Beale Street, Suite 310
San Francisco, California 94105

John Wallace
5731C Boelter Hall
Dept. of Civil and Environmental Engineering
University of California, Los Angeles
Los Angeles, California 90095

Kent Yu (ATC Board Contact)
SEFT Consulting Group
4800 SW Griffith Drive, Suite 100
Beaverton, Oregon 97005

Working Group Members

Dustin Cook
Haselton Baker Risk Group, LLC
120 W 2nd Street, Suite 3
Chico, California 95928

Ian McFarlane
Magnusson Klemencic Associates
1301 Fifth Avenue, Suite 3200
Seattle, Washington 98101

Gulen Ozkula
Dept. of Structural Engineering
University of California, San Diego
409 University Center
La Jolla, California 90293

Hee Jae Yang
Magnusson Klemencic Associates
1301 Fifth Avenue, Suite 3200
Seattle, Washington 98101

Zhi Zhou
Simpson Gumpertz & Heger, Inc.
100 Pine Street, Suite 1600
San Francisco, California 94111