AN ALTERNATIVE PROCEDURE FOR SEISMIC ANALYSIS AND DESIGN OF TALL BUILDINGS LOCATED IN THE LOS ANGELES REGION

A CONSENSUS DOCUMENT

2014 Edition with 2015 Supplements

AUGUST 14, 2015
AN ALTERNATIVE PROCEDURE FOR SEISMIC ANALYSIS AND DESIGN OF TALL BUILDINGS LOCATED IN THE LOS ANGELES REGION

2014 Edition with 2015 Supplements (Last revised on August 2015)

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CONTENTS

1. INTRODUCTION .................................................................................................................. 5
   1.1. General .......................................................................................................................... 5
   1.2. Design Team Qualifications .......................................................................................... 5
   1.3. Significant Changes from the 2011 Edition .................................................................... 6

2. INTENT, SCOPE, JUSTIFICATION, AND METHODOLOGY .................................................. 7
   2.1. Intent ............................................................................................................................... 7
   2.2. Scope .............................................................................................................................. 7
   2.3. Justification .................................................................................................................... 8
   2.4. Methodology .................................................................................................................. 9
   2.5. Strength and Stiffness Properties .................................................................................. 13

3. ANALYSIS AND DESIGN PROCEDURE ........................................................................... 15
   3.1. General .......................................................................................................................... 15
   3.2. Modeling Requirements ............................................................................................... 17
       3.2.1. Mathematical Model ............................................................................................... 17
       3.2.2. Modeling Subterranean Structural Systems ............................................................ 17
       3.2.3. Backstay Effects ...................................................................................................... 18
       3.2.4. Beam-column Joints ............................................................................................... 20
       3.2.5. Floor Diaphragms .................................................................................................... 20
       3.2.6. Column Bases ......................................................................................................... 21
       3.2.7. Concrete Core Walls .............................................................................................. 21
       3.3.1. Classification of Structural Actions ........................................................................ 23
       3.3.2. Limitations on Nonlinear Behavior ....................................................................... 24
   3.4. Serviceability Evaluation ............................................................................................... 26
       3.4.1. General .................................................................................................................... 26
       3.4.2. Service Level Design Earthquake ......................................................................... 26
       3.4.3. Description of Analysis Procedure ........................................................................ 27
       3.4.4. Evaluation of Effects of Accidental Torsion ........................................................... 28
       3.4.5. Acceptability Criteria ............................................................................................. 28
   3.5. Collapse Prevention Evaluation ..................................................................................... 29
       3.5.1. Ground Motion ....................................................................................................... 29
       3.5.2. Mathematical Model ............................................................................................... 31
       3.5.3. Analysis Procedure ................................................................................................ 34
       3.5.4. Acceptability Criteria ............................................................................................. 37
   3.6. Specific Provisions for Reinforced Concrete Structures ............................................... 44
       3.6.1. Reinforced concrete special moment frames .......................................................... 44
       3.6.2. Quality control for high-strength concrete ............................................................. 45

4. PEER REVIEW REQUIREMENTS ...................................................................................... 47
   4.1. Qualifications and Selection of SPRP members ............................................................. 47
   4.2. Peer Review Scope ........................................................................................................ 48

5. SEISMIC INSTRUMENTATION ......................................................................................... 49
   5.1. Overview ........................................................................................................................ 49

2014 LATBSDC Alternative Analysis and Design Procedure with 2015 Supplements
5.2. Instrumentation Plan and Review ........................................................................................................... 49
5.3. Minimum Number of Channels ............................................................................................................... 49
5.4. Distribution ........................................................................................................................................... 50
5.5. Installation and Maintenance .................................................................................................................. 52
5.6. Documentation ....................................................................................................................................... 52

REFERENCES ............................................................................................................................................... 53
ABOUT THE COUNCIL

The Los Angeles Tall Buildings Structural Design Council was formed in 1988 to provide a forum for the discussion of issues relating to the design of tall buildings. The Council seeks to advance state-of-the-art structural design through interaction with other professional organizations, building departments, and university researchers as well as recognize significant contributions to the structural design of tall buildings. The Council is an affiliate of the Council on Tall Buildings and Urban Habitat (CTBUH).

The Council is a nonprofit California corporation whose members are those individuals who have demonstrated exceptional professional accomplishments in the structural design of tall buildings. The annual meeting of the Council represents a program for engineers, architects, contractors, building Official and students. The annual meeting program includes research reports on areas of emerging importance, case studies of current structural designs, and consensus documents by the membership on contemporary design issues.

The 2014 Alternative Procedure Development Committee:

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The Development Committee expresses its sincere gratitude to Mr. Ronald Hamburger for his constructive criticisms of the various drafts of this document and his valuable suggestions and contributions.
1. Introduction

1.1. General

The intent of the document is to provide a performance-based approach for seismic design and analysis of tall buildings with predictable and safe performance when subjected to earthquake ground motions. These provisions result in more accurate identification of relevant demands on tall buildings. As such, the application of the procedure contained in this document is expected to result in buildings that effectively and reliably resist earthquake forces.

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

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

Notwithstanding these potential advantages, engineers contemplating a building design using this document shall give due consideration to the fact that appropriate implementation of these recommendations requires an in-depth understanding of ground shaking hazards, structural materials behavior and nonlinear dynamic structural response.

1.2. Design Team Qualifications

Appropriate implementation of these procedures requires proficiency in structural and earthquake engineering including knowledge of:

- seismic hazard analysis and selection and scaling of ground motions
- nonlinear dynamic behavior of structures and foundation systems including 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

1.3. Significant Changes from the 2011 Edition

The following is a list of major changes that distinguish this document from 2011 LATBSDC Alternative Analysis and Design Procedure:

- incorporation of updates for consistency with relevant provisions of ASCE 7-10, 2012 International Building Code (IBC) and 2013 California Building Code (CBC);
- revised modeling requirements and acceptability criteria for reinforced concrete walls and coupling beams;
- incorporation of sensitivity analysis requirements to bound the possible ramifications of the backstay effect;
- incorporation of adjusted acceptability criteria for buildings in various risk categories;
- revision of design ground motion criteria to permit use of conditional mean spectrum;
- incorporation of the 2011 LATBSDC 2013 Supplement into the document.


2. INTENT, SCOPE, JUSTIFICATION, AND METHODOLOGY

2.1. Intent

The intent of the document is to provide an alternate, performance-based approach for seismic design and analysis of tall buildings with predictable and safe performance when subjected to earthquake ground motions. These provisions result in more accurate identification of relevant demands on tall buildings. As such, the application of the procedure contained in this document is expected to result in buildings which effectively and reliably resist earthquake forces.

C.2.1. Code provisions are intended to provide a minimum level of safety for engineered buildings. The code prescriptive provisions are intended to provide safe design criteria for all types of buildings, ranging from small one and two story dwellings to the tallest structures. As a result of this broad intended applicability, the provisions contain many requirements that are not specifically applicable to tall buildings and which may result in designs that are less than optimal, both from a cost and safety perspective. Advances in performance-based design methodologies and maturity of capacity design principles now permit a more direct, non-prescriptive, and rational approach to analysis and design of tall buildings. This document relies on these advances to provide a rational approach to seismic design of reliable and effective tall building structures. This Document addresses only non-prescriptive seismic design of tall buildings.

This document is not intended to cover essential facilities unless acceptance criteria are modified accordingly.

2.2. Scope

This document was developed for design of tall buildings although the document could be used to design other building types. For the purpose of this document, tall buildings are defined as those with the height, \( h_n \), greater than 160 feet above average adjacent ground surface.

The height, \( h_n \) is the height of Level \( n \) above the Base. Level \( n \) may be taken as the roof of the structure, excluding mechanical penthouses and other projections above the roof whose mass is small compared with the mass of the roof. The Base is permitted to be taken at the average level of the ground surface adjacent to the structure.
2.3. Justification


C.2.2. Nothing in this document precludes its applicability to shorter buildings. The focus, however, has been intentionally narrowed to tall buildings. The procedure contained in this document requires special knowledge and review procedures typically not appropriate to the design of buildings which lend themselves to prescriptive based procedures.

C.2.3. All codes have traditionally permitted the use of alternative analysis and design methods which can be justified based on well-established principles of mechanics and/or supported by convincing laboratory test results.

Section 104.11 of 2012 IBC reads as follows:

“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 by 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 purpose intended, at least the equivalent of that prescribed in this code in quality, strength, effectiveness, fire resistance, durability and safety.”

Section 12.6 of ASCE 7-10 which is adopted by reference in 2013 CBC states:

"The structural analysis required by Chapter 12 shall consist of one of the types permitted in Table 12.6.1, based on the structure's seismic design category, structural system, dynamic properties, and regularity, or with the approval of the authority having jurisdiction, an alternative generally accepted procedure is permitted to be used. ..."

Furthermore, Section 1.3 of ASCE 7-10 also permits performance-based approaches that use analysis, testing, or a combination thereof, as acceptable alternative means.
2.4. Methodology

The procedure contained in this document is based on capacity design principles followed by a series of performance based design evaluations. First, capacity design principles shall be applied to design the structure to have a suitable ductile yielding mechanism, or mechanisms, under nonlinear lateral deformations. Linear analysis may be used to determine the required strength of the yielding actions.

The adequacy of the design and the attainment of acceptable building performance shall be demonstrated using two earthquake ground motion intensities:

A. Serviceable Behavior When Subjected to Frequent Earthquake Ground Motions. The service level design earthquake ground motions shall be taken as the ground motions having a 50% probability of being exceeded in 30 years (43-year return period). Structural models used in the serviceability evaluation shall incorporate realistic estimates of stiffness and damping considering the anticipated levels of excitation and damage. The purpose of this evaluation is to validate that the building’s structural and nonstructural components retain their general functionality during and after such an event. Repairs, if necessary, are expected to be minor and could be performed without substantially affecting the normal use and functionality of the building. Subjected to this level of earthquake ground motion, the building structure and nonstructural components associated with the building shall remain essentially elastic. This evaluation shall be performed using three-dimensional linear or nonlinear dynamic analyses. Essentially elastic response may be assumed for elements when force demands generally do not exceed provided strength. When demands exceed provided strength, this exceedance shall not be so large as to affect the residual strength or stability of the structure.

B. Low Probability of Collapse when subjected to Extremely Rare Earthquake Ground Motions. The extremely rare earthquake motions shall be taken as the
Risk Targeted Maximum Considered Earthquake (MCE\textsubscript{R}) ground motions as defined by ASCE 7-10 and adopted by 2012 IBC and 2013 CBC. This evaluation shall be performed using three-dimensional nonlinear dynamic response analyses. This level of evaluation is intended to demonstrate a low probability of collapse when the building is subjected to the above-mentioned ground motions. The evaluation of demands includes both structural members of the lateral force resisting system and other structural members. Claddings and their connections to the structure must accommodate MCE\textsubscript{R} displacements without failure. A reduction factor, $\kappa$, is applied to adjust the acceptance criteria for certain actions. This reduction factor is a function of building Risk Category as defined in Table 1.5-1 of ASCE 7-10.

The use of conditional mean spectrum (CMS) approach is permitted as long as a minimum of two suites of 7 pairs of site-specific ground motion time-histories are used for the nonlinear response history analysis. If only two suites are used then one suite shall characterize relatively short period motion, and the other suite shall characterize long period motion. The envelope of the two suites shall address periods ranging from 0.1$T$ to 1.5$T$\textsuperscript{1} seconds to the satisfaction of the project’s Seismic Peer Review Panel (see Section 4).

\textsuperscript{1} $T$ is the calculated fundamental period of the building.
C.2.4. ASCE 7-10 has introduced significant changes to the way Maximum Considered Earthquake (MCE) shaking, and therefore, design shaking, is determined. In earlier editions of ASCE 7, MCE shaking was defined as probabilistically derived ground motion having a 2% chance of exceedance in 50 years (2475 year return period), except in locations where such shaking exceeded 150% of the motion typically associated with the seismic zone 4 under the 1997 UBC. On those sites, which were typically within a few kilometers of a major active fault, such as the San Andreas or San Jacinto faults, MCE shaking was deterministically taken as the lesser of 150% of the median motion calculated for a characteristic maximum magnitude event on that fault or the probabilistic motion. The 150% was intended to approximately represent a mean plus one standard deviation estimate of the ground motion resulting from such an earthquake. Under ASCE 7-10, both definitions of MCE motion have changed. Probabilistic motion is now taken as shaking which will produce a 1% chance of collapse in 50 years, for structures having standard collapse fragility. This risk—targeted Maximum Considered Earthquake (MCE_R) shaking will have somewhat different return periods at every site. Typically, this will approximate 2,000 years, plus or minus a few hundred years. Deterministic MCE_R shaking is defined as the lesser of 84th percentile shaking from a characteristic earthquake on the nearly active fault, determined using one or more appropriate ground motion prediction equations (GMPEs), or the probabilistic shaking. Modern GMPEs tend to produce higher uncertainty than the older attenuation relationships used by geotechnical engineers and seismologists under earlier editions of ASCE-7, so 84th percentile motions are often larger than 150% of the median motion previously used. As a result, more sites tend to be controlled by probabilistic motion than under earlier editions of the standard.

In addition, ASCE 7 has changed the ground motion intensity by modifying the definition of horizontal ground motion from the geometric mean of spectral accelerations for two components to the maximum response of a single lumped mass oscillator regardless of direction. These maximum-direction (MD) ground motions operate under the assumption that the dynamic properties of the structure (e.g., stiffness, strength) are identical in all directions. This assumption may be true for some in-plan symmetric structures, however, the response of most structures is dominated by modes of vibration along specific axes (e.g., longitudinal and transverse axes in a building), and often the dynamic properties (especially stiffness) along those axes are distinct.

Stewart et al (2011) have demonstrated that (a) the use of MD ground motions effectively assumes that the azimuth of maximum ground motion coincides with the directions of principal structural response. Because this coincidence is unlikely, design ground motions have lower probability of occurrence than intended, with significant societal costs; and (b) opposition to the changed ground motion definition contained in ASCE 7-10 was voiced by 11 member organizations that either voted “No” or expressed reservations about the proposal. Hence, disagreement with the new definition of ground motions contained in ASCE 7-10, is widely held among respected academicians and design professionals.

(Continued on the next page)
C.2.4. (continued)

ASCE 7-10 does not specify how seed time histories are to be scaled for three-dimensional analyses except that each pair of selected and scaled ground motions shall be scaled such that in the period range of $0.2T$ to $1.5T$, the average of the square root of the sum of the squares (SRSS) spectra from all horizontal component pairs does not fall below a target spectrum. Under ASCE 7-05, the target spectrum was taken as 130% of that determined with the prescriptive approach of Section 11.4.5 or site-specific ground motions in accordance with Section 11.4.7. In recognition that MD ground motions are not appropriate for selection and scaling of ground motions for use in nonlinear response history analysis, ASCE 7-10 revised the target spectrum to 100% of that determined using either the prescriptive approach of Section 11.4.5 or the site-specific procedure of Section 11.4.7. Design teams should be cautious to select the appropriate scaling technique to the hazard definition used on a project.

If geomean spectra are used, the target SRSS spectrum should be taken as 130% of the risk-targeted MCE geomean spectrum. If MD [maximum direction] spectra are used, the target SRSS spectrum should be taken as 100% of the risk-targeted MCE geomean spectrum.

When sites are within 3 miles (5 km) of the active fault that controls the hazard, each pair of ground motion components shall be rotated to the fault-normal and fault-parallel directions of the causative fault and be scaled so that the average of the fault-normal components is not less than the risk targeted MCE response spectrum for the period range from $0.2T$ to $1.5T$.

It should be noted that ASCE 41-13, on the subject of Seismic Evaluation and Retrofit of Existing Buildings, does give further guidance on ground motions. Although ASCE 41-13 addresses existing buildings, the BSE-2N ground motions of ASCE 41-13 are identical to the MCE_R ground motions of ASCE 7-10. Also, ASCE 41-13 is a newer document that has been produced by the same technical organization and reflects later thinking and consensus. ASCE 41-13 Section 2.4.2.1 states that for sites located within 3 miles (5 km) of an active fault that controls the hazard of the site response spectra, the effect of fault-normal and fault-parallel motions shall be considered. ASCE 41-13 does not specify that the fault-normal component be scaled to the risk targeted maximum direction response spectrum and implies that the fault-normal and fault-parallel ground motions can be calculated by acceptable analytical methods. The time histories should be matched in such a way that the average response spectra of the fault-normal and fault-parallel components are not less than the respective target fault-normal and fault-parallel components.

The service level design earthquake ground motions may be based on geometric mean ground motions and need not consider maximum direction response.

A summary of the basic requirements for each step of analysis is presented in Table 1. More detailed information regarding these steps is contained in the following sections of the document.
Table 1. Summary of Basic Requirements

<table>
<thead>
<tr>
<th>Design / Evaluation Step</th>
<th>Ground Motion Intensity</th>
<th>Type of Analysis</th>
<th>Type of Mathematical Model</th>
<th>Accidental Torsion Considered?</th>
<th>Material Reduction Factors (φ)</th>
<th>Material Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>Nonlinear Behavior Defined / Capacity Design</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>50/30</td>
<td>LDP or NDP</td>
<td>3D</td>
<td>Evaluated</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>MCE&lt;sub&gt;R&lt;/sub&gt;</td>
<td>NDP</td>
<td>3D</td>
<td>Yes, if flagged during Step 2. No, otherwise.</td>
<td>1.0</td>
<td>Expected properties are used throughout</td>
</tr>
</tbody>
</table>

1 probability of exceedance in percent / number of years
2 linear dynamic procedure
3 nonlinear dynamic procedure
4 three-dimensional
5 per ASCE 7-10

2.5. Strength and Stiffness Properties

Structural models shall incorporate realistic estimates of stiffness and strength considering the anticipated level of excitation and damage. Expected material properties shall be utilized throughout as opposed to nominal or specified properties. In lieu of detailed justification, values provided in Tables 2 and 3 may be used for expected material strengths and estimates of expected strength and stiffness of various materials and structural elements.

C.2.5. It has been documented that concrete aggregates from many local sources in southern California result in concrete mixtures that fall significantly short of the ACI 318 concrete modulus of elasticity requirements enumerated in Table 3 (ACI 318-11, Section 8.5.1 or ACI 318-14, Section 19.2.2.1). Therefore, it is strongly recommended that values for the minimum modulus of elasticity for concrete of various strengths used in a project be prominently identified in the General Notes section of the structural drawings and Project Specifications for mix design.
Table 2. Expected Material Strengths

<table>
<thead>
<tr>
<th>Material</th>
<th>Expected Strength</th>
<th>Strength (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Strength for Structural steel</td>
<td>Hot-rolled structural shapes and bars</td>
<td>$1.5F_y$</td>
</tr>
<tr>
<td></td>
<td>ASTM A36/A36M</td>
<td>$1.3F_y$</td>
</tr>
<tr>
<td></td>
<td>ASTM A572/A572M</td>
<td>$1.1F_y$</td>
</tr>
<tr>
<td></td>
<td>All other grades</td>
<td>$1.1F_y$</td>
</tr>
<tr>
<td>Hollow Structural Sections</td>
<td></td>
<td>$1.3F_y$</td>
</tr>
<tr>
<td>Steel Pipe</td>
<td></td>
<td>$1.4F_y$</td>
</tr>
<tr>
<td>Plates</td>
<td></td>
<td>$1.1F_y$</td>
</tr>
<tr>
<td>All other products</td>
<td></td>
<td>$1.1F_y$</td>
</tr>
<tr>
<td>Yield Strength for Reinforcing steel</td>
<td></td>
<td>1.17 times specified $f_y$</td>
</tr>
<tr>
<td>Ultimate Strength for Concrete</td>
<td></td>
<td>1.3 times specified $f_c'$</td>
</tr>
</tbody>
</table>

Table 3. Reinforced Concrete Stiffness Properties

<table>
<thead>
<tr>
<th>Element</th>
<th>Serviceability and Wind</th>
<th>MCE-Level Nonlinear Models</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural Walls</td>
<td>Flexural – 0.75 Ig</td>
<td>Flexural – 1.0 $E_c$ **</td>
</tr>
<tr>
<td></td>
<td>Shear – 1.0 Ag</td>
<td>Shear – 0.5 Ag</td>
</tr>
<tr>
<td>Basement Walls</td>
<td>Flexural – 1.0 Ig</td>
<td>Flexural – 0.8 Ig</td>
</tr>
<tr>
<td></td>
<td>Shear – 1.0 Ag</td>
<td>Shear – 0.5 Ag</td>
</tr>
<tr>
<td>Coupling Beams</td>
<td>Flexural – 0.3 Ig</td>
<td>Flexural – 0.2 Ig</td>
</tr>
<tr>
<td></td>
<td>Shear – 1.0 Ag</td>
<td>Shear – 1.0 Ag</td>
</tr>
<tr>
<td>Diaphragms (in-plane only)</td>
<td>Flexural – 0.5 Ig</td>
<td>Flexural – 0.25 Ig</td>
</tr>
<tr>
<td></td>
<td>Shear – 0.8 Ag</td>
<td>Shear – 0.25 Ag</td>
</tr>
<tr>
<td>Moment Frame Beams</td>
<td>Flexural – 0.7 Ig</td>
<td>Flexural – 0.35 Ig</td>
</tr>
<tr>
<td></td>
<td>Shear – 1.0 Ag</td>
<td>Shear – 1.0 Ag</td>
</tr>
<tr>
<td>Moment Frame Columns</td>
<td>Flexural – 0.9 Ig</td>
<td>Flexural – 0.7 Ig</td>
</tr>
<tr>
<td></td>
<td>Shear – 1.0 Ag</td>
<td>Shear – 1.0 Ag</td>
</tr>
</tbody>
</table>

* Modulus of elasticity is based on the following equations:

\[
E_c = 57000 \sqrt{f_c'} \quad \text{for } f_c' \leq 6000 \text{ psi}
\]

\[
E_c = 40000 \sqrt{f_c'} + 1 \times 10^6 \quad \text{for } f_c' > 6000 \text{ psi} \quad \text{(per ACI 363R-92)}
\]

** Nonlinear fiber elements automatically account for cracking of concrete because the concrete fibers have zero tension stiffness.
3. ANALYSIS AND DESIGN PROCEDURE

3.1. General

Seismic analysis and design of the building shall be performed in three steps with the intent to provide a building with the following characteristics:

(1) A well-defined inelastic behavior where nonlinear actions and members are clearly defined and all other members are designed to be stronger than the demand imposed by elements designed to experience nonlinear behavior (Capacity Design Approach).

(2) The building’s structural and nonstructural systems and components remain serviceable when subjected to frequent earthquakes (50% in 30 years).

(3) The building has a low probability of collapse during an extremely rare event (on the order of 10% or less, given MCE\textsubscript{R} shaking).

A comprehensive and detailed peer review process is an integral part of this design criteria and a Seismic Peer Review Panel (SPRP) shall be established to review and approve the capacity design approach and building performance evaluations. Details of peer review requirements are contained in Section 4.

C.3.1. The procedure contained in this document is an embodiment of the philosophy deeply rooted and implicit in most building codes requiring that buildings be able to:

1. Resist minor levels of earthquake ground motion without damage;
2. Resist moderate levels of earthquake ground motion without structural damage, but possibly experience some nonstructural damage;
3. Resist major levels of earthquake ground motion having an intensity equal to strongest either experienced or forecast for the building site, with a low probability of collapse, but possibly with some structural as well as nonstructural damage.
C.3.1. (continued).

These objectives are achieved by requiring serviceability for ground motions having a 50% probability of being exceeded in 30 years and a stable predictable response without excessive deterioration of structural elements for MCE<sub>R</sub> response.

This document transitioned from a three level design in its 2005 Edition to a two level design in the 2008 Edition which is retained for this 2014 Edition.

The Rationale for Elimination of Explicit Life Safety Evaluation:

The 2013 California Building Code is based on the 2012 International Building Code, which adopts by reference the ASCE 7-10 seismic provisions. ASCE 7-10’s definition of Risk Targeted Maximum Considered Earthquake (MCE<sub>R</sub>) is primarily based on attaining a notional 10% (or lower) probability of collapse given the occurrence of MCE<sub>R</sub> shaking, assuming standard structural fragility.

Since its inception, the International Building Code has been intended to provide a low probability of collapse under MCE shaking. However, the newer code requirements are intended to provide more explicit and quantitative protection against collapse than did earlier codes. In order to retain R coefficients and design procedures familiar to users of the older codes, ASCE 7-10 adopts design-level earthquake shaking for purposes of evaluating strength and deformation that is 2/3 of the intensity of MCE<sub>R</sub> shaking. This 2/3 reduction in the design earthquake is in recognition that the R factors traditionally contained in the older codes incorporated an inherent margin of at least 1.5. That is, buildings designed using these R factors should be able to resist ground shaking at least 150% of the design level without significant risk of collapse.

This document adopts a philosophy that is consistent with the philosophy that underlies the 2013 CBC and 2012 IBC. Buildings must be demonstrated, through appropriate nonlinear analyses and the use of appropriate detailing to have a suitably low probability of collapse under MCE shaking. In addition, a service-level performance check is incorporated into the procedure to reasonably assure that buildings are not subject to excessive damage under the more frequent, low-intensity shaking, likely to be experienced by the building one or more times during its life. Protection of nonstructural components and systems is reasonably assured by requirements that such components and systems be anchored and braced to the building structure in accordance with the prescriptive criteria of the building code.
3.2. Modeling Requirements

3.2.1. Mathematical Model

Three-dimensional mathematical models of the physical structure shall be used that represents the spatial distribution of the mass and stiffness of the structure to an extent that is adequate for the calculation of the significant features of the building’s dynamic response. Structural models shall incorporate realistic estimates of stiffness and damping considering the anticipated levels of excitation and damage. Percentage of critical damping used in linear models (which may be used in serviceability evaluations) shall not exceed 2.5%. Expected material properties shall be used throughout.

C.3.2.1. Three-dimensional mathematical models of the structure are required for all analyses and evaluations. Given the current state of modeling capabilities and available software systems, the representation of the actual three-dimensional behavior of tall buildings no longer needs to rely 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.

The equivalent viscous damping of building at or below the yield level may be significantly smaller than the 5% critical damping assumed by the building code. The reader is referred to the ATC-72 (ATC 2009) for detailed information regarding appropriate values of equivalent viscous damping for structural analysis.

3.2.2. Modeling Subterranean Structural Systems

Detailed soil-foundation-structure interaction analysis procedures are currently available but the analyses too often suggest load paths and behavior that cannot be rationally accepted. The simplified procedure explained in this section may be used to include subterranean levels in the structural models used for dynamic response analyses. Soil springs need not be included in the model but floor slab strength and stiffness characteristics shall be reasonably included.

Motion shall 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.
When soil springs are not included in the model, the mass of the subterranean levels may also be modified or ignored.

(a) Building system  (b) Model used for structural analysis

C.3.2.2. The simplified approach presented here can be readily adopted in structural analysis and design practice. More complicated methods may require substantially more effort and still may not necessarily result in more accurate results as shown by Naeim et al (2010). In the short-term future advances in practical computing software are anticipated which would make more sophisticated and realistic modeling of soil-foundation-structure-interaction more useable in a design office environment.

Since the horizontal soil restraint is ignored in the simplified approach presented here, inclusion of the mass of the subterranean floors may substantially exaggerate the forces induced on the structure. Therefore either a small portion of the mass of the subterranean floors should be included in the analysis or alternatively the mass of the subterranean floors may be ignored.

Special care should be afforded before the simplified approach presented here is used in special circumstances. For example, application of this approach to buildings where expansion joints run all the way down to the foundation is not justified and should be avoided.

3.2.3. Backstay Effects

Where applicable, for collapse prevention evaluation, two sets of analyses shall be conducted to evaluate backstay effects:

1. A model which uses upper-bound (UB) stiffness assumptions for floor
diaphragms at the podium and below.

2. A model which uses lower-bound (LB) stiffness assumptions for floor diaphragms at the podium and below.

Table 4 contains recommendations for UB and LB Stiffness parameters for backstay sensitivity analyses. The sensitivity analyses, where applicable, shall be performed in addition to the analyses performed using stiffness properties provided in Table 3.

Table 4. Stiffness parameters for Upper Bound and Lower Bound Models

<table>
<thead>
<tr>
<th>Stiffness Parameters</th>
<th>UB</th>
<th>LB</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diaphragms at the podium and below</td>
<td>0.5</td>
<td>0.20 to 0.25</td>
</tr>
<tr>
<td>$E_c I_{eff}$</td>
<td>0.5</td>
<td>0.20 to 0.25</td>
</tr>
<tr>
<td>$G_c A$</td>
<td>0.5</td>
<td>0.20 to 0.25</td>
</tr>
</tbody>
</table>

C.3.2.3. Any lower part of a tall building structure that is larger in floor plate, and contains substantially increased seismic-force resistance in comparison to the tower above, can be considered a podium. Backstay effects are the transfer of lateral forces from the seismic-force resisting elements in the tower into additional elements that exist within the podium, typically through one or more floor diaphragms. The lateral force resistance in the podium levels, and force transfer through floor diaphragms at these levels, helps a tall building resist seismic overturning forces.

This component of overturning resistance is referred to as the backstay effect, based on its similarity to the backspan of a cantilever beam. It is also sometimes called “shear reversal” because the shear in the seismic-force resisting elements can change direction within the podium levels. Since the stiffness properties of the elements, particularly diaphragms, are both influential on the seismic design and uncertain, a sensitivity analysis is required. The UB analysis provides an upper-bound estimate of forces in the backstay load path and a lower bound estimate of forces in the foundation below the tower. This case will govern the design forces for the podium floor diaphragms and perimeter walls, and the associated connections. The LB analysis provides a lower-bound estimate of forces in the backstay load path and an upper-bound estimate of forces in the foundation below the tower. This case will govern the design forces for the tower foundation elements.
3.2.4. Beam-column Joints

Modeling of joints in moment-resisting frames shall account for the flexibility 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.

C.3.2.4. Additional guidance as to appropriate stiffness assumptions for concrete and steel framing may be derived from appropriate test data or found in Moehle et al. (2008) and Hamburger et al. (2009), respectively. Additional guidance for concrete frames is provided in Elwood et al. (2007) and Elwood and Eberhard (2009).

3.2.5. Floor Diaphragms

Floor diaphragms shall be included in the mathematical model where necessary using realistic stiffness properties. Regardless of the relative rigidity or flexibility of floor diaphragms, flexibility of diaphragms with significant force transfer (e.g., podium levels and other setback levels) shall be explicitly included in the mathematical model. Explicit modeling of chords and drags may not be necessary; however, if modeled, diaphragm chord and drag forces, if modeled, 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.

C.3.2.5. Explicit modeling of chords and drags may not be necessary. Chords and drags are force-controlled elements and their force demand must be calculated based on capacity design principles (i.e., maximum force that can be delivered to these elements). In the cases where the floor diaphragms are explicitly modeled, out of plane bending stiffness of the diaphragms should be reduced to a negligible value and diaphragm elements should be modeled as membrane elements. Inclusion of out of plane bending stiffness may inappropriately influence the distribution of forces in the vertical lateral load resisting members and systems. Attention should be paid to ensure that the diaphragm membrane elements do not artificially reduce forces that chords and drags should be designed for.
3.2.6. Column Bases

Realistic assumptions shall be used 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.

3.2.7. Concrete Core Walls

3.2.7.1. Modeling of Flexural Behavior

Concrete stress-strain behavior for members modeled using fiber-element sections shall comply with ASCE 41 backbone curves or shall be based on suitable laboratory test data. Approximations fitted to analytical curves defined by Collins and Mitchell (1997), and adjustments made to allow for confinement effects as described by Mander et al. (1988) and Saatcioglu and Razvi (1992) are acceptable (see Figure 3.2.7.1). Since high-strength concrete may have stress-strain relationships that are different from those for regular strength concrete, the high-strength concrete stress-strain relationship utilized shall be consistent with the requirements of Section 3.6.2 of this document.

![Figure 3.2.7.1. Examples of acceptable stress-strain models for concrete](image-url)
3.2.7.2. Main Reinforcing Steel

Reasonable bilinear approximation of steel stress-strain curve is acceptable (see Figure 3.2.7.2).

![Example of an acceptable bilinear approximation of expected reinforcing steel stress strain curve](image)

Figure 3.2.7.2. Example of an acceptable bilinear approximation of expected reinforcing steel stress strain curve

3.2.7.3. Plastic Hinge Length

The effective plastic hinge length shall be used to monitor the compressive strain and ascertain the maximum dimensions of the wall elements in the analytical model. The plastic-hinge length \( l_p \) in the walls for analyses purposes may be calculated from the maximum of the following formulas given by Paulay and Priestley (1992):

\[
    l_p = 0.2l_w + 0.03h_n \quad \text{or} \quad l_p = 0.08h_n + 0.15f_y d_b \quad \text{(ksi units)}
\]

where \( l_w \) is the wall length, \( h_n \) the wall height and \( d_b \) the nominal diameter of rebar. The height of the finite element used to model the plastic hinge shall not exceed the length, \( l_p \), or the story height at the location of the critical section.
3.3. **Capacity Design**

The building design shall be based on capacity design principles and analytical procedures described in this document. The capacity design criteria shall be described in the project-specific seismic design criteria. The structural system for the building shall be clearly demonstrated to have well defined inelastic behavior where nonlinear action is limited to the clearly identified members and regions and all other members are stronger than the elements designed to experience nonlinear behavior.

3.3.1. Classification of Structural Actions

All actions (forces, moments, strains, displacements, or other deformations) are to be evaluated either as force-controlled or deformation-controlled actions. Deformation-controlled actions are those where the behavior is ductile and reliable inelastic deformations can be reached with no substantial strength loss. Force-controlled actions are those where the behavior is more brittle and reliable inelastic deformations cannot be reached. Force-controlled 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 (other than diagonally reinforced coupling beams), columns, shear walls, diaphragms, and foundations
- Punching shear in slabs and mat foundations without shear reinforcing
- Force transfer from diaphragms and collectors to vertical elements of the seismic-force-resisting system
- Connections that are not designed explicitly for the strength of the connected components.
**C.3.3.1.** As used in this section, deformation-controlled actions include flexure and axial tension in elements that have been specifically detailed to accommodate inelastic structural behavior. Such behavior may be presumed if materials and detailing conform, as a minimum, to the following requirements.

- **Structural steel elements:** criteria for the system requirements defined in AISC 341 (2010) Chapters E and F.
- **Composite steel and concrete elements:** the system requirements defined in AISC 341 (2010) Chapters G and H.

For elements that do not comply with the above requirements, substantiating data needs to be presented to demonstrate adequate inelastic response capability. Such data has been successfully produced and presented in the past. One of the main reasons for success of performance-based design has been its ability to introduce new design techniques and solutions. Therefore, the above requirements should not be viewed as prohibiting new and innovative approaches not yet adopted by existing building codes.

As an alternative to computing the axial demand that produces a balanced condition in a shear wall or pier, these elements may be considered as deformation-controlled if these elements are provided with special confined boundary elements in accordance with the Building Code and the axial demand on the element under applicable load combinations does not exceed $0.25 f'_{c} A_g$.

### 3.3.2. Limitations on Nonlinear Behavior

Nonlinear action shall be permitted only in clearly delineated zones. These zones shall be designed and detailed as ductile and protected zones so that the displacements, rotations, and strains imposed by the MCE$_R$ event can be accommodated with enough reserve capacity to avoid collapse.
C.3.3.2 Limiting occurrence of nonlinear behavior to limited and clearly identified areas of the building that are designed to dissipate energy and exhibit significant ductility is the essence of Capacity Design.

Typical zones and actions commonly designated for nonlinear behavior are identified in the following table. This table is not meant to be conclusive. Other zones may be included into the design based on sufficient justification.

**Table C.3.3.2 Zones and actions commonly designated for nonlinear behavior**

<table>
<thead>
<tr>
<th>Structural System</th>
<th>Zones and Actions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Special Moment Resisting Frames (steel, concrete, or composite)</td>
<td>• Flexural yielding of Beam ends (except for transfer girders)</td>
</tr>
<tr>
<td></td>
<td>• Shear in Beam-Column Panel Zones</td>
</tr>
<tr>
<td></td>
<td>• P-M-M* yielding at the base of columns (top of foundation or basement podiums)</td>
</tr>
<tr>
<td>Special Concentric Braced Frames</td>
<td>• Braces (yielding in tension and buckling in compression)</td>
</tr>
<tr>
<td></td>
<td>• P-M-M yielding at the base of columns (top of foundation or basement podiums)</td>
</tr>
<tr>
<td>Eccentric Braced Frames</td>
<td>• Shear Link portion of the beams (shear yielding preferred but combined shear and flexural yielding permitted).</td>
</tr>
<tr>
<td></td>
<td>• P-M-M yielding at the base of columns (top of foundation or basement podiums)</td>
</tr>
<tr>
<td>Unbonded Braced Frames</td>
<td>• Unbonded brace cores (yielding in tension and compression)</td>
</tr>
<tr>
<td></td>
<td>• P-M-M yielding at the base of columns (top of foundation or basement podiums)</td>
</tr>
<tr>
<td>Special Steel-Plate Shear Walls</td>
<td>• Shear yielding of web plates</td>
</tr>
<tr>
<td></td>
<td>• Flexural yielding of Beam ends</td>
</tr>
<tr>
<td>R/C Shear Walls</td>
<td>• P-M-M yielding at the base of the walls (top of foundation or basement podiums)</td>
</tr>
<tr>
<td></td>
<td>• Flexural yielding and/or shear yielding of link beams</td>
</tr>
<tr>
<td>Foundations</td>
<td>• Controlled rocking</td>
</tr>
<tr>
<td></td>
<td>• Controlled settlement</td>
</tr>
</tbody>
</table>

* yielding caused by combined axial force and uniaxial or biaxial flexure
3.4. Serviceability Evaluation

3.4.1. General

The purpose of this evaluation is to demonstrate that the building’s structural systems and nonstructural components and attachments will retain their general functionality during and after such an event. Repairs, if necessary, are expected to be minor and could be performed without substantially affecting the normal use and functionality of the building.

C.3.4.1. The intent of this evaluation is not to require that a structure remain within the idealized elastic behavior range if subjected to a serviceability level of ground motion. Minor post-yield deformations of ductile elements are allowed provided such behavior does not suggest appreciable permanent deformation in the elements, or damage that will require more than minor repair.

In typical cases a linear response spectrum analysis may be utilized, with appropriate stiffness and damping, and with the earthquake demands represented by a linear response spectrum corresponding to the serviceability ground motion. Where dynamic response analysis is used, the selection and scaling of ground motion time series should comply with the requirements of ASCE 7-10 with the serviceability-level response spectrum used instead of the MCE response spectrum, and with the design demand represented by the geometric mean of calculated responses for appropriately selected and scaled time series.

3.4.2. Service Level Design Earthquake

The service level design earthquake shall be taken as an event having a 50% probability of being exceeded in 30 years (43 year return period). The Service Level Design Earthquake is defined in the form of a site-specific, 2.5%-damped, linear, uniform hazard acceleration response spectrum. If nonlinear dynamic response analysis is to be performed for Service Level evaluation, the ground motion time series shall be selected and scaled according to the provisions of ASCE 7-10 with the design demand represented by the geometric mean of calculated responses for appropriately selected and scaled time series.
**3.4.3. Description of Analysis Procedure**

Either linear response spectrum analyses or nonlinear dynamic response analysis may be utilized for serviceability evaluations. The analysis shall account for P-Δ effects. Effects of inherent and accidental torsion are considered in order to establish whether accidental torsion needs to be included in the Collapse Prevention evaluation (see Section 3.5). The structure shall be evaluated for the following load combinations:

(a) **Response Spectrum Analysis**

\[
1.0D + L_{\text{exp}} + 1.0E_x + 0.3E_y \\
1.0D + L_{\text{exp}} + 1.0E_y + 0.3E_x
\]

(b) **Nonlinear Dynamic Response Analysis**

\[
1.0D + L_{\text{exp}} + 1.0E
\]

where \(D\) is the service dead load and \(L_{\text{exp}}\) is the expected service live load. \(L_{\text{exp}}\) may be taken as 25% of the unreduced live load unless otherwise substantiated and shall be included in all gravity calculations and P-Δ analyses.

*C.3.4.3. Building Code response modification factors do not apply (that is, \(R, \Omega_0, \rho, \text{ and } C_d\) are all taken as unity). \(L_{\text{exp}}\) need not be included in the mass calculations.*

**3.4.3.1. Elastic Response Spectrum Analyses**

At least 90 percent of the participating mass of the structure shall be included in the calculation of response for each principal horizontal direction. Modal responses shall be combined using the Complete Quadratic Combination (CQC) method.

The corresponding response parameters, including forces, moments and displacements, shall be denoted as Elastic Response Parameters (ERP) and shall not be reduced.
3.4.3.2. Nonlinear Dynamic Response Analyses

The mathematical model used for serviceability evaluation shall be the same mathematical model utilized for collapse prevention evaluation under MCE ground motions.

3.4.4. Evaluation of Effects of Accidental Torsion

Accidental eccentricities need not be considered for serviceability evaluation. However, regardless of the analysis method used for serviceability evaluation, the torsional amplification factor, $A_x$, as defined in Section 12.8.4.3 of ASCE 7-10 shall be calculated for each floor, $x$. If the value of $A_x$ exceeds 1.50 for any floor, then accidental eccentricity shall be considered during Collapse Prevention evaluations (see Sections 3.5.3.1 and 3.5.3.2 for details).

3.4.5. Acceptability Criteria

Regardless of the analysis method used, story drift shall not exceed 0.5% of story height in any story.

3.4.5.1. Elastic Response Spectrum Analyses

The structure shall be deemed to have satisfied the acceptability criteria if none of the elastic demand to capacity ratios (ratio of ERP to the applicable LRFD limits for steel members or USD limits for concrete members using $\phi = 1.0$) exceed:

a) 1.50 for deformation-controlled actions for Risk Category I and II Buildings (ASCE 7-10 Table 1.5-1); 1.20 for deformation-controlled actions for Risk Category III Buildings; and a factor smaller than 1.20 as determined by the SPRP (see Section 4) for Risk Category IV Buildings.

b) 0.70 for force-controlled actions.

3.4.5.2. Nonlinear Dynamic Response Analyses

A minimum of three pairs of ground motion time series scaled per provisions of Section 16.1.3 of ASCE 7-10 shall be utilized (seven or more pairs are recommended). Ground motion time
series shall be scaled to the 2.5% damped serviceability design spectrum. If less than seven pairs are used the maximum response values shall be used for evaluation, otherwise, the average of the maximum values may be used.

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

C.3.4.5. The acceptability criteria implemented in this section was developed to maintain a balance between the requirements for serviceability and collapse prevention so that the serviceability criterion will not control the design process across the board.

LATBSDC believes the serviceability criterion adopted in the final version of the PEER Guidelines (2010) does not maintain such a balance and causes the serviceability criterion to dominate the design of the structural system in many, if not most, cases. Therefore, the provisions contained in this document are different from those adopted in the final version of PEER Guidelines (2010) although they are consistent with the criterion specified in various drafts of PEER Guidelines and utilized for PEER case studies.

Arbitrarily increasing the strength of deformation-controlled elements may adversely affect the performance of the building during the MCE\textsubscript{R} level ground motions. These elements are designed to act as fuses during a large earthquake. Therefore, careful attention should be paid in providing the necessary strength and sufficient ductility for such members.

3.5. **Collapse Prevention Evaluation**

3.5.1. **Ground Motion**

3.5.1.1. **Design Spectra**

Risk-targeted Maximum Considered Earthquake (MCE\textsubscript{R}) ground motions represented by response spectra and coefficients derived from these spectra shall be determined in accordance with the site-specific procedure of Chapter 21 of ASCE 7-10. The MCE\textsubscript{R} ground motions shall
be taken as that defined in Chapter 21 of ASCE 7-10.

3.5.1.2. Ground Motion Time Series

A suite of seven or more pairs of appropriate ground motion time series shall be used in the analysis. Ground motion time series and their selection shall comply with the requirements of ASCE 7-10. Either amplitude-scaling procedures or spectrum-matching procedures may be used. In addition, where applicable, an appropriate number of the ground motion time series shall include near fault and directivity effects such as velocity pulses producing relatively large spectral ordinates at relatively long periods.

The use of conditional mean spectrum (CMS) approach is permitted as long as Conditional Spectra that capture the building’s response in each significant mode is captured. A minimum of two CMS should be used one to capture the building’s first mode translational response in each direction and the other second mode response. In structures where first or second mode periods in the two directions are widely separated, additional CMS are required. A minimum of 7 pairs of site-specific ground motion time histories are selected and scaled, or matched to each CMS and used for the nonlinear response history analysis. The envelope of the two suites shall address periods ranging from 0.17 to 1.5$T^2$ seconds to the satisfaction of the project’s Seismic Peer Review Panel (see Section 4). For purposes of evaluating acceptability of response, the mean response of each suite of motions should be separately evaluated.

C.3.5.1.2. Larger suites of appropriate ground motion time histories provide a more reliable statistical basis for analysis. Since three pairs of ground motions provide less statistical accuracy, the use of seven or more pairs of ground motions is required. Chapter 16 of ASCE 7-10 and NIST GCR 11-917-15 (2011) contain well-established procedures for selection of time-histories and, therefore, are adopted by reference in this document.

$^2 T$ is the calculated fundamental period of the building.
3.5.2. Mathematical Model

3.5.2.1. General

P-Δ effects shall be included in all nonlinear dynamic response analyses. P-Δ effects that include all the building dead load plus expected live load shall be included explicitly in the nonlinear dynamic response analyses. Expected live load need not be considered in building mass calculations.

In addition to the designated elements and components of the lateral force resisting system, all other elements and components that in combination significantly contribute to or affect the total or local stiffness of the building shall be included in the mathematical model.

Expected material properties shall be used throughout. The stiffness properties of reinforced concrete shall consider the effects of cracking on initial stiffness.

All structural elements for which demands for any of the nonlinear dynamic response analyses are within a range for which significant strength degradation could occur, shall be identified and the corresponding effects appropriately considered in the dynamic analysis.

Strength of elements shall be based on expected values and $\phi = 1.0$ (see Table 2).

C.3.5.2.1 Three-dimensional mathematical models of the structure are required for all analyses and evaluations.

Realistic inclusion of P-Δ effects is crucial for establishing the onset of collapse.

Suggested material strength values considering overstrength are based on ASCE 41 for concrete and reinforcing steel; and AISC Seismic Provisions for structural steel.

Realistic modeling of the interface between the building and foundations is important.

3.5.2.2. Damping

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 shall not exceed 2.5% of critical for the primary modes of response.

**C.3.5.2.2** 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-72 (ATC 2009) provides a discussion and recommendations for modeling viscous damping in analytical models of tall building structures.

### 3.5.2.3. Component Analytical Models

Acceptance criteria may be taken equal to the corresponding Collapse Prevention values for primary elements published in ASCE 41 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 shall not be as rapid as indicated in the ASCE 41 curves. Alternatively, the modeling options presented in ATC (2010) may be employed.

**C.3.5.2.3**

(a) 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.

(b) Section 2.2.5 of ATC (2010) proposes four options for component analytical models. In this commentary two of these options which are considered more appropriate are discussed.

(continued on the next page)
C.3.5.2.3 (continued)

Option 1 – explicit incorporation of cyclic deterioration in analytical model. 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 option is more rational, and potentially more accurate. However, at this time, such modeling options are not commonly available in commercially available computer programs used for analysis and design of buildings.

Option 2 – use of a cyclic envelope curve as a modified backbone curve; cyclic deterioration is not considered explicitly. If the cyclic envelope curve is known (e.g., from a cyclic test that follows a generally accepted loading protocol) then this envelope curve may be used 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 $\delta_u$ in any analysis should be limited to the maximum deformation recorded in the cyclic test. Modeling parameters in ASCE 41 were determined using this option. When using this approximation, the negative tangent stiffness portion of the backbone curve must be included except in cases where no component deforms beyond the point where degradation begins.

Figure C.3.5.2.3 illustrates the two options discussed above.

![Figure C.3.5.2.3](image-url)
3.5.2.4. Response Modification Devices

Response modification devices (such as seismic isolation, damping, and energy dissipation devices) shall be modeled based on data from laboratory tests representing the severe conditions anticipated in Maximum Considered Earthquake shaking. If the properties of these devices vary significantly, the structure response simulations shall use alternative models incorporating upper and lower bound properties. If the devices have a functional limit beyond which the devices cease to operate (for example, a displacement limit), this functional limit must be represented in the analytical model. The consequences of attaining this limit must be demonstrated to be tolerable to the structure, or the functional limit will not be attained under 1.5 times the mean demand obtained from Maximum Considered Earthquake response analysis.

3.5.2.5. Foundation Modeling, Rocking and Uplift

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

3.5.3. Analysis Procedure

Three-dimensional nonlinear dynamic response analyses of the structure shall be performed. The effect of accidental torsion shall be examined as described in Section 3.4.4 of this document. When the ground motion components represent site-specific fault-normal ground motions and fault-parallel ground motions, the components shall be applied to the three-dimensional mathematical analysis model according to the orientation of the fault with respect to the building. When the ground motion components represent random orientations, the components shall be applied to the model at orientation angles that are selected randomly; but individual ground motion pairs need not be applied in multiple orientations.

For each horizontal ground motion pair, the structure shall be evaluated for the following load combination:

\[ 1.0D + L_{exp} + 1.0E \]
3.5.3.1. Accidental Torsion

If serviceability evaluation indicates that accidental torsion must be included (see Section 3.4.4), a pair of ground motion time series that results in above mean demand values on critical actions shall be selected and substantiated. This pair shall be applied once with centers of mass at the original locations and once at locations corresponding to a minimum accidental eccentricity in one or both horizontal directions, or in the direction that amplifies the building’s natural tendency to rotate.

The ratio of maximum demands computed from the model with accidental eccentricity over the maximum demands computed from the model without accidental eccentricity shall be noted for various actions. If this ratio ($\gamma$) exceeds 1.20, the permissible force and deformation limits for corresponding actions shall be divided by the corresponding ($\gamma$) value.

Alternatively, all ground motion time series may be included in the analyses with the minimum eccentricity (in addition to the original analyses) without changing permitted capacities.

3.5.3.2. Sensitivity Analyses

In lieu of accidental torsion analysis of Section 3.5.3.1 or as an additional measure, a program of sensitivity analyses may be utilized by varying material properties and/or configurations at various locations of the building to demonstrate the vitality of the building.

C.3.5.3.2 The implemented procedure flags importance or insignificance of accidental eccentricity issue during the less cumbersome, serviceability evaluation. If during the serviceability evaluation, accidental eccentricities are established to be significant, then the accidental eccentricities must be included in collapse prevention evaluations. Even then, a set of sensitivity analyses may be performed in lieu of considering the traditional notion of accidental eccentricities.

3.5.3.3. Multiple Towers on a Common Podium or Basement

Where multiple towers on a common podium or base create a situation in which the number of occupants at or below the podium or ground level may exceed 5,000 persons, then:
1. The $k$ factor as specified in Section 3.5.4.1.1 shall also be applied to all force-controlled actions including those of the podium diaphragm and below, including the foundations placed under the Risk Category III portion of the structure, without any exceptions. The same $k$ factor should be applied to all deformation-controlled and force-controlled elements of the tower passing through the Risk Category III portion of the project.

2. The same $k$ factor shall be also applied as specified in Section 3.5.4.1.2 to all deformation-controlled actions of the first level of each tower immediately above the common podium. As this level is most likely a location of formation of plastic hinges, special ductile detailing and confinement shall also be provided.
3.5.4. Acceptability Criteria

3.5.4.1 Acceptance Criteria at the Component Level

Actions in all lateral load resisting elements must be categorized as either force-controlled or deformation-controlled and if classified as force-controlled, as either critical, or noncritical actions. Table 5 shows a representative and acceptable classification of such actions.

<table>
<thead>
<tr>
<th>Component</th>
<th>Seismic Action</th>
<th>Classification</th>
<th>Criticality</th>
</tr>
</thead>
<tbody>
<tr>
<td>Below grade perimeter walls</td>
<td>Flexure</td>
<td>Force Controlled</td>
<td>Non-Critical</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Force Controlled</td>
<td>Critical</td>
</tr>
<tr>
<td>Parking ramp walls</td>
<td>Flexure</td>
<td>Deformation Controlled</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Force Controlled</td>
<td>Critical</td>
</tr>
<tr>
<td>Podium walls</td>
<td>Flexure</td>
<td>Deformation Controlled</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Force Controlled</td>
<td>Critical</td>
</tr>
<tr>
<td>Tower core walls (over their entire height)</td>
<td>Flexure</td>
<td>Deformation Controlled</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Force Controlled</td>
<td>Critical</td>
</tr>
<tr>
<td>Core wall coupling beams</td>
<td>Shear / Flexure</td>
<td>Deformation Controlled</td>
<td>N/A</td>
</tr>
<tr>
<td>Floor slabs</td>
<td>Out of plane flexure around supports</td>
<td>Deformation Controlled</td>
<td>N/A</td>
</tr>
<tr>
<td>Diaphragms with major shear transfer</td>
<td>Flexure</td>
<td>Force Controlled</td>
<td>Non-Critical</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Force Controlled</td>
<td>Critical</td>
</tr>
<tr>
<td>Gravity columns</td>
<td>Axial</td>
<td>Force Controlled</td>
<td>Critical</td>
</tr>
<tr>
<td>Foundations</td>
<td>Flexure</td>
<td>Force Controlled</td>
<td>Non-Critical</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Force Controlled</td>
<td>Critical</td>
</tr>
</tbody>
</table>
3.5.4.1.1 Force-Controlled Actions

(a) Critical Actions

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

\[ F_{uc} \leq \kappa_i \phi F_{n,e} \]

where

- \( F_{uc} = 1.5 \) times the mean value of demand.
- \( F_{n,e} = \) nominal strength as computed from applicable material codes but based on expected material properties.
- \( \phi = 1.0. \)
- \( \kappa_i = \) Risk reduction factor given in Table 6.

EXCEPTION: For buildings located in the Los Angeles region if the serviceability acceptance criteria are satisfied per requirements of Section 3.4.5.1, then \( \kappa_i \) may be taken as 1.0. This exception does not apply to multiple towers on a common podium or base structure described in Section 3.5.3.3.

C.3.5.4.1.1(a) Use of the mean value would imply a significant probability of failure with associated consequences. The use of mean plus one standard deviation is more appropriate. However, when fewer than 20 ground motion pairs are used in nonlinear dynamic response analysis, little confidence can be placed in the computed value of the standard deviation or the mean. A factor of 1.5 is utilized to represent a simple yet reasonable means to reduce probability of failure caused by these actions.

When the force that can be delivered to a component is less than 1.5 times the mean value of the demand then the maximum force that can actually be delivered may be utilized.

In regions of high seismicity such as Los Angeles the serviceability criteria generally controls the design of deformation-controlled actions. Therefore, if for the serviceability evaluation, higher design forces are used for deformation-controlled actions as required per Section 3.4.5.1, this would automatically result in higher strength for force-controlled actions via the capacity design requirements enforced at the collapse prevention evaluation and therefore the use of risk reduction factor for force-controlled actions will not be necessary.
Non-Critical Actions

Noncritical actions are those force-controlled actions for which failure does not result in structural instability or potentially life-threatening damage such as diaphragm shear and axial forces in diaphragm chords and drag members as well as foundation forces. Force-controlled noncritical actions shall satisfy:

\[ F_u \leq \kappa_i \phi F_{n,e} \]

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.
- \( \phi = 1.0 \).
- \( \kappa_i \) = Risk reduction factor given in Table 6.

EXCEPTION: For buildings located in the Los Angeles region if the serviceability acceptance criteria are satisfied per requirements of Section 3.4.5.1, then \( \kappa_i \) may be taken as 1.0.

**C.3.5.4.1.1(b)** Since such failures do not result in structural instability or potentially life-threatening damage, use of mean demand values are justified. Please note that degradation and loss of story strength are limited by other provisions contained in this document. In regions of high seismicity such as Los Angeles the serviceability criteria generally controls the design of deformation-controlled actions. Therefore, if for the serviceability evaluation, higher design forces are used for deformation-controlled actions as required per Section 3.4.5.1, this would automatically result in higher strength for force-controlled actions via the capacity design requirements enforced at the collapse prevention evaluation and therefore the use of risk reduction factor for force-controlled actions will not be necessary.
3.5.4.1.2. **Deformation-Controlled Actions**

The demand values (member total deformations) shall be permitted to be taken respectively as the average of the values determined from the seven or more pairs of records used in the analyses. Collector elements shall be provided and must be capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces. Every structural component not included in the seismic force-resisting system shall be able to resist the gravity load effects, seismic forces, and seismic deformation demands identified in this section.

Acceptance criterion may be assumed to be equal to \( \kappa_i \) times the corresponding Primary Collapse Prevention values published in ASCE 41 for nonlinear response procedures.

\[
\kappa_i = \text{Risk Coefficient given in Table 6}
\]

**Exception:** Larger values may be used only if substantiated by appropriate laboratory tests. If ASCE 41 Primary Collapse Prevention acceptance criterion multiplied by the corresponding \( \kappa_i \) factor are exceeded, strength degradation, stiffness degradation and hysteretic pinching shall be considered and base shear capacity of the structure shall not fall below 90% of the base shear capacity at deformations corresponding to the ASCE 41 Primary Collapse Prevention limits.

<table>
<thead>
<tr>
<th>Risk Category from ASCE 7-10 Table 1.5-1</th>
<th>Risk Reduction factor, ( \kappa_i )</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>1.00</td>
</tr>
<tr>
<td>II</td>
<td>1.00</td>
</tr>
<tr>
<td>III</td>
<td>0.80</td>
</tr>
<tr>
<td>IV</td>
<td>Value to be established by SPRP (see Section 4)</td>
</tr>
</tbody>
</table>
3.5.4.1.2. For Risk Category III, the risk reduction factor of 0.80 is the reciprocal of the 1.25 Importance Factor assigned to this category by ASCE 7.

Primary Collapse Prevention limits for nonlinear response procedures are selected so that significant degradation would not occur prior to reaching them. Therefore, modeling of degradation is not necessary if deformations are kept below these limits. If, however, the relevant ASCE 41-06 (with Supplement 1) tabulated Primary Collapse limits are exceeded, the mathematical model must explicitly contain various material degradations and pinching effects and hysteretic models.

Use of seven or more ground motion pairs is required because it provides a more reliable statistical basis for the demand values.

Proper performance of collector elements is essential for transferring and delivering the seismic forces to resisting elements. Therefore, proper design and proportioning of these elements is vital for the successful performance of the building.

All structural elements, whether or not their strength is considered in determining the lateral strength of the building (i.e., whether or not the structural elements are designated as part of the seismic-force-resisting system), shall be designed and detailed to accommodate the seismic deformations imposed. Components not included in the seismic force resisting system may be deemed acceptable if their deformation does not exceed the corresponding Secondary Collapse Prevention values published in ASCE 41-06 (with Supplement 1) for nonlinear response procedures multiplied by the reduction factor, $\kappa_i$.

3.5.4.2. Global Acceptance Criteria

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

3.5.4.2.1. Peak Transient Drift

In each story, the mean of the absolute values of the peak transient drift ratios from the suite of analyses shall not exceed $0.03\kappa_i$. In each story, the absolute value of the maximum story drift ratio from the suite of analyses shall not exceed $0.045\kappa_i$.

Drifts shall be assessed within the plane of the seismic-force-resisting element or gravity-framing element being evaluated. For structural systems without primary planes, the principal axes shall be determined for the overall structural system or an alternate assessment method. Cladding systems, including the cladding and cladding connections to the structure, shall be capable of
accommodating the mean of the absolute values of the peak transient story drifts in each story.

**C.3.5.4.2.1.** The use of a story drift limit of 0.03 $\kappa_i$ has resulted in efficient designs that have been judged effective by review panels in recent tall building projects. There is general consensus that, up to this story drift, structures with proper yielding mechanisms and 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 $\kappa_i$ also provides P-Δ control in stories with large vertical loads.

The 0.045 $\kappa_i$ interstory drift limit is a global check and is expected to control only a small number of flexible lateral systems and should not come into play in design of shear wall and conventionally braced frame systems.

Exception to the 0.045 $\kappa_i$ limit may be justified for cases where exceedance is very limited or local in nature or when a new structural system with larger drift capacity is introduced. In such cases the engineer of record must clearly state and substantiate the justification for exceeding this limit.

### 3.5.4.2.2. Residual Drift

In each story, the mean of the absolute values of residual drift ratios from the suite of analyses shall not exceed 0.01 $\kappa_i$. In each story, the maximum residual story drift ratio in any analysis shall not exceed 0.015 $\kappa_i$ unless proper justification is provided.

**C.3.5.4.2.2.** The residual story drift ratio of 0.01 $\kappa_i$ is intended to protect against excessive post-earthquake deformations that likely will cause condemnation or excessive downtime for a building. This criterion is added to provide enhanced performance for tall buildings. The limits on residual drifts also are based on concern that tall buildings with large residual drifts may pose substantial hazards to surrounding construction in the event of strong aftershocks. Repair or demolition of tall buildings with large residual drifts also may pose community risks. In each case, these limits are to be evaluated against the maximum responses calculated in any of the response histories. Larger residual drifts may be acceptable if the large residual is due to peculiarities in the ground motion characterization, that may not be fully appropriate, or it can be demonstrated that the response is reliably simulated and acceptable, even given the large residual drifts.
3.5.4.2.3. **Loss in Story Strength**

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

C.3.5.4.2.3. 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.

A simple method of maintaining this requirement is to make sure that each component contributing to story strength retains at least 80% of its initial strength.
3.6. Specific Provisions for Reinforced Concrete Structures

3.6.1. Reinforced concrete special moment frames

The moment resisting frame elements shall conform to the requirements of Chapter 21 of the ACI 318-11, with the modification noted below. Flexural Members (ACI 318-08 §21.5)

(a) In regions where postyield rotations are expected, the member shall be detailed in the vertical direction as required by ACI 318-11 Eq. (21-5).

C.3.6.1(a). The added requirement is intended to ensure adequate beam confinement in yielding regions of the moment frame beams. In tall building design, high-strength concrete is usually used for columns and ordinary strength concrete used for beams (i.e., $f_c$ of 3,000 to 5,000 psi). Use of ACI 318-11 Eq. (21-5) which is intended for columns by ACI, to ductile beams as well, is intended to ensure that enough confinement pressure exists at the plastic hinge zones to allow large rotations and prevent buckling of reinforcement under MCE level motions.

(b) Column axial load under governing load combinations (average of the values from the seven or more ground motion pairs per Section 3.5.1.2) shall not exceed $0.40f'_cA_g$.

C.3.6.1(b). Tests have shown that column deformation capacity reduces as axial load increases. The intention for placing the $0.40f'_cA_g$ limit is to keep the level of axial forces below $1.1P_{bal}$, where $P_{bal}$ is the axial load corresponding to balanced failure conditions considering expected materials strengths.

For additional information, consult the following references:


3.6.2. Quality control for high-strength concrete

The following revisions/additions to ACI 318-11 provisions are included to address issues specific to the use of high strength concrete. High-strength concrete for purposes of this document is defined as concrete with $f_c'$ equal to or greater than 6,000 psi.

3.6.2.1. Intermediate and Specified Strengths

The following concrete compressive strength limits as indicated in Table 7 shall be attained. In addition, the strain attained at the point of maximum strength shall not be less than 0.002 and the strain attained past the point of maximum strength at stress level equal to half of the maximum strength value shall not be less than 0.004 (see Figure 3.6.2.1).

Table 7. Intermediate and final strength values for high-strength concrete

<table>
<thead>
<tr>
<th>Specified Strength (psi)</th>
<th>Intermediate and final strength values (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$6,000 \leq f_c' &lt; 8000$</td>
<td>• 6,000 at 28 days</td>
</tr>
<tr>
<td></td>
<td>• $1.3 f_c'$ at 90 days</td>
</tr>
<tr>
<td>$8,000 \leq f_c' \leq 12,000$</td>
<td>• 6,000 at 28 days</td>
</tr>
<tr>
<td></td>
<td>• $0.75 (1.3 f_c')$ at 90 days</td>
</tr>
<tr>
<td></td>
<td>• $1.3 f_c'$ at 365 days</td>
</tr>
<tr>
<td>$f_c' &gt; 12,000$</td>
<td>• $0.50 (1.3 f_c')$ at 28 days</td>
</tr>
<tr>
<td></td>
<td>• $0.75 (1.3 f_c')$ at 90 days</td>
</tr>
<tr>
<td></td>
<td>• $1.3 f_c'$ at 365 days</td>
</tr>
</tbody>
</table>

Figure 3.6.2.1. Strain capacity requirements
3.6.2.2. Documentation of Concrete Proportions

Documentation that proposed concrete proportions will produce strength equal to or greater than the required strength $f'_{cr}$ (see ACI 318-11 §5.3.2) shall consist of strength test records as specified below. The following test records shall be reported at the ages of 28, 90 and 365 days and any other control days as deemed necessary.

- The number of tests in accordance with ACI 318-11 §5.3.3.1 shall be 10 or more.
- The test cylinders shall be from concrete batched in a manner consistent with that intended for the project and discharged from a delivery truck or system planned for the project.
- A minimum of 10 cubic yards of concrete shall be batched for the testing.
- A report shall be submitted with the following information:
  
  a. Stress-strain diagrams for the mixture.
  
  b. Modulus of elasticity in accordance with ASTM C 469, and splitting tensile strength in accordance with ASTM C 469.
  
  c. Length change in accordance with ASTM C 157.
  
  d. Load creep testing in accordance with ASTM C 512.

C.3.6.4.2. The aggregates commonly used in concrete mixes in southern California are composed of rocks, which are softer than those in use in eastern United States. Therefore, southern California specific requirements are warranted to address the issues related to utilization of high-strength concrete in this region. The test results available for high-strength concrete in southern California are generally limited to $f'_{c} \leq 12000$ psi at 365 days. Therefore, caution is advised when using concrete with $f'_{c}$ values greater than 12000 psi.
4. PEER REVIEW REQUIREMENTS

For each project, a Seismic Peer Review Panel (SPRP) shall be convened. The SPRP is to provide an independent, objective, technical review of those aspects of the structural design of the building that relate to seismic performance, according to the requirements and guidelines described in this document, and to advise the Building Official whether the design generally conforms to the intent of this document and other requirements set forth by the Building Official.

The SPRP participation is not intended to replace quality assurance measures ordinarily exercised by the Engineer of Record (EOR) in the structural design of a building. Responsibility for the structural design remains solely with the EOR, and the burden to demonstrate conformance of the structural design to the intent of this document and other requirements set forth by the Building Official resides with the EOR. The responsibility for conducting Structural Plan Review resides with the Building Official and any Plan Review consultants.

4.1. Qualifications and Selection of SPRP members

Except when determined otherwise by the Building Official, the SPRP shall include a minimum of three members with recognized expertise in relevant fields, such as structural engineering, earthquake engineering research, performance-based earthquake engineering, nonlinear dynamic response analysis, tall building design, earthquake ground motion, geotechnical engineering, geological engineering, and other such areas of knowledge and experience relevant to the issues the project poses. The SPRP members shall be selected by the Building Official based on their qualifications applicable to the Seismic Peer Review of the project. The Building Official may request the opinion of the Project Sponsor and EOR on proposed SPRP members, with the Building Official making the final decision on the SPRP membership. SPRP members shall bear no conflict of interest with respect to the project and shall not be part of the design team for the project. The SPRP provides their professional opinion to and acts under the instructions of the Building Official.
4.2. Peer Review Scope

The general scope of services for the SPRP shall be indicated by the Building Official. The SPRP, either individually or as a team, shall include a written scope of work in their contract to provide engineering services. The scope of services shall 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.

The SPRP shall be convened as early in the structural design phase as practicable to afford the SPRP opportunity to evaluate fundamental design decisions that could disrupt design development if addressed later in the design phase. Early in the design phase, the EOR, Building Official, and the SPRP shall jointly establish the frequency and timing of SPRP review milestones, and the degree to which the EOR anticipates the design will be developed for each milestone. The SPRP shall provide written comments to the EOR and to the Building Official, and the EOR shall prepare written responses thereto. The SPRP shall maintain a log that summarizes SPRP comments, EOR responses to comments, and resolution of comments. The SPRP shall make the log available to the EOR and to the Building Official as requested. At the conclusion of the review the SPRP shall submit to the Building Official a written report that references the scope of the review, includes the comment log, and indicates the professional opinions of the SPRP regarding the design’s general conformance to the requirements and guidelines in this document. The Building Official may request interim reports from the SPRP at the time of interim permit reviews.

C.4. Formation of an advisory board appointed by the Building Official is strongly recommended. This advisory board shall consist of experts who 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 shall 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.
5. SEISMIC INSTRUMENTATION

Buildings analyzed and designed according to the provisions of this document shall be furnished with seismic instrumentation according to the provisions of this section.

5.1. Overview

The primary objective of structural monitoring is to improve safety and reliability of building systems by providing data to improve computer modeling and enable damage detection for post-event condition assessment. Given the spectrum of structural systems used and response quantities of interest (acceleration, displacement, strain, rotation, pressure), the goal of these provisions is to provide practical and flexible requirements for instrumentation to facilitate achieving these broad objectives. The instrumentation used on a given building shall be selected to provide the most useful data for post-event condition assessment.

The recent advances in real-time structural health monitoring and near real-time damage detection may be extremely useful in rapid evaluation of status of the building after an event and deciding whether the building is fit for continued occupancy or not (Naeim 2011).

5.2. Instrumentation Plan and Review

An instrumentation plan shall be prepared by the EOR and submitted to SPRP and Building Official for review and approval. SPRP Approved instrumentation plans shall be marked accordingly on the structural drawings. If the building is intended to be included in the inventory of buildings monitored by the California Geologic Survey (CGS) then the recorders and accelerometers must be of a type approved by CGS.

5.3. Minimum Number of Channels

The building shall be provided with minimum instrumentation as specified in the Table 8. The minimum number of required channels maybe increased at the discretion of SPRP and Building Official. Please note that for reliable real-time structural health monitoring and performance evaluations a substantially larger number of channels may be necessary (Naeim 2011).
Each channel corresponds to a single response quantity of interest (e.g., unidirectional floor acceleration, interstory displacement, etc.).

**Table 8. Minimum Number of Channels of Instrumentation**

<table>
<thead>
<tr>
<th>Number of Stories Above Ground</th>
<th>Minimum Number of Channels</th>
</tr>
</thead>
<tbody>
<tr>
<td>6-10</td>
<td>12</td>
</tr>
<tr>
<td>11 – 20</td>
<td>15</td>
</tr>
<tr>
<td>21 – 30</td>
<td>21</td>
</tr>
<tr>
<td>31 – 50</td>
<td>24</td>
</tr>
<tr>
<td>&gt; 50</td>
<td>30</td>
</tr>
</tbody>
</table>

**C.5.3.** For example, a 34-story building shall have at least 24 sensors. Three horizontal sensors would be located at the roof level and six other levels, plus two vertical sensors at the base, and one placed either to measure special conditions at the roof, or at the base, near a third wall to get rocking in a second direction. In general, the seven levels would be chosen where there are changes in stiffness or mass or offsets in the structural system, if any, otherwise they would be evenly distributed over the height.

**5.4. Distribution**

The distribution or layout of the proposed instrumentation shall be logically designed to monitor the most meaningful quantities.

The sensors shall be located at key measurement locations in the building as appropriate for the measurement objectives and sensor types. The sensors shall be connected by dedicated cabling to one or more central recorders, interconnected for common time and triggering, located in an accessible, protected location with provision for communication.
C.5.4. Strong motion instrumentation should be located strategically in a building in order to learn as much as possible about the response of the building during an earthquake and to confirm/verify design and analysis assumptions.

1. It is important to measure the horizontal and torsional motion on each of a series of floors, from the base to the roof. This requires (at least) three uniaxial horizontal accelerometers on each chosen floor. These should be located near the perimeter of the building along walls on opposing sides of the building (as distant as practical from the core) to get the best torsional signal. The sensors placed along the walls should be at the same relative position (e.g., at mid length). They should be oriented with their sensing directions parallel to the walls. A third accelerometer should be placed near the center of the floor, oriented perpendicular to the other two, to measure horizontal motion in that direction.

2. Another goal is to measure rocking at the base of the building, especially for a stiff building founded on soft soils, to determine any rocking contribution to the drift. At least two vertical accelerometers are needed, placed near walls on the opposing sides of the building. To measure rocking in both directions, a third is needed near one of the other walls. In general, the upper floors do not need vertical accelerometers.

3. In general, for easy interpretation and analysis of the recorded data, sensors on different floors should be stacked vertically if possible, that is, placed at the same relative position on each floor, so that the same location in the response is measured.

4. If there are special features near the roof, such as mechanical equipment in the penthouse or architectural features with mass, it may be important to place additional sensors there.

5. It is often effective to install the sensors in the interstitial space above the false ceiling, if present. This keeps the sensors out of the way of the occupants and the normal building activities, reducing likelihood of damage to the sensors. Thus, the sensors planned to measure the motion of the 8th floor would actually be located on the underside, above the ceiling on the 7th floor, for example.

6. The central recorder should be located in a utility or electrical room with AC available, on one of the lower floors of the building, for convenience. Generally a communication line (phone line or Internet) should be provided at the recorder location.

7. Cabling from the accelerometers to the recorder should be continuous runs (i.e., no splices). A pathway will need to be established for the vertical run from the sensors on the upper floors to the recorder location. Depending on local ordinances and fire codes, plenum rated cable may be required.
5.5. **Installation and Maintenance**

The building owner shall install and maintain the instrumentation system and coordinate dissemination of data as necessary with the Building Official.

5.6. **Documentation**

The sensor locations shall be well documented for reference during analysis of the motions after an earthquake is recorded. Strong shaking is infrequent in a building, and it is possible that by the time an earthquake occurs the activities in a building have resulted in certain sensors being moved for construction work and not returned with the same orientation or location. Digital photos shall be taken to document location and orientation of the installed sensors at initial installation and whenever changes are made. A sensor layout showing the sensor locations and key structural elements on plan and typical sections shall be prepared. A tag shall be attached at each sensor location to underscore its importance.

C.5.6. A sensor layout will facilitate rapid visual interpretation of the recorded data. It is valuable to archive design plans, especially structural plans, to allow thorough analysis of the data and finite-element modeling of the building when earthquake motion has been recorded. The tag attached at each sensor location to underscore its importance can read, for example, “Seismic sensor - Do not remove without notifying Building Official.” The documentation is particularly important to be maintained since after an earthquake, depending on the level of shaking, it may not be possible to access certain areas in the building until building officials have been able to schedule a visit. With good documentation, analysis of the recorded data and assessment of the structural response can occur without accessing the building.
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Los Angeles Tall Buildings Structural Design Council


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ABOUT THE COUNCIL

The Los Angeles Tall Buildings Structural Design Council was formed in 1988 to provide a forum for the discussion of issues relating to the design of tall buildings. The Council seeks to advance state-of-the-art structural design through interaction with other professional organizations, building departments, and university researchers as well as recognize significant contributions to the structural design of tall buildings. The Council is an affiliate of the Council on Tall Buildings and Urban Habitat (CTBUH).

The Council is a nonprofit California corporation whose members are those individuals who have demonstrated exceptional professional accomplishments in the structural design of tall buildings. The annual meeting of the Council represents a program for engineers, architects, contractors, building officials and students. The annual meeting program includes research reports on areas of emerging importance, case studies of current structural designs, and consensus documents by the membership on contemporary design issues.

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