SEISMIC DEISGN GUIDELINES FOR TALL BUILDINGS

Developed by the

Pacific Earthquake Engineering Research Center

Under its

Tall Buildings Initiative

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SEISMIC DESIGN GUIDELINES FOR TALL BUILDINGS

Dr. Yousef Bozorgnia, PE
Associate Director, Pacific Earthquake
Engineering Research Center

James O. Malley, SE
Senior Principal
Degenkolb Engineers, Inc.

Dr. C.B. Crouse, GE
Principal Engineer/Vice President
URS Consultants, Inc.

Dr. Jack P. Moehle, PE
Director, Pacific Earthquake
Engineering Research Center

Ronald O. Hamburger, SE
Senior Principal
Simpson Gumpertz & Heger, Inc.

Dr. Farzad Naeim, SE
Vice President & General Counsel
John A. Martin & Associates, Inc.

Ronald Klemencic, SE
President
Magnusson-Klemencic Associates, Inc.

Dr. Jonathan P. Stewart, PE
Professor
University of California at Los Angeles

Dr. Helmut Krawinkler, PE
Professor Emeritus
Stanford University

Glossary

Capacity Design – a design approach wherein the structure is configured to concentrate yielding and inelastic behavior in specific locations where elements are detailed to reliably exhibit such behavior, and which, through their ductile behavior, limit the demands on other portions of the structure that are designed with sufficient strength to remain essentially elastic during earthquake response.

Capping Strength – the peak strength attainable by a structural component under monotonic loading

Expected Strength **–** the probable peak strength of a structural element considering inherent variability in material strength and strain hardening

Hazard Curve – a plot of the mean annual frequency of exceedance of a ground motion intensity parameter as a function of the ground motion intensity parameter

Hazard Level – a probability of exceedance within a defined time period (or return period) at which ground shaking intensity is quantified

Lower-bound Strength **–**the probable minimum strength that a structural element might develop considering potential variability in material strength and workmanship

Maximum Considered Earthquake Shaking – the level of shaking specified by the *ASCE-7* standard as a basis for derivation of design ground motions

Monotonic Loading – Time-variant unidirectional, increased force of displacement loading of a structure or structural element without unloading or reloading

Peak Strength – The maximum resistance an element will develop under a specific loading protocol

Return Period – Over a period of many years, the average number of years between repeat occurrence of events having an intensity that is equal to or greater than a specified value. It is approximately equal to the inverse of the mean annual frequency of exceedance.

Service Level Earthquake Shaking – ground shaking represented by an elastic, 2.5%-damped, acceleration response spectrum that has a mean return period of 43 years, approximately equivalent to a 50% exceedance probability in 30 years

Site Response Analysis - analysis of wave propagation through a nonlinear soil medium used to assess the effect on spectral shape of local geology

Uniform Hazard Spectrum – a site-specific, acceleration response spectrum constructed such that the ordinate at each period has the same exceedance probability or average return period

Notation

A_G *Gross cross section area for a concrete section*

C_d a deflection amplification coefficient specified by the *ASCE-7* standard

D Dead Load on a structural element including the effects of the structure's self weight and permanently attached equipment and fixtures as defined in the *ASCE-7* standard

E Demands associated with earthquake effects, including displacement, force, drift, strain, etc., as determined from nonlinear response history analysis

E_S Modulus of elasticity for steel, taken as 29,000 kips per square inch

E_c Modulus of elasticity for concrete

E_x Demands from earthquake effects, including displacement, force, drift, strain, etc., resulting from earthquake shaking applied along the principal axis of building response designated as the x axis

E_y Demands from earthquake effects, including displacement, force, drift, strain, etc., resulting from earthquake shaking applied along an axis that is orthogonal to the x axis

F_c Peak (*c*apping) strength of an element under monotonic loading

$F_{n,e}$ Nominal strength computed using applicable material standard strength formulations, but using expected material strength rather than nominal or specified strength

F_r Post-peak residual yield strength under monotonic loading

F_y Effective yield strength of a component under monotonic loading

F_u Strength demand from a suite of nonlinear response history analyses used to evaluate the adequacy of components with brittle failure modes

G_S Shear modulus for steel, taken as 11,500 kips per square inch

G_c Shear modulus for concrete

I_G Gross moment of inertia for a concrete section

IM A ground motion intensity measure such as peak ground acceleration, spectral response acceleration at a particular period, etc.

K_e effective elastic stiffness

K_p effective post-yield stiffness under monotonic loading

K_{pc} effective post-peak strength stiffness under monotonic loading

L Live load on a structural element taken as the design or maximum "point in time" live load (without reduction) per the *ASCE-7* standard

L_{exp} that portion of the live load expected to be present at the time of a severe earthquake event

M earthquake magnitude

R distance of a site from an earthquake source

R response modification coefficient specified by the building code

u_{FM} ground motion at a building's base mat

u_g ground motion in the free field at the ground surface

ε the number of standard deviations that a spectral response acceleration value lies above (+) or below (-) the median predicted value at a period

δ_c deformation at which the peak (capping) strength of an element is attained under monotonic loading

δ_p plastic deformation available under monotonic loading from effective yield (δ_y) to attainment of peak (capping) strength (δ_c)

δ_{pc} post-peak (capping) strength component deformation available under monotonic loading, prior to failure

δ_u ultimate deformation at which a component loses all strength

δ_y component yield deformation

κ ratio of post-peak (capping) residual yield strength to initial yield strength under monotonic loading

ϕ resistance factor, as obtained from appropriate material standard

Ω_o an overstrength factor specified by the *ASCE-7* standard

σ the standard deviation of a population of values

μ the mean value of a population of values

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1 Introduction

1.1 Purpose

Structural and geotechnical engineers and researchers associated with the Pacific Earthquake Engineering Research Center developed these *Seismic Design Guidelines for Tall Buildings* as a recommended alternative to the prescriptive procedures for seismic design of buildings contained in the *ASCE-7* and other standards incorporated by reference into the *International Building Code (IBC)*. These guidelines may be used as:

- a basis for the seismic design of individual tall buildings under the building code's alternative (i.e., non-prescriptive) design provisions; or,
- a basis for development and adoption of future building code provisions governing the design of tall buildings.

The recommendations presented herein are intended, if appropriately applied and executed, to result in buildings that are more reliably capable of achieving the performance objectives for Occupancy Category II buildings intended by the *ASCE-7* standard then buildings that are designed prescriptively. Individual users may adapt and modify these recommendations to serve as the basis for designs intended to provide superior performance to that targeted for Risk Category II buildings as defined in *ASCE-7.10*.

These Guidelines are intended to serve as a reference source for design engineers, building officials, peer reviewers, and developers of building codes and standards.

> ***Commentary**: This document intentionally contains both requirements, which are stated in mandatory language (i.e., using "shall") and advisor recommendations, which use non-mandatory language (i.e., using "should").*
>
> *An alternative or non-prescriptive seismic design is one that takes exception to one or more of the requirements of the IBC by invoking Section 104.11 of the code, which reads as follows:*
>
> > ***104.11 Alternate materials, design and methods of construction and equipment.** The provisions of this code are not intended to prevent the installation of any material or to prohibit any design or method of construction not specifically prescribed in this code, provided that any such alternative has been approved. An alternative material, design or method of construction shall be approved where the building official finds that the proposed design is satisfactory and complies with the intent of the provisions of this code, and that the material, method or work offered is, for the purposed intended, at least the equivalent of that prescribed in this code in quality strength, effectiveness, fire resistance, durability and safety*
>
> *Alternative or non-prescriptive seismic designs are also recognized in ASCE 7-05, in Section 12.1.1, paragraph 3 and in ASCE 7-10, Section1.3 which states:*

***1.3.1 Strength and stiffness**. Buildings and other structures, and all parts thereof, shall be designed and constructed with adequate strength and stiffness to provide structural stability, protect nonstructural components and systems from unacceptable damage and meet the serviceability requirements of Section 1.3.2.*

Acceptable strength shall demonstrated using one or more of the following procedures:

a. the Strength Procedures of Section 1.3.1.1
b. the Allowable Stress Procedures of Section 1.3.1.2; or,
c. subject to the approval of the authority having jurisdiction for individual projects, the Performance-based Procedures of Section 1.3.1.3.

***1.3.1.3 Performance-based Procedures**. Structural and nonstructural components and their connections shall be demonstrated by analysis or by a combination of analysis and testing to provide a reliability not less than that expected for similar components designed in accordance with the Strength Procedures of Section 1.3.1.1 when subject to the influence of dead, live, environmental and other loads. Consideration shall be given to uncertainties in loading and resistance.*

***1.3.1.3.1 Analysis**. Analysis shall employ rational methods based on accepted principles of engineering mechanics and shall consider all significant sources of deformation and resistance. Assumptions of stiffness, strength, damping and other properties of components and connections incorporated in the analysis shall be based on approved test data or referenced Standards.*

The procedures recommended herein are intended to meet the criteria of ASCE-7.10 Section 1.3.1.3 as stated above.

1.2 Scope

The design recommendations contained herein are applicable to the seismic design of structures that generally have the unique seismic response characteristics of tall buildings including:

- A fundamental translational period of vibration significantly in excess of 1 second
- Significant mass participation and lateral response in higher modes of vibration
- A seismic force-resisting system with a slender aspect ratio such that significant portions of the lateral drift result from axial deformation of the walls and/or columns as compared to shearing deformation of the frames or walls.

The Pacific Earthquake Engineering Research Center developed these guidelines as an alternative means of compliance with the strength requirements for structural resistance to seismic loads specified in *ASCE-7.10* for Risk Category II structures considering the seismic hazard typical in the Western United States. Such structures are intended to resist strong earthquake motion through inelastic response of their structural components. These recommendations may be applicable to the seismic design of structures that do not exhibit substantial inelastic response or that are located in regions with seismicity somewhat different than the Western United States, however, some modification may be appropriate.

Structural design for resistance to loadings other than that associated with earthquakes is beyond the scope of this document. Design of nonstructural components other than exterior cladding for seismic resistance is also not included within the scope of this document. Design for these loadings and systems should conform to the applicable requirements of the building code or other suitable alternatives that consider the unique response characteristics of tall building structures.

1.3 Design Considerations

In recent years, structural engineers have designed a number of tall buildings in the Western United States using seismic-force-resisting systems that do not strictly comply with the prescriptive requirements of the building codes in effect at the time of their design. In some cases, these structures generally complied with the applicable building code criteria, except that the height limit specified by the building code for the selected seismic-force-resisting system was exceeded, while in other cases, seismic force-resistance was provided by structural systems that were not covered by the building code.

The seismic design of these buildings was developed using performance-based capacity design procedures in which the engineer proportioned the building for intended nonlinear response and then used nonlinear structural analysis to verify that the structure's performance would be acceptable when subjected to various levels of ground shaking. Building permits for these buildings have generally been issued under Section 104.11 of the IBC. Section 104.11 permits the use of alternative means and methods of design and construction, provided that the building official finds that such design and construction results in a building with equivalent performance capability to that anticipated for buildings that strictly comply with the code criteria. This same approach is adopted by these guidelines

Seismic design of tall buildings in accordance with these guidelines can offer a number of advantages including:

- More reliable attainment of intended seismic performance
- Reduced construction cost
- Accommodation of architectural features that may not otherwise be attainable
- Use of innovative structural systems and materials

Notwithstanding these potential advantages, engineers contemplating building design using these procedures should give due consideration to the following:

- Appropriate implementation of these recommendations requires extensive knowledge of ground shaking hazards, structural materials behavior and nonlinear dynamic structural response and analysis. Engineers that do not have this knowledge should not use these procedures.

- Seismic response of structures designed in accordance with these criteria, as well as those designed in conformance to the building code, may place extensive nonlinear cyclic strains on structural elements. In order to reliably withstand such strains, structures must be constructed to exacting quality control standards. These design procedures should not be used for structures that will be constructed without rigorous quality standards.

- Acceptance of designs conducted in accordance with these procedures is at the discretion of the building official, as outlined under Section 104.11 of the building code. Each building official can and some building officials have declined to accept such procedures. Prior to initiating a design these recommendations, development teams should ascertain that this approach will be acceptable to the authority having jurisdiction.

- The design and permitting process for a building designed in accordance with these guidelines will generally entail greater effort and take more time than designs that strictly conform to the building code prescriptive criteria.

- Even in communities where the authority having jurisdiction is willing to accept alternative designs, the development team bears a risk that the authority having jurisdiction will ultimately decide that the design is not acceptable without incorporation of structural features that may make the project undesirable from cost or other perspectives.

- In the event that a building designed in accordance with these guidelines is actually affected by strong earthquake shaking, it is possible the building will sustain damage. Some stakeholders may deem that this damage exceeds reasonable levels and may attempt to hold the participants in the design and construction process responsible for this perceived poor performance. In this event the engineer of record may be required to demonstrate that he or she has conformed to an appropriate standard of care. It may be more difficult to do this for buildings designed by alternative means than for buildings designed in strict conformance to the building code.

Section 1.3 of *ASCE-7.10* requires the use of independent third-party design (peer) review as an inherent part of the design process using alternative mean. These guidelines also recommend such review as it can help to provide the building official with assurance that a design is acceptable, can suggest approaches that will assist the design team to improve a design's reliability, and can help establish conformance with an appropriate standard of care. It is essential that reviewers possess sufficient knowledge, skill and experience to serve in this role.

1.4 Design Team Qualifications

Appropriate implementation of the design guidelines presented herein requires sophisticated structural and earthquake engineering expertise including knowledge of:

- seismic hazard analysis and selection and scaling of ground motions

- nonlinear dynamic behavior of structures and foundation systems and construction of mathematical models capable of reliable prediction of such behavior using appropriate software tools
- capacity design principles
- detailing of elements to resist cyclic inelastic demands, and assessment of element strength, deformation and deterioration characteristics under cyclic inelastic loading

Engineers who do not have this expertise and knowledge should not undertake projects utilizing these guidelines, either as the engineer of record or as a third party reviewer.

1.5 Basis

Earthquake, structural, and geotechnical engineering researchers and professionals engaged by the Pacific Earthquake Engineering Research (PEER) Center developed the recommendations presented herein under the Center's Tall Buildings Initiative. PEER is a multi-disciplinary research and education center with headquarters at the University of California, Berkeley. Since 1997, PEER has performed engineering and social science research to support the development of performance-based earthquake engineering under funding from the National Science Foundation, the State of California, individual cities, and private industry. At PEER, investigators from over twenty universities and several consulting companies conduct research into earthquake-related geohazard assessment, geotechnical and structural engineering, risk management, and public policy.

The Tall Buildings Initiative included research into the appropriate performance characteristics of tall buildings in the urban habitat, the characteristics of ground motion that affect tall building performance, appropriate methods of modeling and analyzing tall buildings, as well as research into earthquake-resistant means of construction The Tall Buildings Initiative also draws from the experience gained by engineers in the actual application of performance-based earthquake engineering principles to the seismic design of tall buildings.

These guidelines were developed with funding provided by a grant from the Charles Pankow Foundation.

1.6 Limitations

These recommendations are intended to provide a reliable basis for the seismic design of tall buildings based on the present state of knowledge, laboratory and analytical research and the engineering judgment of persons with substantial knowledge in the design and seismic behavior of tall buildings. When properly implemented, these guidelines should permit design of tall buildings that are capable of seismic performance equivalent or superior to that attainable by design in accordance with present prescriptive building code provisions. Earthquake engineering is a rapidly developing field and it is likely that knowledge gained in the future will suggest that some recommendations presented herein should be modified. Individual engineers and building officials implementing these recommendations must exercise their own independent judgment as to the suitability of these recommendations for that purpose. The Pacific Earthquake Engineering Research Center, the University of California, the Charles Pankow Foundation, the individual contributors to this document and their employers offer no warranty, either expressed or implied, as to the suitability of these guidelines for application to individual building projects.

2 Design Performance Objectives

2.1 Minimum Performance Objectives

Buildings designed in accordance with these guidelines are intended to have seismic performance capability equivalent to that intended for similar buildings designed in full conformance with the requirements of the 2009 *International Building Code, ASCE-7.05 and ASCE-7.10*. As presented in commentary to the FEMA P750 *2009 NEHRP Recommended Provisions for Seismic Regulation for Buildings and Other Structures*, the building code is intended to provide buildings conforming to Occupancy Category II of *ASCE-7.05* (Risk Category II of *ASCE-7.10*) the capability to:

- withstand Maximum Considered Earthquake shaking, as defined in *ASCE-7*, with low probability (on the order of 10%) of either total or partial collapse;
- withstand Design Earthquake shaking, having an intensity 2/3 that of Maximum Considered Earthquake shaking without generation of significant hazards to individual lives through design measures intended to assure that nonstructural components and systems remain anchored and secured to the structure and that building drifts are maintained at levels that will not create undue hazards; and,
- withstand relatively frequent, more moderate-intensity earthquake shaking with limited damage.

The design recommendations presented herein seek to satisfy these objectives through requirements to:

- proportion and configure structures using capacity design principles;
- demonstrate that the structure will be capable of essentially elastic response and limited damage under Service-level Earthquake shaking having a mean return period of 43 years (50% exceedance probability in 30 years);
- demonstrate, with high confidence, that the structure will respond to Maximum Considered Earthquake shaking: without loss of gravity-load-carrying capacity; without inelastic straining of important lateral-force resisting elements to a level that will severely degrade their strength; and without experiencing excessive permanent lateral drift or development of global structural instability;
- detail all elements of the structure for compatibility with the anticipated deformations of the seismic-force-resisting system under Maximum Considered Earthquake shaking; and,
- Anchor and brace all nonstructural components and systems in accordance with the requirements of the building code, or alternatively, such that elements essential to protect life safety are anticipated to function and other elements are anticipated to remain in place and not create falling hazards under Design Earthquake shaking.

Commentary: These guidelines anticipate that damage in response to service-level shaking may include minor cracking of concrete and yielding of steel in a limited number of structural elements. Damage occurring in response to Service-Level shaking should not compromise the structure's ability to survive Maximum Considered Earthquake shaking, nor should it result in unsafe conditions requiring repair prior to occupancy. Some repair may be needed to restore appearance or protection from water intrusion, fire resistance, or corrosion. Nonstructural damage should be below the threshold that would limit the post-event occupancy of the building.

2.2 Enhanced Objectives

It may be desirable to design some structures to achieve performance superior to that described in the previous section. Nothing contained within these guidelines should be interpreted as preventing such design; however it may be necessary to adopt modifications to these recommended design criteria to attain enhanced performance. Such modifications could include:

- selection of an alternative, lower probability of exceedance, either for Service Level Shaking or Maximum Considered Earthquake shaking, or both;
- selection of more restrictive acceptance criteria, potentially including lower limiting levels of lateral drift and/or reduced levels of acceptable cyclic straining of ductile elements and larger margins for capacity-protected elements;
- design of nonstructural components and systems to withstand shaking more intense or inter story drifts larger than that required by the building code;
- design to limit residual displacements as a means of ensuring the structure can be repaired following earthquake ground shaking;
- incorporating the use of damage-tolerant structural elements that are capable of withstanding cyclic inelastic deformation without degradation or permanent distortion; and,
- incorporating the use of response modification devices including isolation systems, energy dissipation systems, and passive and active control systems to limit structural response.

When a design is intended to provide enhanced performance capability, the engineer should prepare a formal written project design criteria that explicitly states both the desired performance and the means to be employed to achieve this performance .

3 Design Process Overview

3.1 Introduction

This Chapter presents an overview of the recommended design process and references the location of detailed recommendations.

3.2 Determine Design Approach

Prior to using these recommendations for design, the structural engineer should ascertain that the building official is amenable to performance-based design alternatives and the use of these procedures. In addition, the structural engineer should assure that the development team is aware of and accepts the risks associated with the use of alternative design procedures, that the engineer has the appropriate knowledge and resources, and that construction quality will be adequate to assure the design is properly executed. Section 1.3 provides additional discussion of these issues.

3.3 Establish Performance Objectives

Section 2.1 describes the target performance capability for buildings designed in accordance with these procedures. The structural engineer should discuss these performance criteria with the development team and the authority having jurisdiction and confirm that these will form an acceptable basis for design. If enhanced performance objectives are desired, the engineer should develop a formal design criteria document that modifies the recommendations contained herein as necessary to permit attainment of the enhanced objectives. Section 2.2 provides discussion of some ways this can be accomplished.

3.4 Seismic Input

These procedures require determination of two levels of ground motion: a Service-Level shaking motion and a Maximum Considered Earthquake shaking motion. Service-Level motion is represented by a 2.5%-damped, acceleration response spectrum having a 43-year mean return period. Maximum Considered Earthquake shaking is represented by a 5%-damped acceleration response spectrum conforming to the requirements of *ASCE-7* and a suite of earthquake ground acceleration records that have been appropriately selected and scaled to be compatible with this spectrum. Chapter 5 provides guidance on the representation of ground motion and selection and scaling of records.

> ***Commentary**: As described in Chapter 8, structural analysis for Maximum Considered Earthquake shaking is performed using not more than 2.5% equivalent viscous damping in the structural model. Use of 5%-damped elastic acceleration response spectra to represent Maximum Considered Earthquake shaking is only for convenience to allow comparison with spectra that are generated in compliance with* the procedures of *ASCE-7.*

3.5 Conceptual Design

In this step the engineer must select the structural systems and materials, their approximate proportions, and the intended primary mechanisms of inelastic behavior. The engineer should use capacity design principles to establish the target inelastic mechanisms. Chapter 6 presents useful information for development of conceptual designs.

3.6 Design Criteria

The structural engineer of record should develop a formal design criteria document that describes: the structural systems and materials of construction; the anticipated mechanisms of inelastic response and behavior; the design performance objectives; the specific design and analysis measures to be conducted to demonstrate acceptable performance capability; and, all exceptions to the prescriptive provisions of the building code. This design criteria document should be submitted to and approved by the authority having jurisdiction and third party reviewers prior to undertaking substantial design effort. Chapter 4 presents a suggested outline for project-specific design criteria.

3.7 Preliminary Design

Dynamic structural analysis is used to confirm that building designs are capable of meeting the intended performance objectives. To perform a meaningful analysis the engineer must develop the building design to a sufficient level of detail to allow determination of the distribution of its stiffness, strength and mass as well as the hysteretic properties of elements that will undergo inelastic straining in response to strong ground shaking. Chapter 6 presents information intended to help engineers developing preliminary designs.

3.8 Service Level Evaluation

The Service-Level evaluation is intended to demonstrate that the building will be capable of withstanding relatively frequent, moderate-intensity shaking with limited structural damage. Section 2.1 describes the performance expectation in this regard. Chapter 7 presents detailed guidelines for performing the Service-Level evaluation and confirming acceptable performance capability for Service-level shaking.

3.9 Maximum Considered Response Evaluation

Chapter 8 presents guidelines for nonlinear dynamic analysis and acceptance criteria used to demonstrate that buildings have acceptable response characteristics when subjected to Maximum Considered Earthquake shaking.

3.10 Final Design

The final design is documented by the construction documents, including detailed drawings and specifications, supported by extensive calculations and analyses that result in the generation of large amounts of data. Chapter 9 presents guidelines for organizing and summarizing this data in a manner that facilitates review by building departments and third party reviewers.

3.11 Peer Review

Independent, third-party review should include the design criteria, seismic hazards analysis, selection and scaling of ground motions, proportioning, layout and detailing of the structure, modeling, analysis, interpretation of results, and construction quality assurance. Chapter 10 presents recommended review procedures.

4 Design Criteria Documentation

4.1 General

The structural engineer of record should prepare a formal Design Criteria document that describes the intended structural systems, performance objectives, any intended deviations from prescriptive building code criteria, and the specific loading, analysis, design procedures, and acceptance criteria to be employed in the design. The engineer of record should prepare an initial draft of the project Design Criteria as early in the design process as is practical and should update and revise this document as the design is advanced and the details of the building's characteristics and performance are better understood. The Design Criteria should contain a summary of the overall design objectives and should be updated at key project milestones. At the conclusion of the design effort, the Design Criteria should provide an accurate summary of the final design and the procedures used to demonstrate its performance capability.

Commentary*: Clear and concise communication of structural design intent through a well-prepared "Design Criteria" document is beneficial for all parties involved in building design, review, and implementation. Within the structural engineer's office, staff members will benefit from consistent and clear direction promoting a well-executed design. Building officials faced with review of the design will gain a clear understanding of how the design is intended to meet or exceed the performance expectations inherent in the building code. Peer reviewers, responsible for completing in-depth review of the design, will benefit from a thorough summary of the design objectives, methods of analysis, and acceptance criteria.*

The structural engineer should submit the design criteria to the peer reviewers and building official for acceptance well in advance of the submittal of documents for building permits.

Commentary: *It is important to obtain agreement regarding the proposed design approach as early in the process as is practical in order to avoid expending needless effort using an approach that will not receive approval. Once agreement on the design approach is reached, it should be possible to obtain approval simply by demonstrating that the design conforms to the agreed upon criteria. It should be noted, however, that as the details of a design are developed, it may become necessary to revise the previously anticipated design approach, analytic procedures and/or proposed acceptance criteria. Multiple submissions of the design criteria, as it evolves, may be necessary and should be anticipated by all project participants.*

4.2 Criteria Content

The sections below indicate the suggested content for typical project design criteria and the types of information that should generally be included.

4.2.1 Building Description and Location

***Commentary**: The purpose of this section is to provide a basic understanding of the project scope and a framework that will place the specific design criteria presented in other sections into perspective.*

a. General

Provide a brief description of the overall building, including any special or unique features and occupancies. This description should include a characterization of the site, its geographic coordinates and the underlying site conditions.

b. Description of Seismic and Wind Force-resisting Systems

Provide a brief description of the seismic and wind force-resisting systems. This discussion should include a description of the primary load paths, the anticipated areas of inelastic behavior and any response modification (isolation bearings, passive or active damping) devices that will be incorporated into the design.

c. Representative Drawings

Provide sufficient floor plans, building sections, and elevations to provide an overview of the building. Drawings should clearly identify the primary lateral force-resisting system's configuration.

***Commentary**: Include sufficient drawings or sketches to illustrate the structural systems.*

4.2.2 Codes and References

a. Controlling Codes, Standards and Other References

Provide a listing of the controlling building codes, including local amendments; and any standards, guidelines, or reference documents, upon which the design will be based.

b. Exceptions to Building Code Provisions

Provide a listing of any exceptions or deviations that will be taken from the prescriptive code provisions, together with a brief description of the justification for such exceptions.

***Commentary**: Most buildings designed in accordance with these guidelines will generally conform to the design and construction requirements of the applicable building code, with the exception that a limited number of exceptions or alternative criteria will be employed. Since all of the prescriptive requirements of the building code are presumed to be important to the building's performance, the structural engineer should indicate why non-compliance with any of these criteria will be acceptable for this particular design. Reasons provided could include identification that the requirement is not applicable to the particular building in one or more ways,* *or that acceptable performance will be assured by other means, such as analysis or testing.*

4.2.3 Performance Objectives

Provide a listing of the expected building performance objectives including the structural and nonstructural components. These objectives should address performance under both Service Level and Maximum Considered Earthquake hazards. A listing of some of the possible components includes:

- Overall Building Performance
- Performance of Structural Elements
  - Walls
  - Columns
  - Beams
  - Braces
  - Floor Slabs
  - Diaphragms
  - Foundations
  - Damping Devices
- Performance of Nonstructural Elements
  - Cladding
  - Partition Systems
  - Elevators
  - Exit Stairs

Commentary*: Tabular summary of the performance objectives for each of the important building components at both Service-level and Maximum Considered Earthquake shaking level is recommended. Include discussion of intended seismic force resisting elements and gravity elements.*

4.2.4 Gravity Loading Criteria

Provide a description of gravity loading criteria including allowances for key structural and nonstructural components, and live loading to be applied in different portions of the building. Specify any live load reductions to be employed as well as any special loads including vehicular or special equipment.

4.2.5 Seismic Hazards

Provide a brief summary of the seismic demands to be considered during design including both Service-level and Maximum Considered Earthquake shaking events as well as any other events that may be selected. The site characterization and definition of specific seismic demands will likely be more thoroughly addressed in a separate report prepared by a seismic ground motion specialist. The purpose of this section is to briefly summarize important details regarding the seismic hazard which will influence the structural design. This section should, as a minimum, include:

- Identification of Site Class per the building code

- Identification of likelihood of seismic hazards other than ground shaking including liquefaction, land sliding, lateral spreading, or inundation.
- Indication of return period (or annual frequency of exceedance), and the deterministic or characteristic events that define both the Service-level and Maximum Considered Earthquake shaking events.
- Elastic acceleration response spectra for the Service-level and Maximum Considered Earthquake shaking events
- Define the acceleration histories that will be used for nonlinear dynamic analysis including, a discussion of adjustment (scaling/matching) procedures employed and the specific earthquake events, their magnitudes, the recordings used, and their distances to the instrument. If amplitude scaling is performed, identify the scale factors used. Provide plots that illustrate the extent to which the individual adjusted records and their average compare with the target design spectra. If spectral matching is used, identify the procedures used to perform such matching.

Include the detailed Site Specific Seismic Hazard report as an appendix.

> ***Commentary****: It is important that the response spectra and corresponding ground motions to be used in analysis are reviewed and approved by the peer review prior to completing the analytical work.*

4.2.6 Wind Demands

Provide a brief summary of the wind demands that will be considered during design including:

- Design wind speed and return period (or annual frequency of exceedance) to be used for strength considerations
- Design wind speed and return period (or annual frequency of exceedance to be used for service level considerations
- Site exposure characteristics
- Method used to determine wind loadings (analytical or test)

If a wind tunnel test is performed, include the detailed wind tunnel report as an appendix.

> ***Commentary***: *Even in regions of very high seismic risk, it is quite possible for wind demands to exceed seismic demands for service level events and on some elements even for Maximum Considered Earthquake shaking. In addition, wind-induced overturning moments may exceed seismic overturning moments when defining the lower bound strength of the structural system. Wind effects should be evaluated early in the design process.*

4.2.7 Load Combinations

Provide a summary of all design load combinations that will be used and the specific elements to which they will be applied. Refer to Chapters 7 and 8 for further guidance on load combinations.

***Commentary**: It is likely that a series of different load combinations will be used for different elements. For example, adequacy of foundations will typically be evaluated using Allowable Stress load combinations. Load and Resistance Factor combinations will typically be used for dead, live, wind, and seismic demands on structural steel and reinforced concrete elements. Different load combinations may be used for elements that are intended to exhibit inelastic behavior as opposed to those elements that are intended to remain elastic. Service-level load combinations may be different than those used for Maximum Considered Earthquake response. Also, the treatment of floor live loading may be different in the various load cases. It is important to identify the specific application for each load combination presented.*

4.2.8 Materials

Provide a listing of the material properties to be specified on the design drawings, as well as any assumptions regarding material over-strengths or lower-bound strengths to be used in the design evaluations.

***Commentary**: Expected material properties will be used in developing mathematical models of the structure, attempting to characterize the expected performance as closely as possible. These same material properties will also likely be used in implementing capacity design concepts and/or evaluating demand/capacity ratios of elements with benign modes of failure. Lower-bound strengths are likely to be used in demand/capacity assessments of elements with brittle failure modes or modes that can result in catastrophic consequences.*

4.2.9 Analysis

a. Procedures

Provide a summary of each method of analysis (linear static, linear dynamic, nonlinear static, nonlinear response history) that will be used and the anticipated application and purpose of each of these.

b. Analysis and Design Software

Provide a listing of the various analysis and design tools (software) being used, including the specific version of this software.

c. Modeling Procedures and Assumptions

Provide a summary of the modeling procedures and key assumptions to be incorporated in each evaluation including:

Material properties
Section property definition
Joint stiffness assumptions
Damping assumptions
Component models and hysteretic behavior
Boundary conditions

> ***Commentary**: Many designs will incorporate different models and analysis procedures for the Service-level and Maximum Considered Earthquake shaking evaluations. Some designs may also incorporate an evaluation of elements for Design Earthquake shaking, as identified in the building code. The Design Criteria should separately and completely describe the modeling approach and assumptions used for each analysis employed.*

4.2.10 Acceptance Criteria

Provide a summary of all acceptance criteria to be used in demonstrating that the design meets or exceeds the stated performance objectives for both Service-level and Maximum Considered Earthquake shaking. Include details regarding:

- Strength calculations
- Demand/capacity ratios
- Drift limits
- Deformation limits
- Strain limits

For demands obtained from nonlinear dynamic analyses, indicate the statistical quantities from the suite of analysis results that will be used to perform evaluations against the acceptance criteria. Refer to Chapter 8 for further guidance on this subject.

Where strain limits will be used as acceptance criteria, describe specifically how predicted strains will be derived from the analysis.

In addition, show representative details necessary to justify the stipulated acceptance criteria should be summarized. Examples include:

- Concrete confinement details
- Slab-column connection details
- Slab-wall connection details
- Moment frame connection details
- Brace connection details
- Collector and drag-strut details
- Damping device details

> ***Commentary**: Acceptance criteria are the acceptable values of the various response quantities obtained from the analysis. They can include individual limits on element strength demands, element inelastic deformation demands, and global parameters such as drift. When nonlinear dynamic analysis is performed using suites of ground motions, a suite of demands will be obtained for each of these response quantities. It is not unusual for the coefficient of variation for the values of individual response quantities to range as high as 50%. While it may be appropriate to use mean, or average demands for response quantities associated with the prediction of failure modes that have relatively benign consequences, it is usually appropriate to use more conservative estimates of demand for behavioral modes that can result in catastrophic consequences. Chapter 5 provides additional discussion of the variability inherent in response quantities obtained from suites of ground motions while Chapters 7 and 8 recommend acceptance criteria for different types of elements associated with their several behavioral modes.*

4.2.11 Test Data to Support New Components, Models, Software

If the design includes innovative components, materials, modeling techniques, or software, include supporting materials justifying their appropriateness. Where laboratory research is used as a benchmark for such justification, provide explicit references to publications documenting the research, if it is in the public domain, or include copies of test reports in an appendix to the report where the information is not publicly available.

4.2.12 Appendices

Include the following materials in appendices, as appropriate.

A. Geotechnical Report
B. Site Specific Seismic Hazard Report
C. Wind Tunnel Report
D. Research Papers as indicated in Section 4.2.11

5 SEISMIC INPUT

5.1 General

Seismic design of tall buildings using these guidelines requires characterization of two levels of ground shaking: Service-level shaking and Maximum Considered Earthquake shaking. This chapter provides guidelines for an overall approach that involves: 1) conducting probabilistic or deterministic seismic hazard analysis to define acceleration response spectra for each of these shaking levels; 2) modifying the spectra as needed for local site effects, and; 3) selecting and modifying appropriate accelerograms for response history analysis at the Maximum Considered Earthquake level and Service-level as needed. This chapter also provides guidelines for appropriate consideration of Soil-Foundation-Structure Interaction affects.

5.2 Seismic Hazard Analysis

Use seismic hazard analysis to determine the appropriate ordinate amplitude of Service-level and Maximum Considered Earthquake shaking level acceleration response spectra. Two types of seismic hazard analysis may be used. Probabilistic Seismic Hazard Analysis should generally be used. At sites that are located within 10 kilometres of one or more known active faults, capable of producing earthquakes with magnitudes in excess of M6, Deterministic Seismic Hazard Analysis should also be used for the Maximum Considered Earthquake shaking level. Refer to the requirements of ASCE-7, Chapter 21 to determine whether the results of probabilistic or deterministic seismic hazard analysis should be used to define the Maximum Considered Earthquake acceleration response spectrum.

5.2.1 Probabilistic Seismic Hazard Analysis

Perform probabilistic seismic hazard analysis for Service-level shaking (43-year return period, 50% probability of exceedance in 50 years) and Maximum Considered Earthquake shaking, as defined in *ASCE-7* using appropriate contemporary models for the description of regional seismic sources and ground motion prediction equations. Ensure that the recent developments in those topics and the use of the models are properly implemented in the probabilistic seismic hazard analysis code being used. The mechanics of probabilistic seismic hazard analysis are described elsewhere (e.g., Stewart et al., 2001; McGuire, 2004) and this section assumes a basic familiarity with the analysis procedures. When conducting probabilistic seismic hazard analysis, account for epistemic (modeling) uncertainties in the input source and ground motion prediction models and in the associated parameter values by including weighted alternatives in the analysis.

Report the following outcomes of probabilistic seismic hazard analysis: 1) mean ground-motion hazard curves at key structural periods including 0.2 seconds, 1.0 second, 2 seconds and the fundamental period of the structure; 2) uniform hazard spectra associated with the Service-level and Maximum Considered Earthquake shaking levels; and; 3) percentage contributions to the ground-motion hazard at the key structural periods for each hazard level. These contributions are a function of the seismic source, earthquake magnitude and source-to-site distance, are evaluated from deaggregation of the seismic hazard.

Compute uniform hazard spectra over a range periods extending sufficiently beyond the building's fundamental period to capture the effective (lengthened) building period during response to Maximum Considered Earthquake shaking.

> *Commentary: Uniform hazard spectra have contributions from many seismic sources and, consequently, no one earthquake is likely to produce a response spectrum consistent with the Uniform Hazard Spectra at all oscillator periods. Thus deaggregation information is an important consideration in the selection of acceleration histories, as described further in Section 5.4.*

Probabilistic seismic hazard analysis results for any location in the U.S. can be obtained using the USGS seismic hazard tool (http://earthquake.usgs.gov/research/hazmaps/index.php). The USGS site is well maintained and is kept current with respect to source models and ground motion predictive equations. When the building code or other seismic regulations call for a "site-specific analysis" of ground motion, a site specific probabilistic seismic hazard analysis is required in lieu of the USGS web site. Site-specific probabilistic seismic hazard analysis can be performed using one of several available commercial codes (e.g., FRISKSP, EZ-FRISK, FRISK88M) and the open source code OpenSHA (Field et al., 2003).

> ***Commentary:*** *The latest revisions to the USGS source models can be found in USGS Open File Report 2008-1128 (Petersen et al., 2008).*

> *Ground motion prediction equations or attenuation relations provide the median and standard deviation of a ground motion Intensity Measure (IM) conditional on parameters related to source (e.g., magnitude, focal mechanism), path (e.g., closest distance, position relative to hanging wall), and site (e.g., average shear wave velocity in upper 30 m of site, basin depth). For shallow crustal earthquakes in tectonically active regions, the best currently available ground motion predictive equations are those developed in the Next Generation Attenuation (NGA) project (Power et al., 2008). Those models should suffice for estimating ground motions from shallow crustal earthquakes in the western U.S. Different ground motion predictive equations are needed for ground motions generated by earthquakes occurring on the interplate (interface between Pacific Ocean and North American tectonic plates) and intraplate (Benioff zone) segments of the subduction zones in the Pacific Northwest or Southern Alaska. Table 5.1 summarizes the recommended empirical ground motion predictive equations for both shallow crustal and subduction sources and their major attributes.*

> *Most ground motion prediction equations include a site term that accounts for average site effects. As described further in Section 5.2.3, in many cases this site term is sufficient for practical purposes and no separate modeling of the site response is needed. In other cases, a site-specific analysis of site response is advisable (or required by the Code). Guidelines on analysis of that type are presented in Section 5.2.3.*

> *The lack of knowledge regarding which model to use within a particular component of probabilistic seismic hazard analysis is referred to as epistemic*

uncertainty. Epistemic uncertainty is ideally incorporated using a logic tree framework with multiple viable values and associated weights of the critical source parameters and multiple ground motion prediction equations. Further details on probabilistic seismic hazard analysis in a logic tree framework are provided in McGuire (2004).

The main drawbacks to the USGS site are (1) ground-motion hazard is computed for a fixed set of source and ground motion predictive equation inputs, thus eliminating the possibility of revising inputs and recomputing the hazard; (2) hazard is computed for a reference site condition of V_{s30}=760 m/s; hence site effects are not included in the probabilistic seismic hazard analysis and must be added subsequently in a deterministic manner, which can introduce bias (Goulet and Stewart, 2009); (3) the user cannot perform logic-tree analyses to estimate epistemic uncertainties on hazard curves or UHS.

The main drawback to site-specific analysis is that it requires knowledge of probabilistic seismic hazard analysis and the underlying models. Inadequate familiarity typically leads to misuse of the codes and erroneous results. Therefore, users unfamiliar with probabilistic seismic hazard analysis tools and related models should consider using the USGS web site in lieu of site-specific analysis.

5.2.2 Deterministic Seismic Hazard Analysis

Deterministic seismic hazard analysis has the same components as probabilistic seismic hazard analysis (source model, ground motion predictive equations). The difference is that the range of possible results at each stage of the analysis is not considered in deterministic seismic hazard analysis. A single earthquake is considered with a prescribed magnitude and location. A single percentile-level of ground motion is taken from the ground motion predictive equation (e.g., 50 %-tile or median motion). The selections made at the various stages of deterministic seismic hazard analysis are arbitrary and it is often difficult to know *a priori* whether the choices made lead to conservative or unconservative estimates of ground motion. Nevertheless, the *ASCE 7* standard, requires the use of deterministic seismic hazard analysis to provide a deterministic cap on ground motion in regions near major active faults (Leyendecker et al., 2000) to limit ground motion to levels deemed “reasonable” for seismic design.

When deterministic seismic hazard analysis is required per ASCE 7, use the same ground motion predictive equations and weights used in the probabilistic seismic hazard analysis for the controlling fault. Assign the same values to the independent parameters, such as V_{s30} and fault type, as assigned in the probabilistic seismic hazard analysis. Select the maximum magnitude for the controlling fault that is the weighted average of alternative maximum magnitudes listed in the logic tree used in the probabilistic seismic hazard analysis.

Commentary: *More than one fault may produce the largest ground-motion response spectrum. For example, a large magnitude event (e.g., M6.5 – 7.0) on a nearby fault may produce the largest ordinates of a response spectrum at short and intermediate natural periods, but a great earthquake (e.g., M~8 or larger) on a fault farther away may produce the largest long period ordinates.*

Table 5-1 Selected Ground Motion Prediction Equations for Horizontal Response Spectra at 5% Damping Ratio

Reference	Regression Method[1]	Applicable M Range[2]	R range (km)	R type[3]	Site Parameters[4]	Site Terms[5]	Other Parameters[6]	Comp[7]	Period Range[8]
Active Regions									
Boore and Atkinson (2008) - NGA	2-S/RE	5-8	0-200	R^{1}_{jb}	V_{s30}	NL	F	gm-rot[i]	PGA-10[i]
Campbell and Bozorgnia (2008)-NGA	2-s/RE	4-7.5(n), 8(r), 8.5 (ss)	0-200	R	V_{s30}-$Z'_{2.5}$	NL	F, Z_{tot}	gm-rot[i]	PGA-10[i]
Abrahamson & Silva (2008) - NGA	RE	5-8.0(r),8.5(ss)	0-200	R, R^{1}_{jb}	V_{s30}-$Z'_{1.0}$	NL	F, W, Z_{tot}, δ, R'_{x}, HW	gm-rot[i]	PGA-10[i]
Chiou and Youngs (2008) - NGA	RE	4-8(n,r), 8.5 (ss)	0-200	R, R^{1}_{jb}	V_{s30}-$Z'_{1.0}$	NL	F, Z_{tot}, δ, R'_{x}	gm-rot[i]	PGA-10[i]
Idriss (2008) – NGA	1-s	4-8r), 8.5(ss)	0-200	R	V_{s30}>450m/sec		F	gm-rot[i]	PGA-10[i]
Subduction Zones									
Atkinson and Boore (2003, 2008)	1-s	5.5-8.3	10-500	r_{HYPO}, R	Rock & soil classes	NL	h'	gm	PGA-3
Crouse (1991a, b)	1-s	4-8.2	10-900	r_{HYPO}, R	Soil only	na	h	gm	PGA-4
Youngs et. al. (1997)	1-s/RE	5-8.2	10-600	r_{HYPO}, R	Rock & soil	na	Z_t, h[i]	gm	PGA 3 or 4
Zhao et. al. (2006)	RE	5-8.3	0-300	r_{HYPO}, R	Rock & soil classes	L	h	gm	PGA-5

2-s = two-step regression; 1-s = one-step regression; RE = random effects
n = normal fault events; r = reverse fault events; ss strike-slip fault events
R = site-source distance; R_{jb} = surface projection distance; r_{Hypo} = hypocenter distance
V_{s30} = average shear wave velocity in upper 30 m of site; $Z_{2.5}$ = depth to V_s = 2.5 km/s; $Z_{1.0}$ = depth to V_s = 1.0 km/s
NL = site effect is nonlinear; L = site effect is linear; na = not applicable
F = style of faulting factor; HW = hanging wall flag; h = focal depth, Z_t = subduction zone source factor; Z_{tor} = depth to top of rupture; δ = fault dip
Component of horizontal motion considered. gm = geometric mean; gm-rot=orientation-independent geometric mean
PGA–T means 0 to T sec, where T = 3, 4, 5, or 10 sec; PGA-3 or 4 means 0 to 3 sec for the rock equations, and 0 to 4 sec for soil eqns

A special case of deterministic seismic hazard analysis is to use seismological simulation techniques to generate site ground motions for a prescribed earthquake source function coupled with wave propagation analysis. Chapter 6 of Stewart et al. (2001) describes a number of simulation methods of this type. Advantages of seismological simulation tools are that they are able to produce ground motions for large magnitude events. The principal disadvantage of these simulation tools is the limited calibration against data and lack of commercial software and understanding of the underlying seismological principles, which has limited their implementation in engineering practice.

5.2.3 Site-Response Analysis

Perform site response analyses, where appropriate and where require by code. Use either equivalent linear or fully nonlinear methods. Conduct such analyses for several input ground motions and for variations in the soil constitutive models and material properties, including the shear strain-dependent shear moduli and material damping ratios, as well as oil shear strength.

Select records for site response analysis for a site condition that is generally compatible with the geologic conditions at the bottom of the site profile being simulated. If bedrock is reasonably shallow and its depth is known, the profile should extend into rock and input motions should be defined for the rock condition. If the site consists of deep soils that cannot be reasonably simulated in their entirety, then the soil profile should extend to a firm soil horizon. In that case, use input motions for weathered bedrock or firm soil conditions. See Section 5.4 for additional considerations for input motion selection.

Commentary: *When performed for a one dimensional medium, site response analysis is often referred to as "ground response analysis," which can serve in some cases as a good approximation of the full 3-dimensional site response problem. Ground response analyses are performed for two principal reasons. The first is to improve predictions of free-field ground surface motions relative to what can be obtained using the site term in a ground motion predictive equation. The second is to estimate the variations with depth of motions through the profile, which can be used to define the seismic demand at the ends of soil springs in soil-structure interaction analyses. However, nonlinear structural dynamic analyses codes presently used for buildings cannot accommodate spatial variations in the input ground motion. This limitation is not considered serious for tall buildings where the spatial variations of long period motions are expected to be minimal over distances equal to the plan and depth dimensions of the subterranean levels of the building. See Section 5.3 for additional information.*

The commentary to the 2003 NEHRP Seismic Provisions (Part 2) provides guidance on obtaining dynamic soil properties. On-site measurement of V_s should be used in lieu of correlations between V_s and other parameters such as penetration resistance. For most practical situations, the use of modulus reduction and damping curves from correlation relationships should suffice, unless large strain response is expected.

Liquefaction problems are especially challenging in a site response context. Equivalent linear methods cannot capture the full behavior of liquefied soils because they utilize total stress representations of soil properties that cannot *simulate pore pressure generation and its effects on soil properties (e.g., Youd and Carter, 2005). However, approximate equivalent linear moduli and damping values can be assigned to liquefied layers based on an analysis of ground motions at vertical array sites that liquefied (Zeghal and Elgamal, 1994; Elgamal et al., 1996).*

5.3 Soil-Foundation-Structure Interaction

Consider soil-foundation-structure interaction effects in accordance with this section when developing analytical models for seismic evaluation of tall buildings with subterranean levels.

Commentary: *Tall buildings generally have subterranean levels to provide space for parking and other facilities. The most common foundation type is mats, although pile systems are used as well, particularly for tall buildings without subterranean levels. A schematic illustration of a building with subterranean levels is shown in Figure 5.1a. Spatial variations of ground motion cause motions on foundation slabs (u_{FIM}) to differ from free-field motions (denoted u_g in Figure 5.1a), which is referred to as a kinematic interaction effect.*

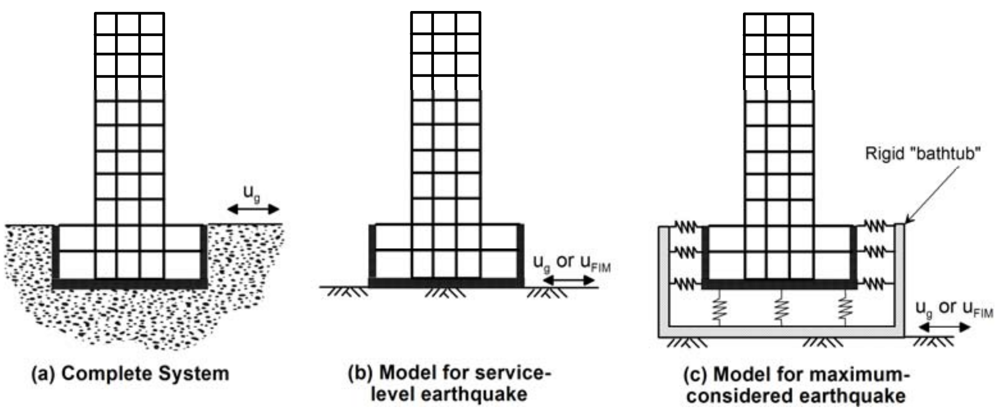


Figure 5.1 Schematic illustration of tall building with subterranean levels and simple models for analysis in which soil-foundation interaction effects are neglected (Part b) and included in an approximate manner (Part c). Part (c) only shows springs but parallel dashpots are generally also used.

5.3.1 Service-level analysis

Extend analytical models used for Service-level response analysis to the structure's base, as shown in Figure 5.1b. Include the subterranean levels in the structural model used for dynamic response analysis. Include appropriate element stiffness and capacities for structural members such as walls, columns, and slabs. Soil springs need not be included in the model. Motion should be applied at the base of the structure and can consist either of free-field motion (u_g) or the foundation input motion (u_{FIM}), which is modified for kinematic interaction effects.

5.3.2 Maximum Considered Earthquake shaking analysis

Include subterranean levels. Include if practical, springs and dashpots representing foundation-soil interaction along basement walls and below the base slab, as shown in Figure 5.1c. Input ground motions to the model via a rigid "bathtub" surrounding the subterranean portions of the structure. Input motion can consist either of free-field motion (u_g) or the foundation input motion (u_{FIM}), which is modified for kinematic interaction effects.

If the above procedure is not practical, for MCE analysis, use option (b) in Figure 5.1(b). If option (b) is used, since the soil springs are not included in the model, the mass of the subterranean levels may also be modified. One option is to include the mass of the core tower below the grade, and exclude the mass of other extended elements in the subterranean levels.

> ***Commentary****: An approach similar to that described above for buildings with mat foundations should be implemented for pile foundations. Typical engineering practice for this foundation type is to (1) define the free field ground motion at the level of the pile caps, (2) excite the building with this motion or feed the motion through linear springs/dashpots attached to the pile cap to model the soil-pile interaction, (3) compute the base forces and moments, and (4) input them in a separate soil-pile interaction model to obtain the pile responses.*
>
> *Procedures for analysis of kinematic interaction effects are given in FEMA-440, ASCE-41 and Stewart and Tileylioglu (2007). Those effects are generally most pronounced at short periods (less than approximately 1 sec), and hence are unlikely to significantly affect fundamental mode responses of tall buildings.*
>
> *The above approach for pile foundations is reasonable for relatively stiff and stable soils, but it may not be acceptable for soils susceptible to failure, where the soil-pile interaction becomes highly nonlinear. In those situations, an iterative solution technique can be implemented in which trial values of equivalent linear springs/dashpots are selected until the base-level displacements computed from the dynamic analysis of the building are compatible with the pile-cap displacements computed from the application of the building base forces and moments to the soil-pile model.*

5.4 Selection and Modification of Accelerograms

5.4.1 Introduction

Select and modify accelerograms for structural dynamic analyses using the following steps:

1. Identify the types of earthquakes that control the ground motion hazard.

2. Select a representative set of at least seven pairs of accelerograms recorded during past earthquakes that are compatible with the controlling events and site condition.

3. Modify those motions in some manner to achieve a match with the target spectrum, either using spectral matching or amplitude scaling.

The following sections provide details on these processes.

5.4.2 Identification of Controlling Sources

Where Maximum Considered Earthquake shaking is controlled by probabilistic seismic hazard analysis, deaggregate the ground-motion hazard for the Maximum Considered Earthquake spectral accelerations at the structural natural periods of interest, and use the results as the basis for selecting representative accelerograms for response history analysis. The structural natural periods of interest will include, as a minimum, the first three translational periods of structural response in each of the structure's two principal orthogonal response directions.

***Commentary**: In probabilistic seismic hazard analysis results some of the considered earthquakes contribute much more to the computed hazard than others. Deaggregation of a point on the hazard curve identifies the relative contributions of various seismic sources, earthquake magnitudes and distances to the computed ground motion. Figure 5.2 shows an example deaggregation for a site in Los Angeles.*

In the figure, the height of the bars at different magnitudes and distances provides information on controlling sources. Deaggregation can also provide information on the degree to which relatively average or extreme ground motions from the ground motion prediction equations contribute to the hazard. This is accomplished through the parameter ε (epsilon), which is defined as:

$$\varepsilon(T)\,|\,(M,R) = \frac{\ln\left(S_a(T)\right) - \mu_{\ln,Sa}(T)\,|\,(M,R)}{\sigma_{\ln,Sa}(T)\,|\,(M,R)} \tag{5.1}$$

where S_a is the level of the spectral response acceleration under consideration (e.g., a spectral acceleration of 0.5 g at a natural period T of interest), $\mu_{\ln Sa}$ is the median ground motion for a given magnitude and distance (M and R) from a ground motion prediction equation and $\sigma_{\ln Sa}$ is the standard deviation from the ground motion prediction equation. Values of ε for different M, R combinations are shown by the colors of the bars in Figure 5.2. The dark blue colors in the figure indicate that relatively extreme realizations of the ground motion prediction equation are controlling the hazard (i.e., ground motions well beyond the median).

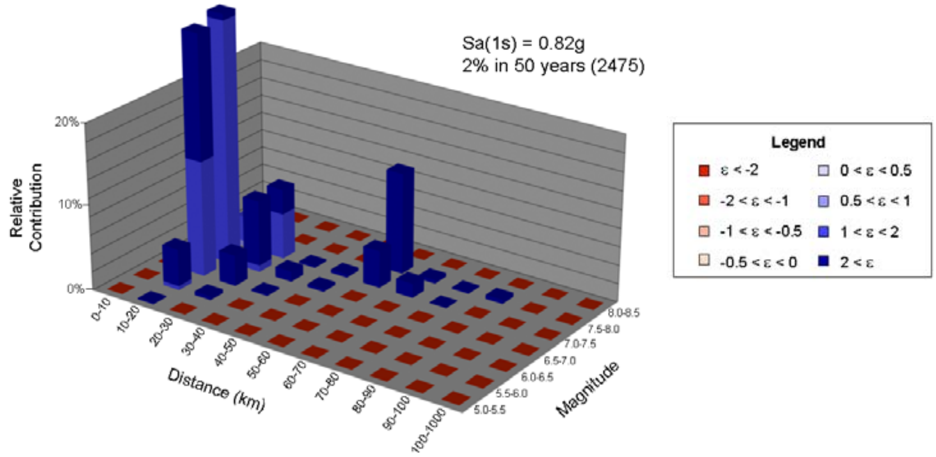


Figure 5.2 Example hazard curve for a site in Los Angeles. The selected IM is 5%-damped 1.0 sec pseudo spectral acceleration and the hazard level is 2% probability of exceedance in 50 years. (Goulet et al., 2007).

For very tall buildings, the fundamental period could be 4 sec or more, which can introduce several practical challenges. First, the deaggregation data from the USGS website is only available for periods of 2 sec or less. Because deaggregation results are generally strongly period-dependent, hazard analysis based on the USGS web site should not be used for buildings with fundamental periods significantly beyond 2 sec. The NGA ground motion predictive equations are capable of predicting ground motions up to 10 sec for active regions. For subduction earthquakes, ground motion predictive equations are not available for periods beyond 3-5 sec, which precludes direct hazard analysis and deaggregation at longer periods.

5.4.3 Record Selection

As required in current building codes, use a minimum of seven accelogram sets for response history analysis. Each accelerogram set selected must consist of at least two horizontal components, and in rare cases, the vertical component may also be included. Select records that are generally compatible with the earthquake magnitude and site-source distance found from deaggregation. If multiple magnitude-distance combinations contribute significantly to the hazard, then select records from each contributing earthquake as part of the total number of records.

When the hazard is controlled by faults producing moderate to large magnitude earthquakes at close distances to the site, an appropriate number of the selected ground-motion records should include near fault and directivity effects, such as velocity pulses producing relatively large spectral ordinates at relatively long periods.

Commentary: *Two important considerations in record selection are the number of records to be used for analysis and the attributes of the selected records. If the* *intent of response history analysis is to reliably characterize both the central value (mean or median) of demand parameters as well as the possible dispersion, a large number of record sets, on the order of 20-30 would be needed because of significant scatter of structural response to ground motion. However, it has become standard practice to use fewer records because of practical difficulties in running large numbers of nonlinear response history analyses. When these smaller numbers of records are used for analysis, the dispersions in response quantities obtained from the analysis should not be considered to be a reliable estimate of the true dispersion. Such dispersions should be either adapted from other research projects that used much larger sets of input ground motions (e.g., Goulet et al. 2007, Moehle et al, 2008), or the designer should use a much larger set of input motions to estimate the scatter of the structural responses.*

Where multiple earthquake events have significant contribution in the deaggregation, it may be necessary to select a larger suite of motions than the seven typically used, to adequately capture the range of response the structure may exhibit in events that could produce Maximum Considered Earthquake shaking.

As described in Section 5.4.2, deaggregation of seismic hazard for long-period spectral accelerations will often indicate large magnitude earthquakes as a controlling source. Record selection for such events is challenging because few such events have been recorded.

Recent research has suggested that record attributes such as magnitude, distance, etc. can produce large dispersion in predictions of certain response quantities such as story drift (e.g., Baker and Cornell, 2006a). This has motivated the development of an alternative approach for record selection, in which the focus is on spectral shape near the periods of interest in lieu of (or in combination with) magnitude, distance, and similar parameters. Parameter epsilon (defined in Eq. 5.1) has been found to be correlated to spectral shape (Baker and Cornell, 2006a), with large epsilon at a given period (T_1) typically associated with rapid decay of spectral ordinates for $T > T_1$.

When using seismological simulation techniques, engineers are cautioned to only use motions from adequately calibrated models that are judged to provide reasonable results. The selected simulation method should incorporate realistic models of fault rupture and wave propagation, including the effects of alluvial basins, which are known to amplify long period ground motions. Moreover, the simulations should be repeated for multiple reasonable realizations of key source parameters (such as slip distribution, rupture velocity, rise time, etc.).

5.4.4 Modification of Accelerograms to Match Target Spectra

Match records either to the uniform hazard spectrum or conditional mean spectrum. If the conditional mean spectrum approach is used, use a suite of conditional mean spectra, each matched to one of the key periods described in Section 5.4.2. Use of conditional mean spectra for only the fundamental period is not recommended for tall buildings.

Match records to the target spectra either by direct scaling or spectral matching.

Commentary: *There are two principal considerations in the modification of accelerograms to match target ground motion intensity measures. The first is the manner by which the record is modified. The second consideration is the target response spectrum that should be considered in the modification process..*

Two principal procedures are used for ground motion modification: direct scaling and spectral matching. The direct scaling procedure consists of determining a constant scale factor by which the amplitude of an accelerogram is increased or decreased. Because elastic response spectra correspond to linear response of a single-degree-of-freedom system, the same scale factor applies to spectral accelerations at all periods. Spectral matching adjusts the frequency content of accelerograms until the response spectrum is within user-specified limits of a target response spectrum over a defined period band. Alternative procedures for spectral matching are elaborated in Chapter 7 of Stewart et al. (2001).

Target spectra can be developed using one of the two following options: (1) the design response spectrum developed from building code procedures (which corresponds roughly to the uniform hazard spectrum for the site) or the uniform hazard spectrum from site-specific analysis; or (2) site-specific scenario spectra (one or more) that preserve realistic spectral shapes for controlling earthquakes and which match the design spectral ordinate at periods of interest to the nonlinear response. In the case of Option 1, the target spectrum is a direct result of the ground motion hazard analysis.

For sites within a few kilometers of an active fault that governs the ground-motion hazard, target response spectra should be developed for the fault-normal (FN) and fault-parallel (FP) directions

Baker and Cornell (2005) describe the mathematical procedure for computing the conditional mean spectrum for a given matching period. The matching periods should be selected in consultation with the structural engineer, and will include the elongated fundamental mode period of the structure due to inelastic structural response. Higher mode periods should also be considered. Note that considering additional periods implies additional conditional mean spectra. When multiple conditional mean spectra are used, multiple suites of each response parameter are obtained from response history analyses. In this case, the envelope value of the response parameter from each suite of analyses should typically used be for design purposes.

6 PRELIMINARY DESIGN

6.1 General

The growing body of experience resulting from the design of tall buildings using performance-based procedures provides much insight that can be used to guide the preliminary design process. This Chapter provides a resource, highlighting important topics shown by experience as critical to consider early in the design process.

***Commentary**: Providing a step-by-step guide for preliminary design of tall buildings conflicts directly with the design innovations these towers many times evoke. Each building and site offers new and unique challenges, requiring careful and specific consideration without preset formulation. The creative design process is generally nonlinear. Therefore, a formal recipe seems out of place. In keeping with this ideal, this section pursues an alternative route, suggesting important design considerations but not providing prescriptive approaches to resolution of the associated issues.*

6.2 System Configuration

To the extent possible, configure the structure to include a simple arrangement of structural elements with clearly defined load paths and regular structural systems. Configurations and geometries which complicate behavior, add to complexity of analysis and uncertainty, and which should therefore be avoided to the extent possible include:

- Large changes in building stiffness (Figure 6.1)
- Large changes in building mass (Figure 6.1)
- Repositioning of bracing elements from floor to floor (Figure 6.2)
- Interaction of two or more towers through a common base structure (Figure 6.3)
- Significant column transfers or offsets (Figure 6.4)
- Gravity induced horizontal shear forces caused by system eccentricities (Figure 6.5)
- Limited connectivity of bracing elements to floor diaphragms

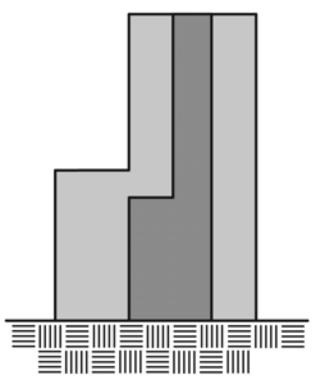


Figure 6.1 Illustration of building with large changes in stiffness and mass

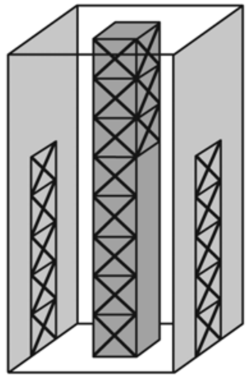


Figure 6.2 Illustration of lateral system with bracing elements repositioned over structure's height

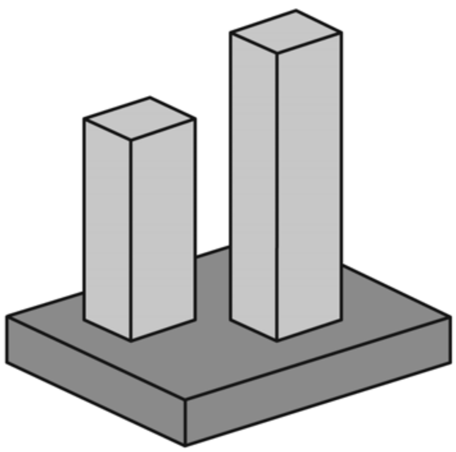


Figure 6.3 Illustration of two towers on a common base

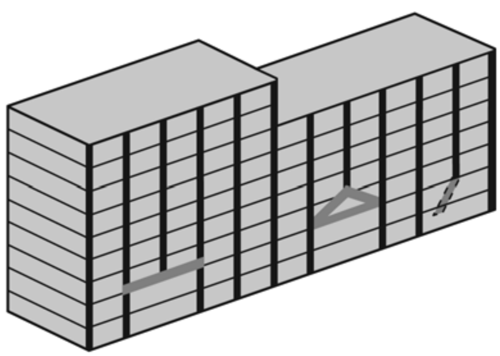


Figure 6.4 Illustration of undesirable column transfer and offset conditions

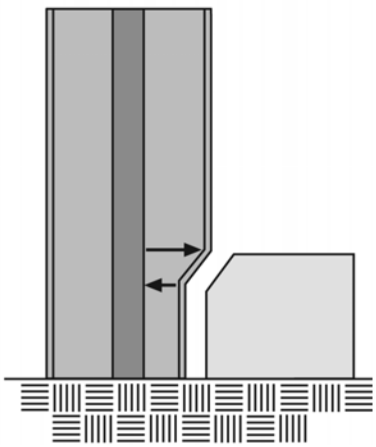


Figure 6.5 illustration of building geometry resulting in gravity-induced shear forces

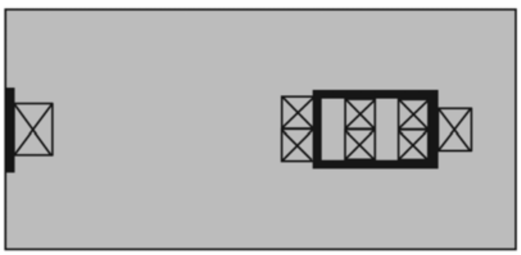


Figure 6.5 Illustration of diaphragms with limited connectivity to vertical elements of the seismic force resisting system

> ***Commentary**: Avoidance of the conditions discussed above will allow for a greater degree of confidence in predicting system behavior. The assumptions inherent in any mathematical structural model add to the uncertainty in predicting behavior. Some of these uncertainties can be eliminated through a simple, well-conceived geometry, thus reducing the analytic studies required to test and prove system behavior.*
>
> *A regular, well-defined system may seem irreconcilable with modern architectural expression. However, a disciplined approach to the architectural design of a tall building, incorporating important structural principles, will generally lead to the most well-behaved and economical structure.*
>
> *This list of irregularities described is by no means comprehensive, nor can these items be avoided in all buildings. As a structure becomes more complex, the uncertainty in predicting its response escalates, requiring more robust analytic work to adequately test and demonstrate performance.*

6.3 Structural Performance Hierarchy

As the structural concept for a tall building is being developed, clearly identify zones or elements where nonlinear response is anticipated. Capacity design concepts are a good starting point when considering desirable system and element actions. While a strict application of capacity design may not be practical or even warranted in the final design, early consideration of these principles will help establish a clear hierarchy to the anticipated building response and will serve to guide the development of the design, which will later be confirmed through nonlinear response history analysis.

> ***Commentary****: Identification of zones of inelastic response will provide clarity in the overall design approach and the ensuing analytic work. In addition, contemplating the hierarchy of likely response actions to increasing levels of ground motion will provide direction to guide the details of the design to follow.*
>
> *Capacity design approaches provide a useful means to configure a structure to produce predictable inelastic behavior. However, the higher mode response common in tall buildings can lead to inelastic behavior in zones that simplistic approaches to capacity design will be unable to predict. Ultimately, engineers must rely on analytical verification of behavior to detect any additional zones of inelastic behavior other than those suggested by initial capacity design proportioning of the structure.*

6.4 Wind

Ensure that the lateral force resisting system is adequate for wind resistance considering both strength and serviceability criteria.

> ***Commentary****: While this guide focuses primarily on seismic design, it is important to remember that the structural response to wind effects may control the strength and stiffness requirements for tall buildings. Many times occupant comfort related to building accelerations in wind events is the controlling design criteria, directly influencing the required building stiffness to appropriately manage these actions.*
>
> *The overall strength of the structural bracing system may be controlled by wind demands. Wind overturning moments and shears in most tall buildings are more closely related to first mode dynamic response, whereas seismic overturning moments and shears can be heavily influenced by higher dynamic modes of vibration. The net result can be substantially higher wind demands as compared to seismic demands at the base of a tall building, whereas seismic demands may find their peak at the mid-height of the tower.*
>
> *Wind tunnel studies which capture the dynamic actions of a tall building within the context of its surroundings are important.*

6.5 Higher Mode Effects

Consider the potential effects of higher mode response when proportioning the main seismic force resisting system.

***Commentary**: It is common for higher dynamic modes of vibration to heavily influence tall building response to ground shaking. Traditional engineering practice has focused strictly on the first translational mode when setting strength requirements and lateral force distributions. For tall buildings, the second or even third mode of vibration can be equally, if not more, important to the overall design.*

As illustrated in Figure 6.6, the influence of these higher modes of vibration can result in significantly higher flexural demands, well above a building's base, as well as shear demands three to four times greater than those anticipated by a typical prescriptive design. Failing to recognize and incorporate these demands into a tower's design can lead to undesirable performance at worst and the need to iterate nonlinear analyses and redesign several times at best.

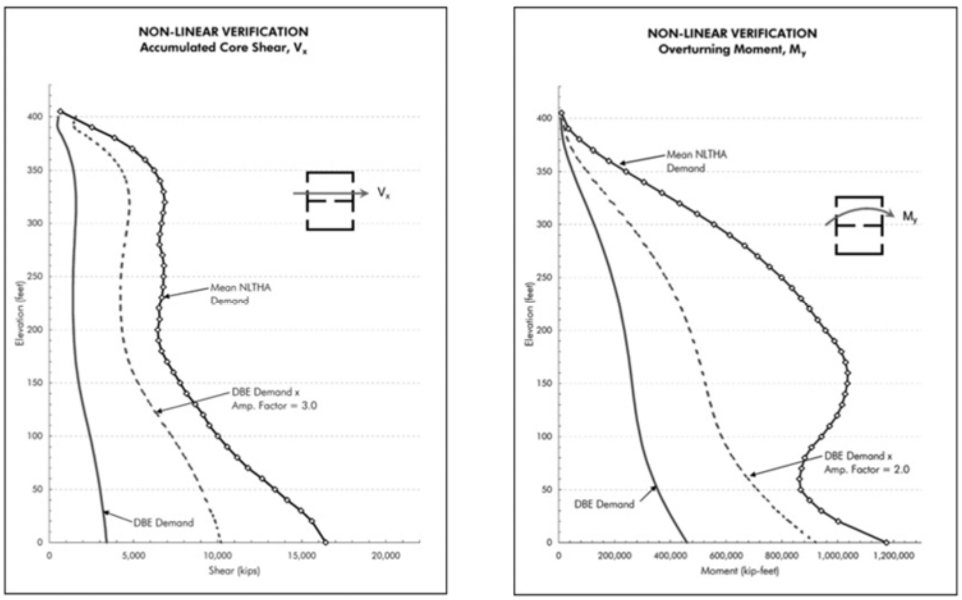


Figure 6.6 Higher mode effects on shear and flexural demand distributions in a tall core-wall building

6.6 Seismic Shear Demands

Consider limiting shear stress demands in concrete walls under Service-level seismic forces to the range of $2\sqrt{f_c^{'}}$ to $3\sqrt{f_c^{'}}$ where $f_c^{'}$ is the specified concrete compressive strength in pounds per square inch.

***Commentary**: As noted in the previous section, the dynamic behavior of high-rise buildings can lead to very high shear demands from higher mode effects. Experience has shown that limiting Service-level shear stresses in concrete walls to* the range of $2\sqrt{f_c^{'}}$ to $3\sqrt{f_c^{'}}$ *will generally result in ultimate shear demands within* *maximum shear stress limits, considering Maximum Considered Earthquake ground motions.*

6.7 Building Deformations

Consider limiting roof displacement predicted by elastic response spectrum analysis under Maximum Considered Earthquake shaking to less than 3% of building height.

> ***Commentary**: Evaluation of overall building deformations at the preliminary design stage offers insight, although limited, to the anticipated behavior considering maximum demands levels. Maximum building displacements in the range of 2 to 3 percent of overall height are generally viewed as acceptable for protecting against global instability under Maximum Considered Earthquake shaking. The dynamic characteristics of tall buildings are such that median estimates of total inelastic displacement are predicted well by elastic spectral analysis as long as the structure is not displaced to deformations near those associated with instability.*
>
> *Story deformation is a more complex action to evaluate. While traditional design practice has focused purely on translational movements, actions in tall buildings related to shear deformation as opposed to total deformation can be equally important. An AISC Journal paper (Griffis, 1993) provides greater insight on this topic. Story deformations and their impact on architectural finishes are the key design parameters to consider.*

6.8 Setbacks and Offsets

Attempt to avoid setbacks and offsets in the lateral force-resisting system. Where such geometric configurations are unavoidable due to architectural considerations, consider the provision of supplemental strength and/or detailing for ductile behavior in the areas of these conditions.

> ***Commentary**: Setbacks in overall building geometry or offsets in lateral bracing elements generally lead to a concentration of demands. Care should be taken in these areas during preliminary design to allow for adequate member sizing, anticipating robust detailing requirements in the final design.*
>
> *Setbacks in concrete core walls or lateral bracing can result in a high concentration of strain demands through the geometry transition. The potential results include localized yielding of structural elements and the need for robust concrete confinement and/or steel detailing.*
>
> *Offsets in bracing systems can also result in significant diaphragm demands. Due consideration of the stiffness degradation of these transfer diaphragms as well as the details of structural "drag" and/or chord elements will be required during later stages of the design process.*

6.9 Diaphragm Demands

Pay careful attention to the configuration and strength of diaphragms at transitions in the lateral force resisting system.

***Commentary**: Diaphragm demands on the floor systems of typical high-rise floors are generally nominal, unless the given floor configuration is long and slender with widely spaced bracing elements or features offsets in the primary lateral bracing system.*

Diaphragm demands at transitions in building geometry (such as a podium structure) or at the base of a building can be extraordinary and warrant special attention early in the design process. Large shear reversals (back-stay forces) may be predicted by the structural analyses. If these load paths are to be realized, many times limitations on diaphragm openings and offsets are required. These requirements can be particularly onerous at the ground level of a building where garage entry ramps, loading docks, retail spaces, and landscaping design often result in geometrically complex floors. Early coordination with the architect is imperative to ensure adequate load paths will exist in the completed structure.

6.10 Outrigger Elements

Outrigger systems are often included in high-rise buildings designs in order to reduce overturning demands on slender vertical elements of the lateral force-resisting system (Figure 6.7). It is important to consider the impact of the outriggers on the supporting columns and walls under maximum demand levels. For example, an outrigger supported by a perimeter column may be capable of delivering an axial load much greater then traditionally considered. Evaluating the over-strength characteristics of an outrigger system and the potential impacts on axial and shear demands is critical to ensuring the overall building system will perform as expected.

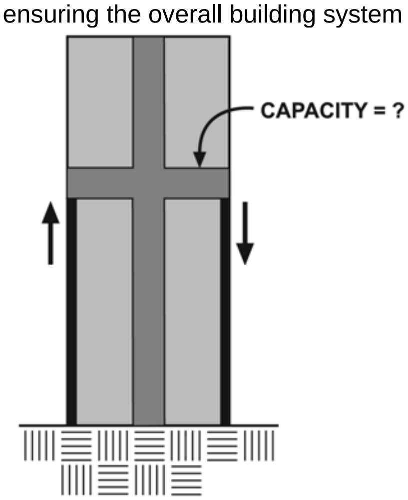


Figure 6.7 Illustration of outriggers used to reduce overturning demands at base of vertical elements of the seismic force-resisting system

6.11 Non-participating Elements

Consider the impacts of all building elements on the ultimate behavior and element demands. In addition to providing for deformation compatibility of gravity load resisting elements consider that axial and shear demands on columns and walls can be significantly influenced by interaction with "gravity framing."

> ***Commentary****: Traditional seismic design practice has assigned primary bracing requirements to a few select elements, while the remaining features of the structure have been deemed as "non-participating elements." This is merely a simplification of the real building actions. Elements intended only to provide gravity resistance can greatly influence the behavior of the main lateral force resisting system and also attract substantial seismic induced stresses themselves.*

6.12 Foundations

The subject of soil-foundation-structure interaction is complex and often neglected in the design process. Due consideration should be given to the uncertainties related to soil-structure interaction. Traditional practice has input seismic ground motions to structural analysis models at the ground's surface in the form of free-field motions. Many times, tall buildings have significant substructures which may play an important role in overall building behavior. A well-considered approach to this topic should be developed during the preliminary design stage. Bounding the response of the structure by varying the foundation support assumptions may be a practical way to address this complex issue. Section 5.3 provides more detailed discussion.

6.13 Slab – Wall Connections

Important to the integrity of buildings supported in whole or part by concrete core walls is the connection between the floor slabs and core walls. As a tower sways due to wind or seismic induced motion, slab-wall connections rotate due to the translation of the structure. In addition, these connections experience vertical motions due to the elongation and shortening of the core walls under flexural action. These vertical displacements compound the translation demands for top reinforcing.

These demands as they relate to post-tensioned floor slabs were investigated and reported in Klemencic (2006), which article makes specific detailing suggestions to enhance the performance of this connection.

6.14 Slab – Column Connections

Robust detailing of slab-column connections in slab-column systems is important to the integrity of tall concrete buildings. As slab-column connections experience lateral deformations, increased moment and shear demands result. These demands may result in yielding of slab reinforcing steel. More critical is the increased shear demand. Robust details which address/prevent punching shear failure are essential.

7 SERVICE LEVEL EVALUATION

7.1 General

This Chapter provides recommended Service-level evaluation criteria including shaking hazard level, performance objectives, modeling, design parameters and acceptance criteria.

7.2 Service-level Earthquake Shaking

Service-level earthquake shaking shall have a minimum mean return period of 43 years (50% probability of exceedance in 30 years). Service-level Earthquake shaking shall be represented in the form of a site-specific, 2.5%-damped, elastic, uniform hazard acceleration response spectrum. If nonlinear response history analysis is performed, ground motions shall be selected and modified to be compatible with the Service-level spectrum in accordance with the recommendations of Chapter 5.

Commentary: *Since these guidelines include no design level earthquake evaluation, many engineers will use service-level earthquake shaking, together with wind demands, to set the structure's strength in preliminary design, that is later confirmed for adequacy as part of the Maximum Considered Earthquake shaking evaluation. In regions of relatively high seismicity including Los Angeles, San Francisco and Seattle, service-level shaking will result in required building strength that is of the same order as the strength required using the prescriptive building code procedures. However, in some cities with lower seismicity, including Portland, Oregon; Sacramento, California; and, Salt Lake City, Utah; service-level shaking will result in substantially less required strength than would conformance with the building code. Engineers designing buildings in locations with this lower seismicity should be aware of this and that service-level strength requirements may not result in a building of adequate strength. Chapter 8 provides additional discussion of this issue.*

A number of studies have attempted to characterize the effective damping in real buildings. These studies range from evaluation of the recorded response to low amplitude forced vibrations to review and analysis of strong motion recordings. Using data obtained from 8 strong motion California earthquakes Goel and Chopra (1997) found that effective damping for buildings in excess of 35 stories tall ranged from about 2% to 4%. Using data obtained from Japanese earthquakes, Satake et al. (2003) found effective damping in such structures to be in the range of 1% to 2%. Given this information and the impossibility of precisely defining damping for a building that has not yet been constructed, these guidelines recommend a default value of 2.5% damping for all modes as a reasonable estimate for use in Service-level evaluations.

The ASCE 7.10 standard requires that buildings assigned to Risk Categories III and IV have minimum strength respectively at least 125% or 150% of the strength required for buildings in lower Risk Categories. One way to achieve compatibility with this requirement is to increase the amplitude of the Service-level spectrum for such buildings by a factor of 1.25 for Risk Category III and 1.5 for Risk Category IV.

Another approach would be to use a somewhat longer return period for the service-level spectrum.

Regardless of the return period used for service-level motion, the free-field design spectra obtained from seismic hazard analysis should not be reduced for embedment or kinematic effects unless specific soil structure interaction analyses are undertaken.

7.3 Performance Objective

Some, very limited structural damage may be anticipated when a tall building is subjected to Service-level earthquake shaking. If not repaired, this damage should not affect the ability of the structure to survive future Maximum Considered Earthquake ground motions. However, repair may be necessary for cosmetic purposes and to avoid compromising the building's long term integrity for fire resistance, moisture intrusion and corrosion.

Commentary: *Tall buildings may have useful lives of 100 years or more. Therefore, shaking with a 43 year return period is representative of events that a tall building may be expected to experience at least once and possibly several times during its useful life. The performance expectation expressed herein assumes that service-level events affect the building before, rather than after a more severe event. It is recognized that severe shaking, of the level of the design shaking defined by the building code or of Maximum Considered shaking could result in conditions that would render the building more susceptible to damage under Service-level events that may follow.*

Because tall buildings may house hundreds to thousands of individuals, either as residences or places of business, it is desirable that such buildings remain operable immediately after a service-level event. Such performance is achievable with minor structural damage that does not affect either immediate or long term performance of the building and therefore does not compromise safety associated with continued building use. Repair, if required, should generally be of a nature and extent that can be conducted while the building remains occupied and in use, though some local disruption of occupancy, around the areas of repair may be necessary during repair activities.

It is important to note that the fitness of a tall building for occupancy depends not only on its structural condition, but also the functionality of key nonstructural components including elevators, stairs, smoke evacuation systems, fire sprinklers and alarms, plumbing and lighting. This guideline does not cover the design of these nonstructural features, but rather, assumes that as a minimum, these components and systems will be designed and installed in accordance with the requirements of the applicable building code and that such design will be adequate to provide the required protection for service-level events. It should be noted that the design of many such components requires determination of a design displacement which is typically obtained from an elastic analysis for design earthquake shaking.

If unique features of the building's structural design results in response likely to lead to increased susceptibility of these critical nonstructural components to failure, alternative means to protect these critical systems should be considered.

7.4 Analysis Method

7.4.1 General

As a minimum, Service-level evaluation shall include a response spectrum analysis in accordance with Section 7.4.2. When demand to capacity ratios determined from such analysis exceed acceptable levels, either the structure can be redesigned or nonlinear response history analysis, in accordance with Section 7.3.3, may be used to investigate and possibly demonstrate that performance is acceptable.

7.4.2 Response Spectrum Analysis

Elastic response spectrum analysis shall be conducted using three-dimensional models and two horizontal components of motion represented by the elastic design spectra defined in Section 7.2. The analysis shall include sufficient modes to include participation of at least 90 percent of the building's mass for each principal horizontal direction of response. Modal responses shall be combined using the Complete Quadratic Combination (CQC) method. The corresponding response parameters, including forces and displacements, termed herein Elastic Response Parameters shall be used to evaluate acceptable performance.

***Commentary**: The results of Service-level elastic response spectrum analysis should not be modified by occupancy importance factors, I, response modification coefficients, R, or overstrength factors, Ω_0, nor should the results be scaled to minimum base-shear criteria. Rather, the displacement and strength demands computed form the response spectrum analysis should be compared directly with the acceptance criteria of Section 7.7.*

7.4.3 Nonlinear Response History Analysis

Nonlinear response history analysis may be performed to demonstrate acceptable performance when the demand to capacity ratios computed using the elastic response parameters from the response spectrum analysis exceed the criteria of Section 7.7. When nonlinear response history analysis is performed, modeling shall be in accordance with the recommendations of Chapter 8 and selection and scaling of ground motions shall be in accordance with the recommendations of Chapter 5. Perform analyses using not less than 3 appropriate ground motion pairs, which shall be selected and modified to be compatible with the Service-level response spectrum. If less than 7 ground motion pairs are used, the maximum absolute value of each response parameter obtained from the suite of analyses shall be used to determine acceptable performance. Where 7 or more ground motions are used, the mean value of each response parameter shall be used to determine acceptable performance.

7.5 Elastic Structural Modelling

7.5.1 General

Conduct analyses using a three-dimensional mathematical model of the structure that represents the spatial distribution of mass and stiffness to an extent adequate for calculation of the significant features of the building's linear dynamic lateral response. Models shall include representation of the stiffness of the intended lateral force-resisting system as well as any vertical load-bearing elements and nonstructural components that add significant lateral stiffness or that will experience significant stress in response to Service-level shaking.

Commentary: *Three-dimensional mathematical structural models are required for all analyses and evaluations. Given the current state of modeling capabilities and available software systems, there is no reason to estimate the actual three-dimensional behavior of tall buildings by relying on approximate two-dimensional models. The accuracy obtained by using three-dimensional models substantially outweighs the advantage of the simplicity offered by two-dimensional models.*

Although analytical models used to perform response spectrum analysis as part of a prescriptive code-based design typically do not include representation of elements other than those that comprise the intended lateral force-resisting system, in tall buildings the gravity load-carrying system and some nonstructural components can add significant stiffness. Since the goal of service-level evaluation is to accurately project the building's probable performance under service-level shaking, it is important to include such elements in the analytical model and also to verify that their behavior will be acceptable.

7.5.2 Material Stiffness and Strength

Structural models shall incorporate realistic estimates of stiffness and strength considering the anticipated level of excitation and damage. Use expected, as opposed to nominal or specified properties when computing modules of elasticity. In lieu of detailed justification, values provided in Tables 7-1 and 7-2 may be used for expected material strengths and estimates of component stiffness, respectively.

Table 7-1 Expected Material Strengths

Material		Expected Strength
Structural Steel		
	Hot-rolled structural shapes and bars	
	ASTM A36/A36M	**1.5F_y**
	ASTM A572/A572M Grade 42 (290)	**1.3 F_y**
	ASTM A992/A992M	**1.1 F_y**
	All other grades	**1.1 F_y**
	Hollow Structural Sections	
	ASTM A500, A501, A618 and A847	**1.3 F_y**
	Steel Pipe	
	ASTM A53/A53M	**1.4 F_y**
	Plates	**1.1 F_y**
	All other Products	**1.1 F_y**
Reinforcing Steel		**1.17 times specified F_y**
Concrete		**1.3 times specified f'_c**

Table 7-2 Effective Component Stiffness Values

Component	Flexural Rigidity	Shear Rigidity	Axial Rigidity
Structural steel Beams, Columns and Braces	E_SI	G_SA	E_SA
Composite Concrete Metal Deck Floors	$0.5E_cI_g$	G_cA_g	E_cA_g
R/C Beams – nonsprestressed	$0.5E_cI_g$	G_cA_g	E_cA_g
R/C Beams – prestressed	E_CI_g	G_cA_g	E_cA_g
R/C Columns	$0.5E_cI_g$	G_cA_g	E_cA_g
R/C Walls	$0.75E_cI_g$	G_cA_g	E_cA_g
R/C Slabs and Flat Plates	$0.5E_cI_g$	G_cA_g	E_cA_g

Notes:

E_c shall be computed per ACI 318, using expected material strength per Table 7-1

G_c shall be computed as $E_c/\left(2\left(1+\nu\right)\right)$, where ν shall be taken as having a value of 0.2

7.5.3 Torsion

The mathematical model shall address torsional behavior of the structure. Inherent eccentricities resulting from the distribution of mass and stiffness shall be included. Accidental eccentricities need not be considered for serviceability evaluation.

> ***Commentary**: The ASCE-7 standard requires consideration of accidental eccentricities when determining the forces and displacements used for design. These accidental eccentricities are intended to assure that the structure is torsionally stable and also to account for the real torsional conditions that occur even in nominally symmetric buildings as a result of variation in material strength, tenant build out, furniture and storage loads. These guidelines do not require consideration of accidental torsion because the three-dimensional modal analyses that are required will detect any torsional instability and because in tall buildings, the torsional*

eccentricity associated with random variability in loading and material properties will tend towards a mean of zero when considered over many stories and floor levels.

7.5.4 Beam-column Joints

Modeling of joints in moment-resisting frames shall accurately account for the stiffness of the joint, including the panel zone. In lieu of explicit modeling of beam-column panel zone behavior, center-to-center beam dimensions may be used.

***Commentary**: Additional guidance as to appropriate stiffness assumptions for concrete and steel framing may be found in two publications by the National Institute of Standards and Technology. Moehle, et. al. (2008) and Hamburger, et al. (2009) respectively provide guidance on modeling of concrete and steel special moment frames.*

7.5.5 Floor Diaphragms

Floor diaphragms shall be included in the mathematical model using realistic stiffness properties. Regardless of the relative rigidity or flexibility of floor diaphragms, floors with significant force transfer (i.e., podium effect, etc.) shall be explicitly included in the mathematical model. Diaphragm chord and drag forces shall be established in a manner consistent with the floor characteristics, geometry, and well established principles of structural mechanics. Both shear and bending stresses in diaphragms must be considered. At diaphragm discontinuities, such as openings and reentrant corners, the dissipation or transfer of edge (chord) forces combined with other forces in the diaphragm shall be evaluated.

***Commentary**: Explicit modeling of diaphragms at locations where significant force transfer occurs is necessary to properly capture these effects. Common assumptions of perfectly rigid or flexible diaphragms will not in general be able to capture these effects properly. It is important to recognize that the vertical location of significant force transfer may occur at diaphragm levels adjacent to the level at which frames or walls are discontinued or introduced.*

7.5.6 Foundation-Soil Interface

Soil-foundation-structure interaction analysis is not required for serviceability evaluation. However, the model shall extend to the base mat. Refer to Chapter 5 for additional discussion.

***Commentary:** Soil-foundation-structure interaction typically has little effect on the response of tall buildings. Its effect is most significant with regard to the demands on basement walls and slabs, which have typically been demonstrated to be robust in moderate level shaking. Detailed soil-structure interaction is therefore not necessary for service level evaluations where simple yet generally conservative assumptions suffice.*

7.5.7 Subterranean Levels

The analytical model of the structure:

1) should include the entire building including the subterranean levels (i.e., floors, columns, walls, including the basement walls) ; as shown in Figure 7.1,

2) should include appropriate representation of the mass and mass moment of inertia of the subterranean levels

3) may ignore the horizontal effect of soil surrounding the subterranean levels, and

4) may assume rigid soil beneath the foundations (i.e., no vertical soil springs).

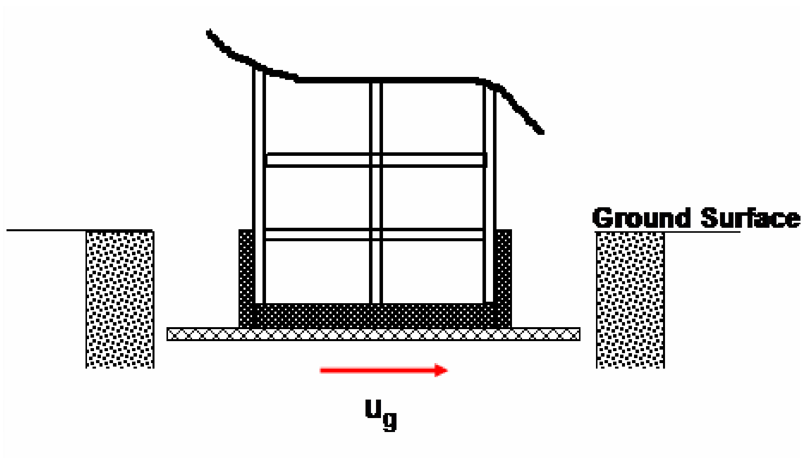


Figure 7.1 Sketch of simplified model for the building and subterranean levels.

7.5.8 Column bases

Use realistic assumptions to represent the fixity of column bases. A column base may be considered fixed if the column base connection to the foundation is capable of transferring columns forces and deformations to the foundation with negligible joint rotation, considering the flexibility of the foundation itself.

7.6 Design Parameters and Load Combinations

Evaluate roof displacement, story drifts, and member forces (axial, flexure, shear, and torsion) for all members that experience significant load as a result of earthquake response.

7.6.1 Load Combinations – Response Spectrum Analysis

Evaluate the structure for the following load combinations:

$$1.0D + L_{exp} \pm 1.0E_x \pm 0.3E_y \quad (7\text{-}1)$$

$$1.0D + L_{exp} \pm 1.0E_y \pm 0.3E_x \quad (7\text{-}2)$$

L_{exp} should be taken as 25% of the unreduced live load unless otherwise substantiated.

Commentary: Code response modification factors do not apply to serviceability evaluation (i.e., *R, Ω_0, ρ, C_d, are all taken as unity*).

7.6.2 Load Combinations – Nonlinear Response History Analysis

When nonlinear response history analysis is used for service-level evaluation, evaluate the structure for the following load combination.

$$1.0D + L_{exp} \pm 1.0E \quad (7\text{-}3)$$

Where L_{exp} shall be taken as described in Section 7.6.1.

7.7 Acceptance Criteria

7.7.1 Ductile Actions

Ductile actions shall be those actions (flexure, tension) for which elements and connections are specifically detailed to accommodate inelastic energy dissipation without unacceptable strength deterioration. Axial compressive resistance of load bearing columns and wall piers shall not be considered ductile actions for this purpose.

***Commentary**: As used in this section, ductile elements are those elements that have been specifically detailed to accommodate inelastic structural behavior. For reinforced concrete elements this will typically require materials and detailing that conform, as a minimum to the confinement and shear strength criteria of ACI 318.08, Sections 21.1.4, 21.1.5, 21.1.6, 21.1.7, 21.5, 21.6, 21.7, 21.8, 21.9, and 21.10, as applicable. For structural steel elements, this will typically require materials and detailing that conform as a minimum to the criteria of ANSI/AISC 341.05 Part I, Sections 6, 7, 8, 9, 12, 13, 15, 16, and 17. For composite steel and concrete elements this will typically require materials and detailing that conform as a minimum to ANSI/AISC 341.05 Part II, Sections 5, 6, 7, 9, 12, 14, 16, or 17. For elements that do not comply with these criteria, substantiating data should be presented to demonstrate adequate inelastic response capability.*

a. Response Spectrum Analysis

Ductile elements shall be permitted to have demand to capacity ratios not exceeding 1.5, where demand is computed from equations 7-1 and 7-2 and capacity shall be calculated as:

(1) For reinforced concrete elements and their connections, the applicable ACI-318 strengths using expected material properties and resistance factors of unity (1.0).

(2) For structural steel and composite steel and concrete elements and their connections, the applicable LRFD strengths in accordance with ANSI/AISC 341.05 and 3601.05 using expected material properties and resistance factors of unity (1.0)

b) Nonlinear Response History Analysis

Deformation demands shall not exceed a value at which sustained damage, as substantiated by suitable laboratory testing, requires repair, for reasons of strength deterioration or permanent deformation, as demonstrated by appropriate laboratory testing. Repair, if required should not exceed a level that requires removal and replacement of structural concrete, other than cover, removal or replacement of reinforcing steel or structural steel. In lieu of the use of laboratory test data, it shall be permissible to use the acceptance criteria for Immediate Occupancy performance as contained in ASCE 41.

7.7.2 Other Actions

Other actions are all those actions (loading and deformation) of structural elements that are not qualified as ductile actions under Section 7.7.1.

Elements with actions other than ductile actions shall be evaluated for their adequacy to resist demands computed from equations 7-1 and 7-2 for response spectrum analysis and equation 7-3 for nonlinear response history analysis. Capacities of reinforced concrete elements shall be computed using specified material strengths and the applicable resistance factors in ACI 318. Capacities of structural steel and composite steel and concrete elements shall be computed using specified material strengths and the LRFD procedures and applicable resistance factors of AISC 360. Capacities of elements other than those within the scope of ACI 318 and AISC 360 shall be determined based on testing and shall provide a suitably low probability of failure considering uncertainties associated with material strength, construction quality, and loading. Demand-capacity ratios shall not exceed 1.5.

***Commentary**: Commentary to chapters 1 and 2 of the ASCE-7.10 standard describe the anticipated performance goals and methods of calculation for resistance factors used by ACI 318, ACI 530.1, AISC 360 and other Load and Resistance Factor format design specifications. For structural elements not covered by those specifications, the computation of appropriate resistance factors should be determined in accordance with the ASCE 7-10 commentary.*

In a yielding structure, internal forces throughout the structure, including in the force-controlled actions, will be limited by the capacities of the yielding components. For example, in a simple structure having demand-capacity ratio of 1.5 for ductile actions, yielding will commence for forces equal to 2/3 of the member capacity (strength). Thus, an appropriate design level for all components is 2/3 times the elastic demand. For this reason, the same demand-capacity ratio of 1.5 is allowed for both force-controlled actions and ductile actions. It is important to have an appropriate strength hierarchy in a structure so that yielding mechanisms are primarily limited to ductile modes. Consequently, a designer may decide that a smaller demand-capacity ratio is appropriate to guide preliminary design decisions for force-controlled actions. Ultimately, nonlinear dynamic analysis under MCE loading (see Chapter 8) will be used to demonstrate that inelastic response is restricted to ductile modes with force-controlled actions responding essentially in the linear range.

7.7.3 Displacements

Story drift shall not exceed 0.5% of story height in any story.

> ***Commentary***: *The story drift limit of 0.5% for service-level shaking is intended to provide some protection of nonstructural components and also to assure that permanent lateral displacement of the structure will be negligible. It is important to understand that at story drift levels of 0.5%, nonstructural damage, particularly for elements such as interior partitions may not be negligible and considerable cosmetic repair may be required.*

8 MAXIMUM CONSIDERED RESPONSE EVALUATION

8.1 Objective

This Chapter sets recommended criteria for Maximum Considered Earthquake shaking evaluation. The objective of this evaluation is to provide adequate safety against collapse. This objective is implicitly achieved by using nonlinear response history analysis to evaluate the structure's response to a limited suite of ground motions that represent Maximum Considered Earthquake shaking as defined in Chapter 5. This response evaluation does not provide a quantifiable margin against (or a probability of) collapse, but is intended to demonstrate that collapse under the selected ground motions does not occur, i.e., the structure maintains stability, and forces and deformations are within acceptable limits.

Commentary: *The seismic design procedures contained in the ASCE 7 standard are intended to assure an acceptably low conditional probability of collapse for structures subjected to Maximum Considered Earthquake shaking. As noted in commentary to the 2009 NEHRP Provisions (BSSC, 2009) and to the ASCE-7.10 standard, for Occupancy (Risk) Category II structures, the target conditional probability of collapse is intended to be 10% or less, with lower acceptable collapse probabilities for structures in higher Occupancy Categories.*

The conditional probability of collapse for a structure, at a particular ground motion intensity, is a function not only of the structure's strength, deformation and nonlinear response characteristics, but also of a number of uncertainties including our ability to predict the ground motion characteristics and our ability to model and predict the structure's response given the ground motion. The technical capability exists to predict the probability of collapse as a function of ground motion intensity (ATC 2008, Zareian and Krawinkler 2007), however the process of collapse prediction is complex and is based on the presumption that the force-deformation characteristics of all important structural components can be modeled for the full range of deformations associated with inelastic behavior leading to collapse. At this time insufficient knowledge exists to model such behavior with confidence for all the types of structural components that might be utilized in tall buildings and the tools available to engineers do not permit such evaluations within the resources and time constraints available on most design projects. Until such knowledge is developed and the available tools improve to a level that will permit practical implementation of rigorous collapse probability evaluation, the stability evaluation recommended in this section is the preferred method for providing adequate safety against collapse.

8.2 Design and Evaluation Process

8.2.1 Design Considerations

Structural system and component selection and layout should pay careful attention to capacity design considerations. The target should be to pre-select zones and behavior modes in which ductility (large inelastic deformation capacity) can be provided through proper detailing, and to tune the structural system such that response evaluation will confirm that: 1) inelastic deformations are indeed concentrated in these zones; 2) inelastic behavior is in desirable behavior modes and 3) excessive force and deformation demands for undesirable behavior modes (such as large shear forces in columns) are prevented.

Desirable behavior modes include, but are not limited to the following:

- Flexural yielding in beams, slabs shear wall piers and coupling beams
- Tension yielding in diagonally reinforced coupling beams
- Tension yielding in steel braces and steel plate shear walls, and tension/compression yielding in buckling restrained braces
- Post-buckling compression in braces that are not essential parts of the gravity load system
- Shear yielding in steel components (e.g., panel zones in moment frames, shear links in EBFs, steel coupling beams)
- Yielding in ductile fuses or energy dissipation devices
- Controlled rocking of foundations

In zones of intended inelastic behavior it is imperative to provide ductile detailing that assures inelastic deformation capacity prior to deterioration in strength.

For behavior modes in which inelastic deformation capacity cannot be assured (force-controlled actions) it is essential to avoid overloads that exceed the reliable force capacity and to protect, through appropriate detailing, against unexpected brittle failure modes (such as fracture in steel moment connections or gusset plate to frame connections in braced steel frames).

> ***Commentary****: Tall buildings are complex dynamic systems and in many cases it will not be possible to identify up front all zones in which inelastic deformations may occur. Nonlinear dynamic analysis will disclose whether or not the pre-selected zones are indeed the only ones in which inelastic deformations are concentrated. An important goal of the response evaluation process is to identify all regions of potential inelastic behavior, whether or not they have been targeted up front as zones of desired inelastic behavior. A typical example of "non-targeted zones" of inelastic behavior is flexural yielding in mid- or even upper stories of shear walls, which often is caused by higher mode effects. If such yielding is observed in the response evaluation, then these "non-targeted zones" have to be detailed appropriately for ductility.*

Deleted:

Except when specifically exempted, design for strength and ductility should conform to all criteria specified in the most recent material codes and specifications and referenced in the most recent *ASCE-7* edition.

If non-standard materials or components are utilized as part of the structural system, sufficient documentation should be provided to justify all assumptions on which the computation of strength and deformation capacities is based. The Structural Peer Review should evaluate the adequacy of the provided information and assumptions.

In the context of design for force and deformation demands, the following structural components and elements require special attention:

- Diaphragms and collectors
- Components and elements affected by podium/backstay effects
- Opening in walls
- Outrigger systems
- Locations of abrupt change in lateral stiffness or mass
- Gravity framing system, unless it is considered part of the lateral load resisting system

8.2.2 Evaluation Criteria

Section 8.3 describes general analysis requirements. Sections 8.4 and 8.5 present recommendations for structural modeling. Section 8.6 presents criteria for evaluating the adequacy of response at a component level. Section 8.7 presents criteria for evaluation of response adequacy at a global level, including consideration of peak transient and residual drift, and loss of story shear strength.

8.3 Loads and Response Prediction

8.3.1 Seismic Input

Analyze the structure for a minimum of seven pairs of orthogonal ground motion components, selected and modified for compatibility with the target Maximum Considered Earthquake shaking spectrum in accordance with the recommendations of Chapter 5.

Apply the pairs of accelerograms along the structure's principal directions of response, unless near-fault directionality effects dominate the ground motion in which case the accelerograms should be applied in the fault-parallel and fault-normal directions.

Derive the effective seismic mass from the full dead loads, including appropriate contributions from partitions and other transient loads that might contribute significantly to structural response.

8.3.2 Contributions of Gravity Loads

The following gravity loads should be applied as initial conditions in the analysis:

$$1.0D + L_{\exp} \qquad (8\text{-}1)$$

Where L_{exp} can normally be taken as 25% of the specified design live load (without reduction) unless case specific conditions demand a larger (e.g., storage loads) or justify a smaller value.

> ***Commentary**: Nonlinear analysis is load path dependent, and the results depend on combined gravity and lateral load effects. The gravity load applied in the analysis should be equal to the expected gravity load. The dead load should include the structure's self weight, architectural finishes (partitions, exterior wall, floor and ceiling finishes), and an appropriate allowance for mechanical and electrical services and equipment. The live load should be reduced from the nominal, unreduced design live load to reflect: 1) the low probability of the full design live load occurring simultaneously throughout the building, and 2) the low probability that the design live load and Maximum Considered Earthquake shaking will occur simultaneously.*

8.3.3 Response Prediction Method

Perform nonlinear response history analysis (NRHA) using a three-dimensional model of the structure including subterranean levels. Soil-foundation-structure interaction effects may be included as described in Chapter 5. Ground motion shall be introduced at the base mat or through soil springs as described in Chapter 5.

> ***Commentary:** Nonlinear static procedures (pushover analysis) may be useful as a design aid, but should not be relied upon to quantify response characteristics for tall buildings. Depending on the option used, they produce results of unknown reliability, and in general are unable to reproduce phenomena that are a consequence of significant inelastic redistribution, such as shear force amplification in shear walls caused by flexural plastic hinging at the base of the wall. There is much intrinsic value to a pushover analysis (for instance it permits graphical representation and visualization of progression of inelastic behavior) and can assist in identifying the primary modes of inelastic behavior under first mode response. However, in many practical cases such an analysis is not capable of capturing the effects of variations in the frequency content of the ground motions and of variations in higher mode effects.*

8.4 System Modelling

The three-dimensional model of the structural system should represent all components and force and deformation characteristics that significantly affect the prediction of seismic demands at the Maximum Considered response level. The implication is that components and force or deformation characteristics that do not significantly affect important demands can be ignored. This might apply to components of the foundation system, its interface with the soil, or to the superstructure. Chapter 5 provides additional guidance on foundation modeling and soil modeling.

> ***Commentary**: Good engineering knowledge and judgment are needed to make these decisions. For instance, if adequate safeguards are taken against excessive shear deformations and shear failure in reinforced concrete components (walls, beams and columns) through the use of appropriate capacity design concepts, then simulation of shear deformations might not be warranted.*

But such decisions will require a careful review of analysis results to verify that the analysis assumptions made are indeed justified, and might require post-analysis strengthening or a re-analysis if the assumptions made are shown to be incorrect.

Evaluate force and deformation demands for all components and elements that form an essential part of the lateral and gravity load path and the failure of which might affect the stability of the structure during and after Maximum Considered Earthquake shaking. Explicitly incorporate in the analysis model components and elements of the gravity load-resisting system that contribute significantly to lateral strength and stiffness. In order to assure adequate performance of elements that are not explicitly modeled, perform a deflection compatibility check for all components, elements, and connections not included in the analysis model considering the maximum story drifts predicted by the analysis.

Significant hysteretic energy dissipation shall be captured directly by inelastic elements of the model. A small amount of equivalent viscous or combined mass and stiffness proportional damping may also be included. The effective additional modal or viscous damping should not exceed 2.5% of critical for the primary modes of response.

Commentary: *Damping effects of structural members that are not incorporated in the analysis model (e.g., gravity framing), foundation-soil interaction, and nonstructural components that are not otherwise modeled in the analysis can be incorporated through equivalent viscous damping. The amount of viscous damping should be adjusted based on specific features of the building design and may be represented by either modal damping, explicit viscous damping elements, or a combination of stiffness and mass proportional damping (e.g., Rayleigh damping). Section 2.4 of ATC (2009) provides a discussion and recommendations for modeling viscous damping in analytical models of tall building structures.*

The analysis model should be capable of representing the flexibility of the floor diaphragms as necessary to realistically simulate distribution of inertia forces to the various vertical elements and to produce information for strength design of diaphragms and collector elements.

Commentary: *Chapter 5 of ATC (2009) provides recommendations for modeling of diaphragms and collectors.*

The analysis model should be capable of representing “podium and backstay effects” caused by locking vertical elements (e.g., shear walls or braced or moment frames) between diaphragms that are supported by stiff vertical elements such as exterior basement walls.

Commentary: *Because of the large in-plane stiffness of the diaphragms and the supporting stiff vertical elements, the locked vertical element may experience a drastic change in shear force and overturning moment below the diaphragm(s) supported by exterior walls. The change in shear force and overturning moment demands will depend strongly on the in-plane stiffness and strength of the diaphragm and its supporting elements, as well as on the foundation type below the podium structure. Perhaps most important are the relative stiffness values.*

Realizing that these stiffness values depend on the extent of cracking, it might be necessary to make bounding assumptions on stiffness properties in order to bracket the forces for which the various components of the podium structure have to be designed. Chapter 5 of ATC (2009) provides recommendations for modeling of the podium and backstay effects.

Represent P-Delta effects in the analytical model, whether or not elastic concepts indicate that such effects are important.

***Commentary**: The widely used elastic stability coefficient ($\theta = P\delta/Vh$) is often an insufficient indicator of the importance of P-Delta effects in the inelastic range. P-delta effects may become an overriding consideration when strength deterioration sets in and the tangent stiffness of the story shear force – story drift relationship approaches zero or becomes negative. When this happens, the story drift ratchets, i.e., it increases in one direction without the benefit of a full reversal that otherwise would straighten out the story. For this reason, and many others, realistic modeling of component deterioration and post-yield stiffness are critical aspects of modeling. The potential for dynamic instability is relatively high in flexible moment frame structures and in braced frames and shear wall structures in which one or several of the lower stories deform in a shear mode and the tributary gravity loads are large so that P-Delta will lead to a significant amplification of story drift demands. Section 2.3 of ATC (2009) provides detailed information on P-Delta effects and why and when it becomes an important consideration in the inelastic response of structures.*

8.5 Structural Component Modelling

8.5.1 Important Modeling Parameters

Hysteretic models must adequately account for all important phenomena affecting response and demand prediction as the structure approaches collapse including: (a) monotonic response beyond the point at which maximum strength is attained; (b) hysteretic properties characterizing component behavior without the effect of cyclic deterioration; and, (c) cyclic deterioration characteristics.

***Commentary**: Hysteretic models based on cyclic skeletons, like those presented in ASCE-41 will often be inadequate to accurately predict demands at response levels approaching collapse. These models under-predict strength and deformation under monotonic loading and loading incorporating few cycles, and depending on the actual load path, may not accurately portray strength and deformation capacity under greater cycles of loading. There are many alternatives for describing hysteretic properties in a manner that better predicts behavior near collapse. Presented here is the alternative discussed in detail in Section 2.2 of ATC (2009). Additional alternatives are also presented in that reference.*

Monotonic response may be characterized by a multi-linear diagram of the type shown in Fig. 8.1 and referred to herein as the monotonic backbone curve. It is described by the parameters shown in Fig. 8.1 and represents the theoretical *component force-deformation behavior, if the component is pushed in one direction, without cycling, to failure.*

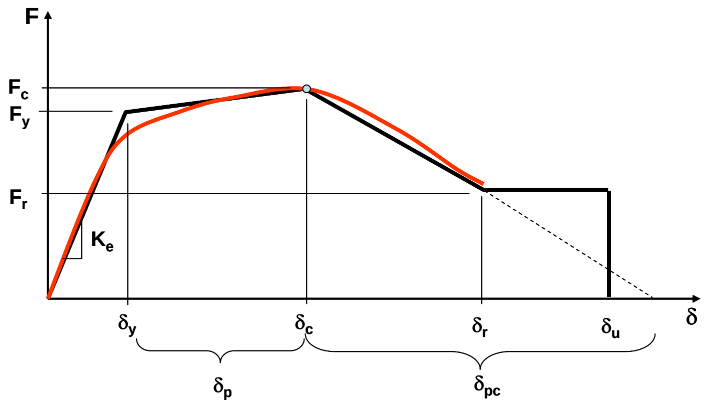


Figure 8.1 Monotonic backbone curve parameters

Key parameters for this monotonic backbone curve are:

- *Effective yield strength and deformation (F_y and δ_y)*
- *Effective elastic stiffness, $K_e=F_y/\delta_y$*
- *Strength cap and associated deformation for monotonic loading (F_c and δ_c)*
- *Pre-capping plastic deformation for monotonic loading, δ_p*
- *Effective post-yield tangent stiffness, $K_p=(F_c-F_Y)/\delta_p$*
- *Post-capping deformation range, δ_{pc}*
- *Effective post-capping tangent stiffness, $K_{pc}=F_c/\delta_{pc}$*
- *Residual strength, $F_r=\kappa F_y$*
- *Ultimate deformation, δ_u*

Hysteretic modeling can follow preselected rules, such as bilinear, peak oriented, or pinched hysteresis rules. Cyclic deterioration can be described by a cyclic deterioration parameter, such as the energy based parameter discussed in Section 2.2.4 of ATC (2009).

The effect of cyclic deterioration is that the point of maximum strength moves closer to the origin, i.e., both the peak strength that can be achieved and the deformation at which peak strength is realized become smaller with successive cycles. The amount that the peak strength and deformation at which peak strength is attained move with increasing cycles depends on the loading history.

There are important differences between monotonic backbone curves like that shown in Figure 8-1 and cyclic envelope curves such as obtained from cyclic laboratory testing (Section 2.2.4 of ATC (2009)) such as shown in Figure 8-2. The monotonic curve will exhibit greater deformation capacity and reduced post-peak strength negative stiffness, relative to the envelope curves obtained from typical cyclic component tests.

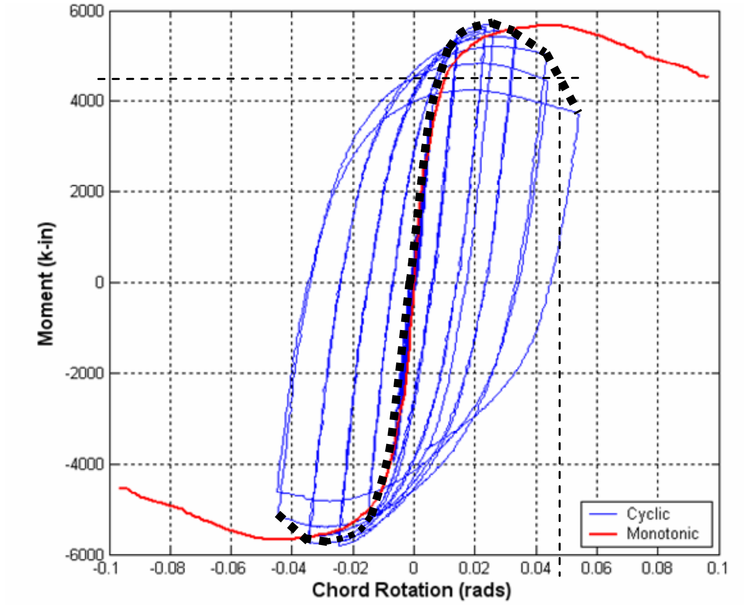


Figure 8-2 Typical Monotonic Backbone Curve and Cyclic Envelope Curve

8.5.2 Methods for Computing Component Properties

The properties of the component backbone curve and of cyclic deterioration characteristics may be obtained from a combination of appropriate analytical approaches and experimental observations. Table 8.1 lists sources of deterioration that should be considered unless precluded by detailing.

Table 8.1 Sources of hysteretic deterioration

Structural Steel	Reinforced Concrete or Masonry
Compressive buckling of members	Tensile cracking, crushing and spalling
Local buckling of flanges or webs	Rebar buckling and fracture
Lateral torsional buckling of members	Bond slip
Ductile tearing of base metal	Loss of reinforcement anchorage
Fracture of weldments	Dowel action
Net section fracture of tensile elements	Confinement steel fracture
Bolt slippage	Reduction in aggregate interlock
Block shear failure	Sliding at joints
Bolt yielding and bearing	
Prying action	
Shear buckling	

Strength properties of the component backbone curve should be based on expected material strengths. The values provided in Table 7.1 may be used for expected material strengths.

Acceptable hysteretic modeling can be attained by detailed continuum finite element models, curvature and fiber models, and experiment-based phenomenological models.

> ***Commentary:*** *Continuum finite element models are usually appropriate provided that cyclic material properties and the aforementioned deterioration/failure modes are adequately simulated. The cost of analysis is prohibitive in most practical applications.*
>
> *Curvature and fiber models can be appropriate provided all important deterioration modes can be simulated adequately. Great difficulties are often encountered in simulating deterioration due to local and lateral torsional buckling in steel components, and rebar buckling, bond slip, and shear deformations in reinforced concrete components. Thus, the use of such models often necessitates the specification of artificial limits to simulate these often critical deterioration modes. It is inappropriate to ignore these deterioration modes in curvature and fiber models. In cases of important bi-axial load effects (e.g., many columns and shear wall configurations) such models may present the only viable alternative. However, models of this type need to be validated through experimental results in order to capture, through strain limits or other means, limit states beyond which severe deterioration is likely and no reliance can be placed on a reproduction of response.*
>
> *Phenomenological modeling usually implies the use of concentrated hinge models whose properties are determined from principles of engineering mechanics and are calibrated by means of experimental data. This requires the availability of experimental databases that can be employed to calibrate a phenomenological model for a wide range of parameters. Several extensive databases are available for steel and reinforced concrete beam and column components and have been used to calibrate parameters for the generic deterioration model discussed in Section 8.5.1. Point hinge models are difficult to implement in components subjected to bi-axial bending (many columns and shear wall configurations) and large variations in axial force.*

8.5.3 Options for Component Analytical Models

Deformation capacities may be taken equal to the corresponding Collapse Prevention values for primary elements published in *ASCE 41* (with Supplement 1) for nonlinear response procedures, or may be based on analytical models validated by experimental evidence. When applicable, the *ASCE 41* component force versus deformation curves may be used as modified backbone curves, with the exception that the drop in resistance following the point of peak strength should not be as rapid as indicated in the *ASCE 41* curves. Alternatively, the modeling options presented in ATC (2009) may also be employed.

> *Commentary: The rapid post-peak drop in resistance indicated in the ASCE-41 curves is not realistic (unless fracture occurs) and is likely to cause numerical instabilities in the analysis process.*

Component models that account neither for post-capping strength deterioration nor for cyclic deterioration should not be used for Maximum Considered Earthquake response evaluation, unless appropriate limitations on the maximum deformation are specified and no credit is given to undefined strength properties beyond this level of deformation. The choice of an appropriate component modeling option and of the basic hysteresis model used to represent the cyclic response of structural components should be justified and become part of the analysis documentation.

Commentary*: Section 2.2.5 of ATC (2009) proposes the following four options for component analytical models.*

<u>Option 1 – explicit incorporation of cyclic deterioration in analytical model</u>: This option explicitly incorporates post-capping strength deterioration and cyclic deterioration in the analytical model, by using the monotonic backbone curve as a reference boundary surface that moves "inward" (towards the origin) as a function of the loading history. This is the preferred option.

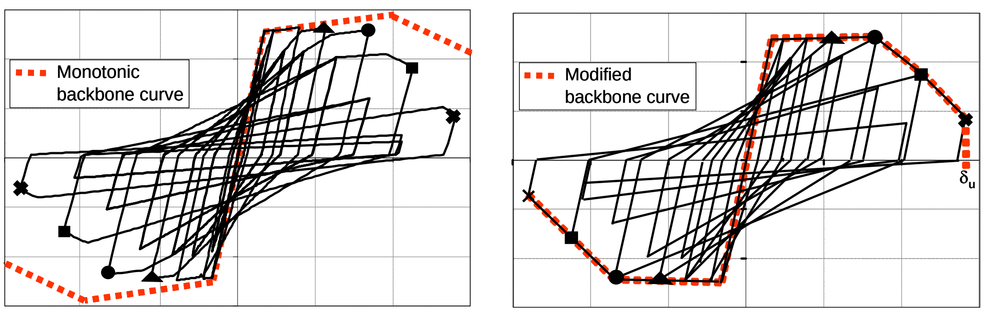
<u>Option 2 – use of a cyclic envelope curve as a modified backbone curve; cyclic deterioration is not considered explicitly.</u> If the cyclic envelope curve is known (e.g., from a cyclic test that follows a generally accepted loading protocol) then it is acceptable to use this envelope curve as the modified backbone curve for analytical modeling and ignore additional cyclic deterioration - provided that no credit is given in the analysis to undefined strength characteristics beyond the bounds established by the cyclic envelope curve, i.e., the ultimate deformation δ_u in any analysis should be limited to the maximum deformation recorded in the cyclic test. When using this approximation, one must make sure to include the negative tangent stiffness portion of the cyclic envelope curve as part of the modified backbone curve of the analytical model.

<u>Option 3 – use of factors for modification of backbone curve; cyclic deterioration is not considered explicitly:</u> If only the monotonic backbone curve is known (or predicted) and cyclic deterioration is not incorporated in the analytical model, then the shape of the backbone curve must be modified to account approximately for cyclic deterioration effects. Numerical values of the modification factors might depend on material, configuration, and detailing of the structural component. Until more accurate and component-specific data becomes available, it is recommended to use the following values for the modified backbone curve:

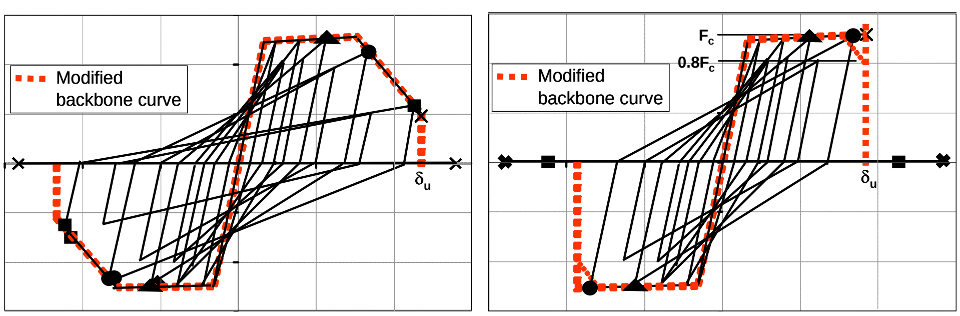
- *Strength cap F_c': 0.9 times the monotonic backbone curve value F_c*
- *Pre-capping plastic deformation δ_p': 0.7 times the monotonic backbone curve value δ_p*
- *Post-capping deformation range δ_{pc}': 0.5 times the monotonic backbone curve value δ_{pc}*
- *Residual strength F_r': 0.7 times the monotonic backbone curve value F_r*
- *Ultimate deformation δ_u': 1.5 times δ_c of the monotonic backbone curve.*

<u>Option 4 – no deterioration in analytical model:</u> If the post-capping (negative tangent stiffness) portion of the modified backbone curve of option 2 or 3 is not incorporated in the analytical model (i.e., a non-deteriorating model is employed), then the ultimate deformation of the component should be limited to the deformation associated with 80% of the strength cap on the descending branch of the modified backbone curve as obtained from option 2 or 3. No credit should be given in analysis to undefined strength characteristics beyond this deformation limit.

Figure 8.3 illustrates the four options for a typical experimental cyclic loading history and a peak-oriented hysteresis model. Several equivalent points of equal peak displacement for the four options are identified with symbols. The differences appear to be small, but primarily because the illustrations are for a symmetric and step-wise increasing loading history, which is typical for experimental studies but not for response at the Maximum Considered Earthquake shaking levels. As intended, the larger the simplification the more the inelastic deformation capacity is being reduced. This is most evident in Figs. 8.3(c) and (d), in which the attainment of the estimated δ_u limits the inelastic deformation capacity.



(a) Option 1 – with cyclic deterioration *(b) Option 2 – modified backbone curve = envelope curve*



(c) Option 3 - modified backbone curve = factored monotonic backbone curve *(d) Option 4 – no strength deterioration*

Figure 8.3 Illustration of implementation of the four options for analytical component modeling

8.5.4 Specific Component Modeling Issues

8.5.4.1 Steel beams and columns in bending.

The rotation values provided in Section 3.2 of *ATC (2009)* should be employed rather than those given in *ASCE 41*. The deformation values given in ATC (2009) are for the monotonic backbone curve illustrated in Fig. 8.1 and shall be modified unless modeling option 1 is utilized

> ***Commentary**: These values are based on the assumption that point hinge models are used to represent inelastic flexural behavior and that one of the four analytical modeling options summarized in the commentary to Section 8.5.3 is utilized. The ATC (2009) values may also be applied to "Fully Restrained Moment Connections". The values in ASCE 41 Table 5-6 plastic rotation angles for "Beams – Flexure" and "Column – Flexure" should not be used as these large values are not confirmed through available experimental data.*
>
> *One important conclusion drawn from the ATC (2009) data and proposed parameters is that for steel beams the pre-capping plastic rotation (θ_c) is relatively small (on the order of 2%) but the post-capping deformation (θ_{pc}) is large, i.e., the decrease in strength after peak strength is slow.*
>
> *Very few experimental data are available for rotation values for plastic hinging in columns. Until such data becomes available, low values for θ_c and θ_{pc} should be used, with the maximum assumed values not being larger than those given for beams in Section 3.2 of ATC (2009).*

8.5.4.2 Steel beam-column joint panel zones

Models shall include the effect of panel zone distortion on overall frame stiffness and on the plastic rotation capacity of fully restrained moment connections. Section 3.3 of *ATC (2009)* presents acceptable modeling rules for panel zone behavior.

> ***Commentary**: Experimental evidence indicates that deterioration in the shear force – shear distortion response of a joint panel zone is small unless shear buckling occurs. The latter mode is unlikely to occur because of code detailing criteria. Thus, it should be acceptable to neglect deterioration in the modeling of joint panel zones, unless there is clear indication that deterioration will occur within the range of deformations expected at maximum considered response levels.*

8.5.4.3 Steel EBF link beams

Plastic deformation values should be based on experimental evidence, particularly if non-standard boundary conditions are employed. When applicable, the values listed in Table 5-6 of *ASCE 41* may be utilized.

8.5.4.4 Steel coupling beams

The plastic deformation values for eccentric braced frame links may be used for steel coupling beams in walls, provided that the full strength of the coupling beam can be developed through anchorage into the wall. If shear wall anchorage is incapable of providing fixity, provide additional rotational springs at the ends to account for relative rotation between the coupling beam and the shear wall.

8.5.4.5 Steel axially loaded components

Modeling should consider post-buckling deterioration, ductile tearing due to localized strain reversal during post-buckling cyclic loading, and fracture at connections.

> ***Commentary:*** *Post buckling modeling and ductile tearing depends strongly on brace section and slenderness ratio. Recent references on these deterioration and failure modes are Jin and El-Tawil (2003), Uriz (2005), Uriz et al (2008), Fell et al. (2006), Fell (2008).*
>
> *Braces in frame configurations and in outriggers depend strongly on the ability of the connections to transfer pre- and post-buckling forces from the brace to horizontal and vertical chord members. Additional strain may be placed on the connection by relative rotations of the chord members at the brace intersections. It is of paramount importance to consider all conceivable failure modes at the brace connection when assigning strength and deformation parameters to the bracing member.*

8.5.4.6 Steel plate shear walls

Modeling shall be adequate to capture the effective story shear strength and stiffness, including the pinching effect caused by tension field reversal, deterioration due to connection failures and possibly due to combined bending and axial load effects in the vertical boundary elements. If cyclic strip models are used, a sufficient number of strips must be used to adequately simulate the column bending moments due to force transfer between the shear wall panel and the vertical boundary elements.

> ***Commentary****: At large story drifts the combined bending and axial load capacity of the vertical boundary elements might deteriorate due to shear racking that causes large localized rotations in these boundary elements. P-Delta might become a critical issue if the shear wall deforms in a shear racking mode that concentrates inelastic deformations in the lower stories. Information on modeling of steel plate shear walls can be found in AISC (2006) and in the many references listed in that publication.*

8.5.4.7 Reinforced concrete beams and columns in bending

Either the values provided in Section 3.4 of *ATC (2009)* or those given in *ASCE 41* (including Supplement 1) may be used. The *ATC (2009)* values are based on the assumption that point hinge models are used to represent inelastic flexural behavior and that one of the four analytical modeling options summarized in the commentary to Section 8.5.3 is utilized. The deformation values given in *ATC (2009)* are for the monotonic backbone curve illustrated in Fig. 8.1 and need to be modified unless modeling option 1 is utilized.

> ***Commentary:*** *The rotation values in Section 3.4 of ATC (2009) are in many cases significantly larger than those listed in ASCE 41. The reasons are (1)*

bond slip at the beam-column interface contributes significantly to plastic rotations, (2) the listed plastic rotations are for the monotonic backbone curve and would have to be modified (by a recommended factor of 0.7) for comparison with the ASCE 41 values, and (3) the listed values are expected values whereas the ASCE 41 values represent a lower percentile value.

The ATC (2009) values have been derived from a database that contains mostly experimental results from column tests. The regression equations have been extrapolated to an axial load of zero in order to be applicable for beams. This process may not be fully justified because beams have unequal longitudinal top and bottom reinforcement and no distributed side face reinforcement, and in most cases have contributions from a slab system. Guidance for modeling slab contributions can be found in NEHRP (2008).

Elastic stiffness used in the analytical model may follow the guidance of Section 3.4 of ATC (2009) or that in *ASCE 41 Supplement 1.*

***Commentary**: ATC (2009) and ASCE-41 give somewhat different values for effective elastic stiffness of concrete members. The effect of the different stiffness assumptions is not believed to be important in the prediction of deformation demands for beams and columns, i.e., either the ASCE 41 or the ATC (2009) recommendations should be adequate.*

8.5.4.8 Reinforced concrete beams and columns in shear

Recommendations for modeling shear strength, stiffness and deformation capacity are provided in *ASCE 41*, including *Supplement No. 1.*

***Commentary**: Beams and columns should be protected from excessive shear deformations through capacity design requirements. But flexural plastic hinging might reduce the shear strength to the extent that inelastic shear deformation possibly will occur. In such cases a shear force – shear deformation model (usually a translational spring) should be inserted in the element.*

8.5.4.9 Reinforced concrete slabs in slab-column frames

Section 4.6 of *ATC (2009)* and *Supplement 1* of *ASCE 41* provide appropriate guidelines for modeling of slab-column frames.

***Commentary**: Rotational springs should be used to model torsional behavior at the column-slab connection. This enables tracking of the "unbalanced" moment transferred from the slab to the column. The issue of transferring moments from the slab to the column through direct shear and eccentricity in shear deserves careful modeling and design detailing.*

8.5.4.10 Reinforced concrete beam-column joints

Explicit modeling of concrete beam-column joints is not required where capacity design principles are employed to preclude joint shear failure. Where modeling is necessary, use the rules provided in *ASCE-41 Supplement 1.*

> ***Commentary:*** *Bond slip of longitudinal reinforcement in the joint region is best represented in the models of the beams and columns framing into the joint. A joint model, if employed, is therefore concerned primarily with the shear force – shear distortion behavior of the joint, which invites the use of a rotational spring inserted at the joint (Lowes and Altoontash (2003)). Modified compression field theory has been shown to work well for conforming joints.*

8.5.4.11 Reinforced concrete shear walls in bending and shear

Either fiber or moment – curvature models based on realistic cyclic material models may be used providing that excessive shear deformation is avoided through capacity design concepts.

> ***Commentary****: Both fiber and moment-curvature models can provide good representations of wall bending behavior over the full height of the wall. Shear behavior is usually decoupled from bending behavior. Coupled models (shear-flexure-axial) do exist but are difficult to implement at this time. Information on modeling of flexural and shear strength and stiffness properties of beam-column models and fiber models are presented in Sections 4.2 and 4.3 of ATC (2009). Most of the models presented in these sections do not address deterioration due to rebar buckling and fracture, which necessitates the specification of strain limits to account approximately for these often critical deterioration modes.*
>
> *In analysis it is often assumed that outside of designated protected regions a shear wall can be modeled with elastic models. The designation of "protected regions" is usually made based on elastic design concepts. However, seismic force demands at the Maximum Considered Earthquake response level in tall and slender walls structures depend very much on inelastic redistribution and higher mode effects, which might lead to large moment and shear force amplifications compared to values estimated from elastic behavior. Therefore, it is necessary to perform a comprehensive post-analysis demand/capacity review of the structure in order to verify that the demands in all protected regions are indeed small enough to justify the assumption of elastic behavior. The results might disclose the need for re-design or re-analysis.*

8.5.4.12 Reinforced concrete coupling beams

Use the modeling recommendations Section 4.4 of *ATC (2009)* or coupling beams that are flexure controlled. For short and stocky coupling beams that are shear controlled use the modeling parameters listed in Table 6-19 of *ASCE 41*.

> ***Commentary****: New provisions for diagonally reinforced coupling beams are included in ACI 318-08 that allow two detailing options, one with transverse reinforcement around the groups of diagonal bars and the other with transverse reinforcement around the entire beam. Test results indicate that the load – displacement response for both detailing options are nearly the same.*
>
> *Consideration should be given to the phenomenon that walls will "grow" on the tension side due to shifting of the neutral axis, which in turn will increase the vertical deflection at the wall-coupling beam interface and therefore will increase the coupling beam rotation demand.*

8.5.4.13 Non-standard components

For components whose design and behavior characteristics are not documented in applicable codes and standards, develop appropriate design criteria and component models from analytical and experimental investigations. In general, experimental verification will be necessary for proposed models for inelastic behavior including deterioration. The modeling guidelines of Sections 8.5.1 to 8.5.3 should be considered in the development of analytical models and experimental validation.

8.5.4.14 Response modification (seismic isolation, damping and energy dissipation) devices

Model properties of response modification devices based on data from laboratory tests representing the severe conditions anticipated in a Maximum Considered Earthquake event. If there is significant variability in properties of these devices, the structure response predictions use alternative models incorporating upper and lower bound properties. If there is a functional limit beyond which the devices cease to operate (e.g., a displacement limit), represent this functional limit in the analytical model. It should be demonstrated either that the consequences of attaining this limit can be tolerated by the structure, or. that this functional limit will not be attained under 1.5 times the mean demand obtained from Maximum Considered Earthquake response analysis.

8.5.4.15 Foundation modeling

Foundation components that have significant flexibility or will experience significant inelastic behavior should be modeled following the same guidelines outlined for components of the superstructure.

When soil foundation structure interaction is accounted for in the model, evaluate the sensitivity of the predicted response to variation in important soil properties including strength and stiffness. Establish likely variability in soil properties in consultation with the geotechnical engineer.

> ***Commentary**: Caution needs to be exercised in designing for, and modeling of, shear and bending in mat foundations. Rigorous analysis will often result in great variations of shear and bending stresses across a mat foundation, whereas it is customary practice to distribute reinforcement uniformly over a large width. This practice might underestimate the importance of local stress distributions close to concentrated loads delivered from core walls. Guidelines are under development by the ACI Task Group on Fixed Foundations*

8.5.4.16 Foundation rocking and uplift

Foundation rocking and uplift should be considered as a deformation controlled mode. The orientation and properties of springs and other elements used to account for these effects should also account for the redistribution of soil stresses and deformations caused by changes in the contact surface between the foundation and the soil and assure transfer of axial and shear forces to the soil. The effect of varying assumptions on soil properties should be evaluated in consultation with the geotechnical engineer.

8.6 Acceptance criteria at the component level

All actions (axial, shear and flexural deformation) should be evaluated either as force-controlled or deformation controlled. Deformation-controlled actions are those in which reliable inelastic deformation capacity is achievable through adequate detailing. Force-controlled actions are those in which inelastic deformation capacity cannot be assured. These actions include, but may not be limited to:

- Axial forces in columns (including columns in gravity frames)
- Compressive strains due to flexure, axial or combined flexure and axial actions in shear walls or piers that do not have adequate confinement
- Compressive strains due to combined axial and flexural actions in shear walls or piers of shear walls where the axial demand exceeds that associated with the balanced point for the cross section
- Shear in reinforced concrete beams, columns, shear walls, and diaphragms without adequate detailing
- Punching shear in slabs and mat foundations without shear reinforcing
- Force transfer from diaphragms and collectors to lateral load resisting units
- Connections that are not designed explicitly for the strength of the connected component (e.g., brace connections in braced frames)

Commentary: *As an alternative to computing the axial demand that produces a balanced condition in a shear wall or pier, it is considered acceptable to classify such elements as deformation controlled when they are provided with adequate confinement reinforcing and the axial demand, P on the element does not exceed 30% of the axial capacity of the section with zero applied moment,* P_o*.*

8.6.1 Force controlled actions

8.6.1.1 – Critical Actions

Critical actions are those force-controlled actions in which the failure mode poses severe consequences to structural stability under gravity and/or lateral loads. Evaluate critical actions for adequacy to satisfy:

$$F_u \leq \phi F_{n,e} \qquad (8\text{-}2)$$

Where

F_u = the demand obtained from statistical evaluation of nonlinear response history analysis. Where the computed demand for an action is not limited by a well defined yielding mechanism, use 1.5 times the mean. Where the computed demand for an action is limited by a well defined yield mechanism, use the mean plus 1.3 times the standard deviation obtained from the individual response history analyses but not less than 1.2 times the mean.

$F_{n,e}$ = nominal strength as computed from applicable material codes but based on expected material properties.

ϕ = resistance (strength reduction) factor obtained from applicable material standards.

***Commentary**: Use of the mean value would imply a significant probability of failure with associated catastrophic consequences. The use of mean + σ is more appropriate however, when fewer than 20 records are used in nonlinear response history analysis, little confidence can be placed in the computed value of the standard deviation or the mean. Past studies, e.g., Zareian and Krawinkler (2007) and Yang and Moehle (2008)) have shown that the true coefficient of variation due to record to record variability is on the order of 0.4. A default value of 0.5 is used for the coefficient of variation to account for the effect of modeling uncertainties and uncertainty in the mean value.*

The use of 1.3 times the σ value obtained from maximum considered response analysis is permit\ted for specific cases, such as beam shear in a moment-resisting frame, where localized or global mechanisms may limit the force value to a rather stable maximum value and inflation to 1.5 times the mean value may be too large.

8.6.1.2 Noncritical Actions

Noncritical actions are those force-controlled actions the failure of which does not result in structural instability or potentially life-threatening damage. Evaluate noncritical actions for adequacy to satisfy:

$$F_u \le F_{n,e} \quad (8\text{-}3)$$

Where

F_u = the mean demand obtained from the suite of analyses,

$F_{n,e}$ = nominal strength as computed from applicable material codes but based on expected material properties.

8.6.2 Deformation controlled actions

If the ultimate deformation capacity (δ_u, see Figure 8.1) is exceeded in any of the response history analyses, the strength associated with this mode of deformation should be assumed as zero and the stability of the structure and the effects on related strength quantities should be evaluated.

***Commentary:** To implement this criterion it is necessary to define the ultimate deformation capacity for each component. This may be done directly (see modeling options 1 to 4 in Section 8.5.3) or indirectly by specifying strain limits in* *cases in which known but unquantifiable severe deterioration modes exist. For instance, the maximum concrete compressive strain in confined concrete might be limited to 0.015 and the rebar tensile strain might be limited to 0.05 in tension and 0.02 in compression in order to limit the occurrence of rebar buckling and fracture. Chapter 6 of ASCE 41-06 and Chapters 3 and 4 of ATC (2009) provide suitable recommendations for rotation and strain limits for reinforced concrete components. For steel components the recommendations for rotation limits given in Chapter 5 of ASCE 41 and Chapter 3 of ATC (2009) may be utilized.*

8.7 Global Acceptance Criteria

Global acceptance criteria include peak transient and residual story drift and loss of story strength.

8.7.1 Story Drift

8.7.1.1 Peak Transient Drift

The mean of the absolute values of the peak transient drift ratio in each story from the suite of analyses should be less than 0.03. The maximum story drift ratio in any analysis anywhere in the structure should not exceed 0.045. Cladding systems, including the cladding itself and its connections to the structure, should be capable of accommodating the mean of the absolute values of the peak transient story drifts in each story.

***Commentary**: The use of a story drift limit of 0.03 has shown to result in good and efficient designs in recent tall building projects. There is general consensus that up to this story drift structures with good detailing will perform well (without significant loss of strength), and that properly attached nonstructural components will not pose a major life safety hazard. The drift limit should be applied to the "total" story drift (caused by story racking and story flexural rotation) because it is intended to protect all components of the structure including the gravity system components that are surrounding shear walls or braced frames and are subjected mostly to a story shear (racking) mode of deformations. A story drift limit of 0.03 also provides P-Delta control in stories with large vertical loads. The residual story drift ratio of 0.01 is intended to protect against excessive post-earthquake deformations that likely will cause condemnation or excessive downtime for the building. This criterion is added to provide enhanced performance for tall buildings.*

When evaluating peak transient drifts, the maximum absolute value of the drift in each story from each of the analyses in the suite should be used to determine the mean drift, rather than by taking the mean of the maximum drift in the positive direction, and the maximum drift in the negative direction separately. This is because the phasing of ground motion is unpredictable and has equal likelihood of producing large positive drift as it does large negative drift.

8.7.1.2 Residual Drift

The average of the absolute values of residual drift ratio in each story from the suite of analyses should be less than 0.01. The maximum residual story drift ratio in any analysis should not exceed 0.015 unless justification is provided and accepted by the Peer Review.

Commentary: *The maximum transient drift of 0,045 has been selected judgmentally based on the authors' understanding of the limits beyond which structural analysis using present day tools loses reliability. Similarly, the limit on maximum residual drift of 0.015 has been selected judgmentally because of a concern that tall buildings with residual drifts in excess of this amount may pose substantial hazards to surrounding construction in the event of strong aftershocks. In each case, these limits are to be evaluated against the maximum response predicted in any of the response histories. It may be acceptable to accept drifts that are larger than this amount, if the Peer Review agrees either that the large predicted response is due to peculiarities in the ground motion characterization, that may not be fully appropriate, or agreement that the structure's response is reliably predicted and acceptable, even given the large predicted drifts.*

8.7.2 Acceptable loss in story strength

In any nonlinear response history analysis, deformation imposed at any story should not result in a loss of total story strength that exceeds 20% of the initial strength.

Commentary*:* *Component deterioration will lead to a loss in lateral and gravity load resistance, even if deterioration occurs only in deformation controlled actions. Since no absolute limit is placed on the deformations that can be tolerated in any one component, it is prudent to check that the loss in story resistance does not become excessive. As a general target, the loss in lateral story resistance at maximum drift should not be more than about 20% of the undeteriorated resistance.*

9 PRESENTATION OF RESULTS

9.1 General

A key element to the successful completion of a performance-based Design and peer review is the documentation of the process and the presentation of the results. Key elements of this documentation include the design criteria, the design of the components of the lateral force resisting elements and primary gravity members, the results of the nonlinear response history analyses, and the documentation of the design in the construction documents (primarily drawings and specifications). Defining the scope and details of the results to be presented to reviewers as completely as possible at the beginning of the peer review will help to align the expectations of the various participants, and lessen the chance of significant re-work by the design team. This alignment of expectations and follow through by the design team and reviewers throughout the various phase of the project will require commitment of all parties.

The scope and detail of each presentation of information developed by the design team for review will be directly related to the phase of the project, moving from the global to the specific as the design advances from concepts to final design. At all steps in the process, highlight all assumptions that are significant to the building response, or which may be outside of widely accepted standard practice or procedures, and/or otherwise controversial for specific review and comment by reviewers. Clearly state assumptions and provide discussion of the potential implications of their implementation.

Present documentation in a way that facilitates the efficient transfer of information to the reviewers. Interpretation of the results and validation of assumptions and design criteria are key elements in an effective presentation of results. More is not necessarily better, since nonlinear dynamic analyses generate so much raw response data. For example, presenting graphical results of key maximum response components with explanations of "what it means" is far more effective than submitting binders (or CD's) full of raw analysis data. In addition, all spreadsheets key to the structural analysis or design should be accompanied by a fully worked out example to explain the spreadsheet operations.

Another item that needs to be discussed and understood is the intended construction phasing. If an early excavation/foundation package is anticipated, this should be discussed to determine how it will impact the design and review process.

9.2 Design Criteria

The Design Criteria is the first, and in many ways most critical document in the process. The Design Criteria is the key element in describing the design intent, primary assumptions, analyses to be performed, acceptance criteria, etc. Once agreed to by all participants, the Design Criteria becomes the rules by which subsequent design and analyses are checked. Complete and clear documentation of this document will help avoid misunderstandings later in the process, and the potential for expensive re-work and delayed progress. Generally, the more detail included in the document, especially as related to material response and acceptance criteria assumptions, the greater the chance for an efficient presentation and review process. Section 4 provides a recommended outline of Design Criteria contents.

9.3 Geotechnical/Seismic Ground Motion Report

The Geotechnical/Ground Motion Report should also be developed and reviewed in the early stages of the project. The geotechnical portion of the report, which provides design parameters for foundation elements, information on groundwater, retaining wall design pressures, etc., should basically be similar to that required for any significant design project. One item that may be included in the report that is beyond the typical scope could be stiffness and nonlinear displacement quantities of supporting soils that can be incorporated into the building analysis model.

A properly documented ground motion report is extremely important to the success of the design process. There is continuing debate over the validity and appropriateness of many significant recent and ongoing developments in the procedures used in the generation of response histories for nonlinear analyses. These issues may be especially contentious for Maximum Considered Earthquake shaking levels in the long period range of interest in tall building design, resulting in the need for extensive description of the assumptions and process followed. Spectral matching, scaling processes, use of recently developed attenuation procedures, near field and directivity effects, hazard disaggregation, etc., all can have significant impact on the suite of response histories to be generated for use in the structural analyses and the way in which they are modified to match the target spectrum. Whenever possible, present the unmodified and modified acceleration, velocity and displacement plots for each of the proposed response histories. This is especially important when spectral matching approaches with large scaling factors are used to generate Maximum Considered Earthquake ground motions.

Refer to Section 5 for further details on the procedures to be followed and recommended contents of the Ground Motion report. Summarize this report in the Design Criteria Document, and included it in complete form as an Appendix to the Design Criteria.

9.4 Preliminary/Conceptual Design

The preliminary/conceptual design package should include a design narrative of the structural system, similar to, but potentially more fully developed than that presented in the Design Criteria Document. Present drawings for both gravity and lateral force resisting systems, including preliminary member sizes, wall thicknesses, etc. Provide proposed detailing approaches for ductile elements of lateral system. Note special force transfers (e.g. at podium and outrigger levels), and approach to design of these elements, including sample design calculations and preliminary detailing concepts. If damping or energy dissipation elements are to be incorporated, describe assumptions used in their initial design. Provide outline specifications for structural sections, highlighting material requirements that are unusual. Provide initial design calculations used to develop member sizes including member stiffness assumptions, period calculations, base shear capacity, etc. If full building model has been developed, present model input and basic results (base shear, overturning moment, story drift plots, etc.).

9.5 Service Level Evaluation

Provide executive summary discussion. Re-state response spectrum for this evaluation. Provide input model with description of elements and modeling assumptions. When response history analysis is used, provide plots of story drifts, moments, shears and axial loads on key elements that vary with height, showing the peak quantities for each ground motion, the acceptable values and the statistical quantity of demand against which it is compared. All demand/capacity ratios that exceed a value of 1.0 should be clearly described. Provide information needed to compare model with design drawings. Provide capacity design calculations for major structural elements. Present base shear results. Provide story drift plots and comparison to design criteria limits. Provide maximum D/C ratios for major structural elements. Discuss any elements that may exceed drift or capacity limits. Note torsional response, if significant. Discuss level of dispersion of major response quantities. Verify that results are consistent with Design Criteria Document.

9.6 Maximum Considered-Level Evaluation.

Provide executive summary discussion. Re-state response spectrum for this evaluation. Provide input model with description of elements and modeling assumptions. Present time history plots for acceleration, velocity and displacement. Provide detailed description of nonlinear element modeling, with clear and complete discussion of assumptions. Provide information needed to compare model with design drawings. Provide capacity design calculations (both force and deformation) for major structural elements. Compare critical element deformation demands to capacity limits. Present base shear and overturning moment results. Provide plots of story drifts, moments, shears and axial loads on key elements that vary with height, showing the peak quantities for each ground motion, the acceptable values and the statistical quantity of demand against which it is compared. Discuss any elements that may exceed drift or capacity limits. If special elements (outriggers, damping or energy dissipation elements, e.g.) are included in the design, provide a separate discussion of the response of these elements. Include evaluation of foundation elements and major force transfer elements/levels, such as the podium and outriggers.

10 PROJECT REVIEW

10.1 General

Because of the complexity of the analyses used to demonstrate building performance (which typically explicitly include nonlinear response effects), most building departments have initiated a requirement for independent peer review when designs are submitted for permit under the alternative means and methods clause. This requirement is included in the *ASCE-7.10* standard as well. The composition of the peer review should typically be jointly determined by the owner/design team and the building department. Additional members of the peer review team may be added as appropriate to fully address the special features of the proposed project that are not evident at initiation of the process. There is no particular recommendation as to whether an individual person or firm, or a team of individuals and firms provides the peer review. However, the peer reviewer or reviewers should jointly possess expertise in geotechnical engineering and seismic hazards, seismic performance of structures as a whole, as well as knowledge of the design and behavior of structures with elements of the type employed and structural design of tall buildings.

The peer review process should initiate as early in the design process as possible. Early agreement and discussion of the fundamental design decisions, assumptions and approaches will help avoid re-work later in the design process that will impact both the project cost and schedule. With projects of the size and complexity of typical tall buildings, there may be differences of opinion on a number of issues during the process that need to be negotiated between the parties. The earlier in the process that these issues can be identified and resolved, the less effect that they will ultimately have on the building cost and design/construction schedule. Early participation by the peer reviewer should also help to establish a good working relationship with the design team.

It should be noted that the existence of peer review on a project does not relieve the engineer of record from any of their design responsibility. However, because of the level of complexity incorporated in tall building design, in many cases it is recognized that review of these elements of the design effectively constitutes the plan review of the seismic system (even though contracts may say that this is not the case). Peer review participation is not intended to replace quality assurance measures ordinarily exercised by the engineer of record. Responsibility for the structural design remains solely with the engineer of record, as does the burden to demonstrate conformance of the structural design to the intent of the design criteria document. The responsibility for conducting plan review resides with the building official.

The scope of peer review comments should begin with broad, general issues, and progressively move toward the more detailed. It is generally not fair to the engineer of record to bring up new general issues at later stages of the design.

Proper documentation of the peer review process is important for incorporation into the project records. It is best to develop a systematic process for establishing, tracking, and resolving comments generated by the peer review. In many cases, this takes on the form of a written spreadsheet that logs all the comments and resolutions, with dates attached. Comments that are discussed and/or any resolutions that are reached during project review meetings or conference calls should be formally written into project review comment spreadsheet.

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

10.2 Reviewer Qualifications

On many projects peer review is provided by a review team, often comprising 3 persons. The first is typically an expert in the generation of site-specific ground motions and accelerograms for use in the nonlinear analyses; geotechnical engineering or geological engineering. The second is often a practicing structural engineer that is felt to have the expertise needed to properly review the proposed structural system, with experience in structural engineering, performance-based earthquake engineering, nonlinear response history analysis, and tall building design. This engineer’s supporting staff often performs detailed reviews of analytical models. The final person on many panels is often a Professor of structural engineering with research experience and expertise in the proposed structural system, and expertise in structural engineering, earthquake engineering research, performance-based earthquake engineering, nonlinear response history analysis, tall building design. There is no requirement that a panel be comprised of 3 members. The number of members may be expanded or contracted as appropriate, however, the reviewer(s) as a whole, should possess expertise in all of the areas noted above.

Selection of reviewers is often a joint effort of the building official and the owner/design team. It is important for the selection process to obtain reviewers that have both the proper background and expertise to perform the peer review, and also the time available to commit to help the process proceed in a timely manner. Reviewers should not bear a conflict of interest with respect to the project and should not be part of the project design team. The reviewers provide their professional opinion to and act under the instructions of the Building Official.

When review is performed by a team, one team member should serve as the review team chair and should be responsible for mediating disputes between the reviewers and the engineer of record, and for expressing the official positions and opinions of the review team. The review team chair should be a structural engineer licensed to practice in the jurisdiction in which the structure is to be constructed.

10.3 Scope

It is important to have a clear definition of the peer review scope. The building official should define the minimum acceptable scope. In most cases, the review is limited to the seismic design, even though design for wind forces and deformations (specifically drift limits for serviceability and occupant comfort) may control the design of many tall buildings. The design of gravity load resisting elements is typically excluded as well, except for evaluation of deformation compatibility issues. Nonstructural elements that can create hazards to life safety are often included to ensure that proper anchorage and/or deformation accommodation has been provided.

Based on the scope of review identified by the building official, the reviewers, either individually or as a team, should develop a written scope of work in their contract to provide engineering services. The scope of services should include review of the following: earthquake hazard determination, ground motion characterizations, seismic design methodology, seismic performance goals, acceptance criteria, mathematical modeling and simulation, seismic design and results, drawings and specifications.

> ***Commentary:*** *At the discretion of the building official, as well as other members of the development team, the scope of review may be expanded to include review of other building aspects, including wind design and critical non-structural elements.*

Early in the design phase, the engineer of record, the building official, and the reviewers should jointly establish the frequency and timing of review milestones, and the degree to which the engineer of record anticipates the design will be developed for each milestone.

Reviewers should provide written comments to the engineer of record and to the building official. The engineer of record should provide written responses to review comments, with multiple rounds of comment/response sometimes needed for key issues. A log should be jointly maintained by the engineer of record and the reviewers, summarizing all comments, responses to comments, and resolutions. At the conclusion of the review, the reviewers should submit a written report to the building official documenting the scope of the review, the comment log, and indicating the reviewers' professional opinion regarding the design's general conformance to the requirements of the design criteria document. The building official may request interim reports from the reviewers at the time of interim permit reviews.

> ***Commentary****: None of the reports or documents generated by the review are Construction Documents. Under no circumstances should letters or other documents from the review be put into the project drawings or reproduced in any other way that makes review documents appear to be part of the Construction Contract Documents. The engineer of record is solely responsible for the Construction Contract Documents. Documents from the reviewers should be retained as part of the building department project files.*

10.4 Dispute Resolution

Given the complexity of tall buildings and the performance-based analyses being performed, it is not uncommon for disagreements to arise between the engineer of record and the reviewers. Generally, these disagreements fall into one of two categories. The first is regarding the level of complexity of analysis/evaluation that has been performed to validate an aspect of the design. In most cases, this should be resolvable with additional analyses, confirming studies, etc. The second case is related to differences of opinion in the interpretation of results, specifically as to whether or not elements of the design criteria have been met. Resolution of such issues may be obtained through sensitivity analyses, bounding analyses or other means.

If cases arise where disputes between the engineer of record and reviewers are not resolved, the building official is required to "break the tie". The building official can do so based on their knowledge of the situation, or in some cases, may retain other experts to review the material and generate a recommended course of action.

For jurisdictions that have a significant number of tall building projects incorporating performance based design procedures, establishment of an advisory board should be considered. Such an advisory board should consist of experts that are widely respected and recognized for their expertise in relevant fields, including but not limited to, structural engineering, performance-based design, nonlinear analysis techniques, and geotechnical engineering. The advisory board members may be elected to serve for a predetermined period of time on a staggered basis. The advisory board should oversee the design review process across multiple projects periodically; assist the Building Official in developing criteria and procedures spanning similar design conditions, and resolve disputes arising under peer review.

10.5 Post-review Revision

Because of the fast track nature of many modern large building projects, it should be expected that significant changes to the design may occur during the final stages of design and/or the construction phase. In this event, the engineer of record should inform the building official, describing the changes to the structural design, detailing, or materials made subsequent to the completion of peer review. At the discretion of the building official, such changes may be subject to additional reviewed by the peer review team and approved by the building official.

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