Los Angeles Tall Buildings Structural Design Council

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

A CONSENSUS DOCUMENT

2008 EDITION

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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 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 Council develops and sponsors an annual issue of the journal *Structural Design of Tall and Special Buildings*.

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1. Introduction

1.1. General

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.

1.2. Changes from the 2005 Edition

The current edition of this document (2008 LATBSDC) contains significant and numerous changes from the previous edition (2005 LATBSDC). The publication of the 2005 Edition of this document initiated a flurry of activities in development of guidelines and methodologies with the aim of improving the analysis, design, and construction of tall buildings in seismic regions. The San Francisco Department of Building Inspection (SFDBI) was the first entity that followed suit and their first drafts of the document which eventually became AB-083 and was published by the Structural Engineers Association of Northern California (SEAONC) and adopted by SFDBI closely resembled the 2005 LATBSDC document. During its development, however, AB-083 took its own form and became a very different document from 2005 LATBSDC by the time it was published (SEAONC 2007). The authors of AB-083 graciously encouraged LATBSDC to adopt as much of the language of AB-083 as deemed necessary in development of 2008 LATBSDC (Maffei 2007). While AB-083 and 2008 LATBSDC share the same roots and much of the same language, their approaches to seismic design of tall buildings are markedly different. Both 2005 LATBSDC and AB-083 embody prescriptive code approaches where a few carefully enumerated exceptions are permitted. 2008 LATBSDC, in contrast, completely disengages from prescriptive requirements and bases all of its provisions on Capacity Design and Performance Based Design methodologies.

The following is a list of major changes that distinguish this document from 2005 LATBSDC:

- explicit adoption of Capacity Design approach to proportioning the structural system of the building;
- elimination of explicit and prescriptive code-based life safety evaluation step;
- significant revisions in serviceability provisions;
- significant revisions in collapse prevention provisions;
- simplified load combinations
- significant revisions in peer review requirements; and
- adoption of detailed seismic instrumentation requirements.

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.

2.2. Scope

Application of the procedure contained in this document is limited to tall buildings. For the purpose of this document, tall buildings are defined as those with 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.

C.2.2. Although nothing in this document limits applicability to shorter buildings, the scope of application has been intentionally narrowed to tall buildings. The reason is twofold. First, the Council considers shorter buildings outside its scope of activities. Second, the procedure contained in this document is complex and time-consuming and therefore such an elaborate procedure is doubtful to be justified for smaller, less complex buildings.

2.3. Justification

This provisions of this document are justified based on Section 104.11 of 2006 edition of International Building Code (2006 IBC) and Section 108.7 of 2007 California Building Code. These code provisions permit application of alternative lateral-force procedures using rational analysis based on well-established but complex principles of mechanics in lieu of prescriptive code provisions.

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 2006 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 108.7 of 2007 CBC states the following:

> *"The provisions of this code, as adopted by the Department of Housing and Community Development are not intended to prevent the use of any alternative material, appliance, installation, device, arrangement, method, design or method of construction not specifically prescribed by this code."*

Furthermore, Section 12.6 of ASCE 7-05 which is adopted by reference in 2007 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. ..."*

2.4. Methodology

The alternative procedure contained in this document is based on capacity design principles followed by a series of state-of-the-art 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 design and acceptable building performance shall be demonstrated using two distinct levels of earthquake ground motions:

1. Serviceable Behavior When Subjected to Frequent Earthquakes. 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). 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 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. Under this level of earthquake 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.

2. Very Low Probability of Collapse Under Extremely Rare Earthquakes. The extremely rare earthquake shall be taken as an event having a 2% probability of being exceeded in 50 years (2,475 year return period) with a deterministic cap. This earthquake is the Maximum Considered Earthquake (MCE) as defined by

ASCE 7-05 and adopted by 2006-IBC and 2007-CBC. The purpose of this evaluation is to safeguard against collapse during extremely rare events. This evaluation shall be performed using three dimensional nonlinear dynamic response analyses. This level of evaluation is intended to demonstrate that collapse does not occur when the building is subjected to the above-mentioned ground motions. Demands are checked against both structural members of the lateral force resisting system and other structural members. Claddings and their connections to the structure must accommodate MCE displacements without failure.

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

Design / Evaluation Step	**Ground Motion Intensity[1]**	**Type of Analysis**	**Type of Mathematical Model**	**Accidental Torsion Considered?**	**Material Reduction Factors (ϕ)**	**Material Strength**
1	Nonlinear Behavior Defined / Capacity Design					
2	50/30	LDP[2] or NDP[3]	3D[4]	Evaluated	1.0	Expected properties are used throughout except when calculating the capacity of brittle elements where specified strength values shall be used.
3	MCE[5]	NDP	3D[4]	Yes, if flagged during Step 2. No, otherwise.	1.0	

1 probability of exceedance in percent / number of years
2 linear dynamic procedure
3 nonlinear dynamic procedure
4 three-dimensional
5 per ASCE 7-05

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 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 very low probability of collapse during an extremely rare event (2% in 50 years with deterministic cap).

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 a state-of-the-art embodiment of the philosophy deeply rooted and implicit in most building codes requiring that buildings be able to[1]:

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, without collapse, but possibly with some structural as well as nonstructural damage.

In its conceptual framework for performance based design, SEAOC[2] suggested the following levels for design and verification:

Event	Recurrence Interval	Probability of Exceedance
Frequent	43 years	50% in 30 years
Occasional	72 years	50% in 50 years
Rare	475 years	10% in 50 years
Very Rare	975 years	10% in 100 years

The same SEAOC performance based design framework recommends the following seismic performance objectives for new construction:

Earthquake Design Level	Earthquake Performance Level			
	Fully Operational	Operational	Life Safe	Near Collapse
Frequent (43 years)	Basic Objective	Unacceptable	Unacceptable	Unacceptable
Occasional (72 years)	Essential/Hazardous Objective	Basic Objective	Unacceptable	Unacceptable
Rare (475 years)	Safety Critical Objective	Essential/Hazardous Objective	Basic Objective	Unacceptable
Very Rare (975 years)	Not Feasible	Safety Critical Objective	Essential/Hazardous Objective	Basic Objective

The intent of the procedure contained in this document is that buildings designed according to it meet or exceed the Basic Objective delineated above.

1. SEAOC, *Recommended Lateral Force Requirements and Commentary*, 1967 Edition, Section 2313(a).

2. SEAOC, *Recommended Lateral Force Requirements and Commentary*, 1999, 7th Edition, Appendices G and I.

C.5.1. (continued).

This objective is achieved by requiring serviceability at a 50% in 30 years event and collapse prevention at a 2% in 50 years event (with deterministic limit).

The Rational for Elimination of Explicit Life Safety Evaluation:

The 2007 California Building Code is based on the *2006 International Building Code*, which adopts by reference the ASCE 7-05 seismic provisions. Commentary to the ASCE 7-05 seismic provisions can be found in *FEMA 450 Part 2, Commentary*. This commentary clearly states that for buildings of ordinary occupancy, the intent of the provisions is to provide a low probability of collapse for buildings experiencing the Maximum Considered Earthquake (MCE) shaking. MCE shaking is defined either as that shaking having a 2% probability of exceedance in 50 years (2,475 year mean recurrence interval) or at sites near major active faults, 150% of the median shaking resulting from a characteristic magnitude earthquake on that fault, whichever is less. This approach is in stark contrast to predecessor codes, such as the *Uniform Building Code*, which sought a design goal of Life Safety performance for a design earthquake, having a 10% probability of exceedance in 50 years (475 year recurrence).

The older codes did not directly provide for protection against collapse under extreme shaking such as the MCE. Thus, the newer code requirements provide more explicit protection against collapse than did earlier codes. In order to retain *R* coefficients and design procedures familiar to users of the older codes, the new code adopts design-level earthquake shaking for purposes of evaluating strength and deformation that is 2/3 of the intensity of MCE 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 2007 CBC. 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. 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 a project-specific seismic design criteria. The project-specific seismic design criteria shall clearly describe how the structural system will achieve the following characteristics:

(a) Structural system for the building has well defined inelastic behavior where nonlinear actions and members are clearly defined and all other members are stronger than the elements designed to experience nonlinear behavior; nonlinear action is limited to the clearly defined members and regions; and

(b) Structural system for the building has a minimum amount of base shear strength (see Section 3.2.2).

3.2.1. 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 event can be accommodated with enough reserve capacity to avoid collapse.

C.3.2.1 Limiting occurrence of nonlinear behavior to limited and clearly identified areas of the building that are designed to absorb 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.

C.3.2.1 (continued).

Table C.3.2.1 Zones and actions commonly designated for nonlinear behavior

Structural System	***Zones and Actions***
Special Moment Resisting Frames (steel , concrete, or composite)	• *Flexural yielding of Beam ends (except for transfer girders)* • *Shear in Beam-Column Panel Zones* • *P-M-M* yielding at the base of columns (top of foundation or basement podiums)*
Special Concentric Braced Frames	• *Braces (yielding in tension and buckling in compression)* • *P-M-M yielding at the base of columns (top of foundation or basement podiums)*
Eccentric Braced Frames	• *Shear Link portion of the beams (shear yielding preferred but combined shear and flexural yielding permitted).* • *P-M-M yielding at the base of columns (top of foundation or basement podiums)*
Unbonded Braced Frames	• *Unbonded brace cores (yielding in tension and compression)* • *P-M-M yielding at the base of columns (top of foundation or basement podiums)*
Special Steel-Plate Shear Walls	• *Shear yielding of web plates* • *Flexural yielding of Beam ends*
R/C Shear Walls	• *P-M-M yielding at the base of the walls (top of foundation or basement podiums) or other clearly defined locations with plastic hinge region permitted to extend to a reasonable height above the lowest plane of nonlinear action as necessary.* • *Flexural yielding and/or shear yielding of link beams*
Foundations	• *Controlled rocking* • *Controlled settlement*

* yielding caused by combined axial force and uniaxial or biaxial flexure

3.2.2. Minimum Base Shear Strength

The buildings designed according to the provisions of this document shall satisfy the following minimum base shear strength requirement:

$$V_{\min} = 0.030W \qquad (1)$$

where V_{min} is the base shear strength corresponding to an essentially elastic behavior (see Section 3.3.6) of the structure and W is the total weight of the building above the base.

This requirement may be satisfied by demonstrating existence of the minimum base shear strength by performing elastic response spectrum analyses where the design spectrum is scaled to produce a CQC base shear of equal or larger than V_{min} or by application of static lateral loads according to the provisions of Sec. 12.8.3 of ASCE 7-05.

C.3.2.2 Admittedly, imposition of a minimum base shear strength requirement is not a performance based design provision. Tall buildings designed and constructed in Los Angeles during the last high-rise construction boom of 1980s and early 1990s commonly used a minimum base shear of 0.03W as a lower limit on design base shear. The 0.03W minimum base shear related to yield level forces for steel structures. Requiring the same minimum base shear strength corresponding to essentially elastic behavior of the structure, is simply retention of this Los Angeles tall building design tradition.

The 2005 Edition of this document utilized 0.025W minimum base shear strength requirement which was 2.5 times the absolute minimum base shear requirement imposed in ASCE 7-05.

LATBSDC and its invited advisory group were of the opinion that elimination of prescriptive code evaluation from the current edition of this document justified retaining a minimum base shear strength requirement. As more information is developed on the performance of buildings analyzed and designed according to this document, this limit may be either modified or eliminated.

3.3. Serviceability

3.3.1. General

The purpose of this evaluation is to validate that the building's structural systems and its nonstructural components and attachments 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.3.3.1. The intent of this document is not to require that a structure remain fully linearly elastic for the serviceability ground motion. The analysis is permitted to indicate minor yielding of ductile elements of the primary structural system provided such results do not suggest appreciable permanent deformation in the elements, or significant damage to the elements requiring more than minor repair. The analysis is permitted to indicate minor and repairable cracking of concrete elements.

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 response history analysis is used, the selection and scaling of ground motion time series should comply with the requirements of Section 16.1.3 of ASCE 7-05 with the serviceability-level response spectrum used instead of the MCE response spectrum, and with the design demand represented by the mean of calculated responses for not less than seven appropriately selected and scaled time series.

3.3.2. 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) and may be represented in the form of either a site-specific design spectrum for elastic analyses or a suite of time histories if nonlinear analyses is performed. Ground motion time histories, if utilized, shall be scaled according to the provisions of Section 16.1.3 of ASCE 7-05.

3.3.3. Mathematical Model

A three-dimensional mathematical model 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. Expected properties are used throughout except when calculating the capacity of brittle elements where specified strength values shall be used.

C.3.3.3. 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, there is no reason to estimate the actual three-dimensional behavior of tall buildings by relying on approximate two-dimensional models. The accuracy obtained by using three-dimensional models substantially outweighs the advantage of the simplicity offered by two-dimensional models.

3.3.4. 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-delta 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. The structure shall be evaluated for the following load combination:

$$1.0D + L_{exp} + 1.0E$$

where D is the service dead load and L_{exp} is the expected service live load.

3.3.4.1. 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.

3.3.4.2. 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.

3.3.5. 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 7-05 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 3.4.3.1 and 3.4.3.2 for details).

3.3.6. Acceptability Criteria

3.3.6.1. 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:

- 1.0 for brittle actions such as shear, torsion and axial load.
- 1.2 for ductile actions such as flexure and tension (in steel members).

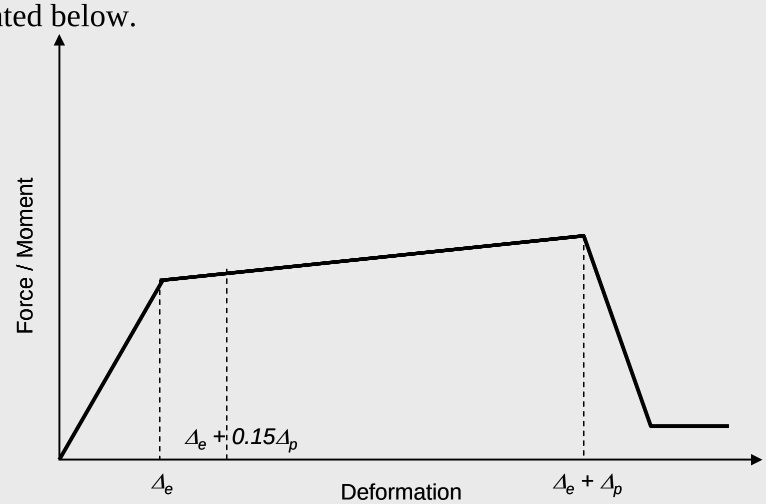
The overall drift of the structure does not exceed $0.005h_n$.

3.3.6.2. Nonlinear Dynamic Response Analyses

A minimum of three pairs of time histories scaled per provisions of Section 16.1.3 of ASCE 7-05 shall be utilized (use of seven or more pairs recommended). Note that time histories are scaled to the 5% damped serviceability design spectrum. If three pairs are used the maximum response values are used for evaluation, otherwise the average of the maximum values are used. The structure shall be deemed to have satisfied the acceptability criteria if:

- Force demands do not exceed the capacities for brittle actions (i.e., shear, axial force, etc.).
- Inelastic deformation demand ratios do not exceed $\Delta_e + 0.15\Delta_p$ for ductile actions where Δ_e is deformation level corresponding to maximum elastic deformation and Δ_p is the deformation corresponding to maximum plastic deformation without significant degradation, and
- The overall drift of the structure does not exceed $0.005h_n$.

C.3.3.6. Limited nonlinear behavior is permitted for ductile actions. This limited nonlinear behavior is evaluated in elastic response spectrum analysis by increasing the permitted maximum demand to capacity ratio to 1.2 instead of 1.0 for ductile actions. For nonlinear response evaluations, 15% of the inelastic deformation capacity for ductile actions may be utilized as illustrated below.



3.4. Collapse Prevention

3.4.1. Ground Motion

3.4.1.1. Design Spectra

Maximum Considered Earthquake (MCE) 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 7-05. The MCE ground motions shall be taken as that defined in Chapter 21 of ASCE 7-05.

3.4.1.2. Time Histories

A suite of seven or more appropriate ground motion time histories shall be used in the analysis. Ground motion histories and their selection shall comply with the requirements of Section 16.1.3 of ASCE 7-05. 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.

C.3.4.1.2. 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. Section 16.1.3 of ASCE 7-05 contains well-established procedures for selection of time-histories and, therefore, is adopted by reference in this document.

3.4.2. Mathematical Model

A three-dimensional mathematical model of the physical structure shall be used that represents the spatial distribution of the mass and stiffness of the structure. P-Δ effects shall be included in all nonlinear response history analyses. 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 properties are used throughout except when calculating the capacity of brittle elements where specified strength values shall be used. The stiffness properties of reinforced concrete shall consider the effects of cracking on initial stiffness.

The effective initial stiffness of steel elements embedded in concrete shall include the effect of the embedded zone. For steel moment frame systems, the contribution of panel zone (beam-column joint) deformations shall be included.

All structural elements for which demands for any of the response-history 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.

P-delta effects that include all the building dead load shall be included explicitly in the nonlinear response history analyses.

The properties of elements in the analysis model shall be determined considering earthquake plus expected gravity loads. In the absence of alternative information, gravity load shall be based on the load combination $1.0D + L_{exp}$.

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

Table 2. Expected Material Strengths

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

C.3.4.2. 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 41-06 for concrete and reinforcing steel; and 2005 AISC Seismic Provisions for structural steel.

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

3.4.3. Analysis Procedure

Three-dimensional nonlinear response history (NLRH) analyses of the structure shall be performed. The effect of accidental torsion shall be examined as described in Section 3.4.3.1 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; 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:

$$1.0D + L_{exp} + 1.0E$$

3.4.3.1. Accidental Torsion

If serviceability evaluation indicates that accidental torsion must be included (see Section 3.3.5), a pair of time histories 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 their 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 (γ) exceeds 1.20, the permissible force and deformation limits for corresponding actions shall be divided by the corresponding (γ) value.

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

3.4.3.2. Sensitivity Analyses

In lieu of accidental torsion analysis of Section 3.4.3.1 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.3.4.3. In the 2005 Edition of this document accidental eccentricity analysis was included in the code prescribed life-safety evaluation procedures. Since in the current document those prescriptive provisions have been eliminated, this issue needed to be addressed within either serviceability evaluation or collapse prevention evaluation, or both. The implemented procedure flags importance or insignificance of accidental eccentricity issue during the less cumbersome, serviceability evaluation. If during 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.

3.4.4. Acceptability Criteria

Structural strength and deformation capacities shall not be less than demands determined under Section 3.4.3 of this document. The structural elements or actions that are designed for nonlinear seismic response shall be clearly identified. All other elements and actions shall be demonstrated by analysis to remain essentially elastic.

For structural elements or actions that are designed for nonlinear seismic response, the adequacy of individual elements and their connections to withstand the deformation demands shall be evaluated. Force and deformation capacities shall be based on applicable documents or representative test results, or shall be substantiated by analyses using expected material properties. The average result, over the NLRH analyses, of peak story drift ratio shall not exceed 0.03 for any story.

The demand values (for member forces, member inelastic deformations, and inter-story drift) 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.

C.3.4.4. Deformation capacities may be assumed to be equal to the corresponding Primary Collapse Prevention values published in ASCE 41 (with Supplement 1) for nonlinear response procedures.

> Exception: Larger deformation capacities may be used only if substantiated by appropriate laboratory tests and approved by the Peer Review Panel and the Building Official. If ASCE 41-06 (with Supplement 1) Primary Collapse Prevention deformation capacities 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 41-06 (with Supplement 1) Primary Collapse Prevention limits.

ASCE 41-06 (with Supplement 1) 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.

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 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 should include a minimum of three members with recognized expertise in relevant fields, such as structural engineering, earthquake engineering research, performance-based earthquake engineering, nonlinear response history 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. Based on this, 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 should include review of the following: earthquake hazard determination, ground motion characterizations, seismic design methodology, seismic performance goals, acceptance criteria, mathematical modeling and simulation, seismic design and results, drawings and specifications.

The SPRP should 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 should 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 should be selected to provide the most useful data for post-event condition assessment, although variations in the instrumentation scheme for a given building type may be warranted to provide a broader range of data given the relatively sparse instrumentation required.

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. Recorders and accelerometers must be of a type approved by the California Geologic Survey (CGS).

5.3. Minimum Number of Channels

The building shall be provided with minimum instrumentation as specified in the Table 3. The minimum number of required channels maybe increased at the discretion of SPRP and Building Official.

Each channel corresponds to a single response quantity of interest (e.g., unidirectional floor acceleration, interstory displacement, etc.).

Table 3. Minimum Number of Channels of Instrumentation

Number of Stories Above Ground	Minimum Number of Channels
10 – 20	15
20 – 30	21
30 – 50	24
> 50	30

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.

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.

References

American Institute of Steel Construction (2005), *Seismic Provisions for Structural Steel Buildings,* ANSI/AISC 341-05, Chicago, Illinois..

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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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