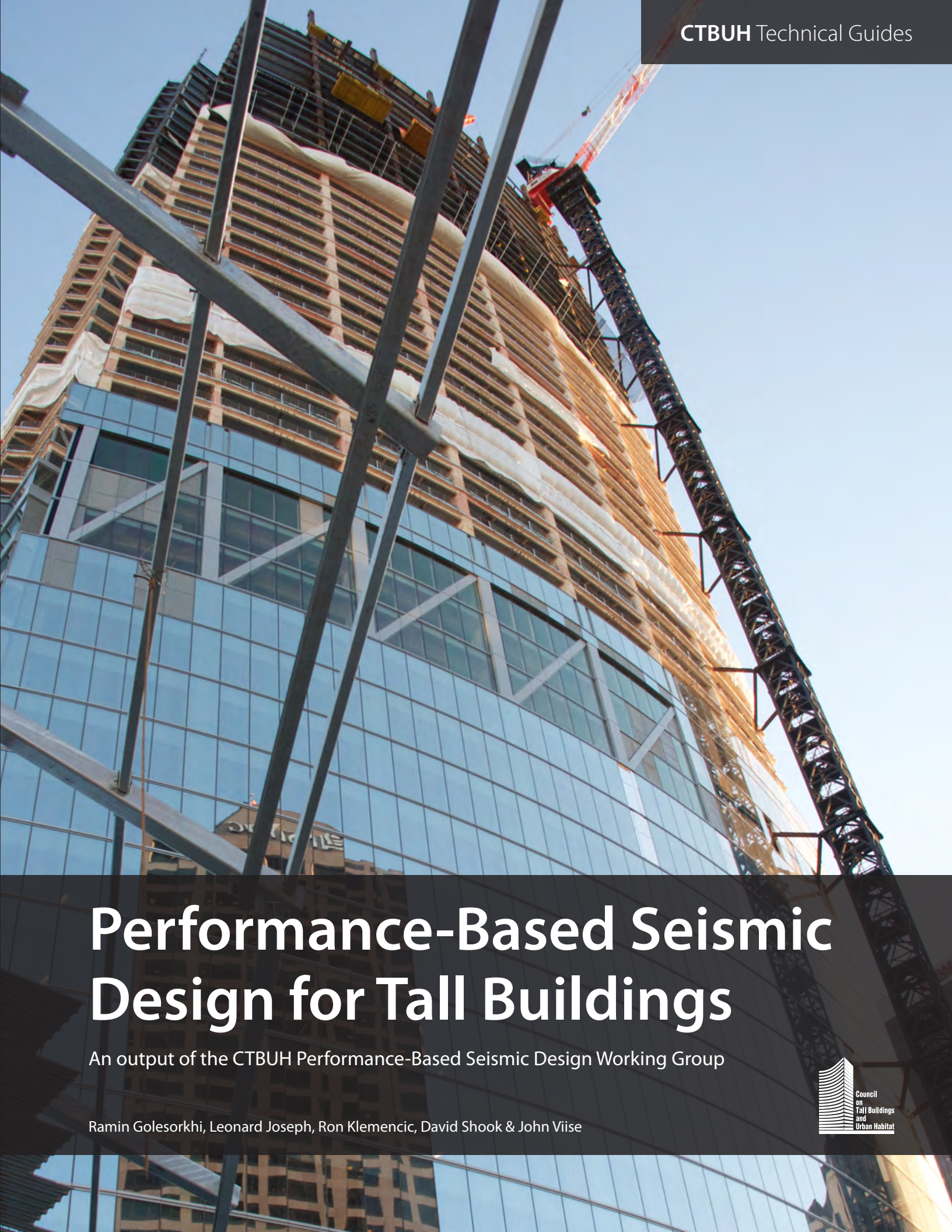

CTBUH Technical Guides

Performance-Based Seismic Design for Tall Buildings

An output of the CTBUH Performance-Based Seismic Design Working Group

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Council on Tall Buildings and Urban Habitat

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Front Cover Image: Wilshire Grand, Los Angeles, under construction in 2015. © Gary Leonard/AC Martin
Opening Chapter Image: 350 Mission St., San Francisco, designed using PBSD principles. © Cesar Rubio

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About CTBUH Working Groups

CTBUH Working Groups report on specific aspects of the planning, design, construction, and management of tall buildings. They are not standing committees, but groups that form for a period of time, specifically around a need or important topic in the industry, with the aim of disseminating their findings through the publication of a technical guide.

See more on other CTBUH Working Groups at: workinggroups.ctbuh.org

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Preface

In 2008, the Council on Tall Buildings and Urban Habitat (CTBUH) Seismic Working Group authored the publication "Recommendations for the Seismic Design of High-Rise Buildings." This document and subsequent working group meetings established that a consensus of practitioners believe the process of Performance-Based Seismic Design (PBSD) is often more appropriate than prescriptive code-based approaches for the design of tall buildings in regions of high seismicity. Given that 75% of the tallest buildings completed in 2016 were constructed in seismic regions of the world where some form of non-prescriptive design and analysis was necessary for building approval (The Skyscraper Center, 2016), it is apparent that publishing the design principles inherent to the PBSD design process would be useful to an international audience. As a result, the CTBUH Performance-Based Seismic Design (PBSD) Working Group was formed with the goal of producing a publication to introduce PBSD principles to an international audience and provide examples of its application.

Although the practice and protocol for non-prescriptive design is quite mature in certain countries (e.g., China and Japan), the methods used for PBSD as practiced in western regions of the United States are of high interest to other countries. PBSD guidelines have been reconsidered based on local practice and implemented into the design of tall buildings in areas including Turkey, the Philippines, and Russia. The methods used for PBSD have produced innovative and cost-effective buildings in these regions. As a result, this publication may be an especially helpful reference for practitioners working internationally, and for jurisdictions looking to develop their own PBSD guidelines and protocols for a design approval process.

In adopting this methodology for use outside the US, it is recognized that local design practices will vary internationally. Technical areas where significant differences with US practice may occur would include performance criteria, approvals processes, materials design standards, and definitions of seismic hazard. Local structural engineers should closely review and resolve these issues with local municipalities and appropriate approval authorities.

It is the intent of the CTBUH PBSD Working Group to disseminate these methods to an international audience for the advancement and expansion of PBSD principles. Additionally, the presentation of several case study examples demonstrate the issues commonly encountered when using PBSD for the design of tall buildings and how practicing engineers have resolved them.

1.0 Introduction

Performance-Based Seismic Design (PBSD) is a structural design methodology that has become more common in urban centers of the western United States, especially for the design of high-rise buildings. It is a design methodology that allows for design flexibility and offers design opportunities to enhance building performance and encourage innovation. The most common use of PBSD in practice is to substantiate exceptions to specific prescribed code requirements, such as height limits on select structural systems. A second use of PBSD is the ability to demonstrate higher performance levels for a structure at various intensities of a seismic event.

An integral component of PBSD is Nonlinear Response History Analysis (NRHA). This advanced method of analysis has been incorporated into the design process in regions with high seismicity, such as in China, Philippines, Malaysia, etc. The design checks required by the process of PBSD are extensive and require substantial knowledge of nonlinear seismic design, building performance, and analytical modeling. These demands have not limited the design of structures, but instead led to a number of highly efficient tall building designs that would not be possible following a traditional code-prescriptive design approach.

▲ **Figure 1.1: One Rincon Hill, San Francisco, designed using PBSD principles.** © Magnusson Klemencic Associates

PBSD is currently accepted in numerous urban centers of the United States such as Los Angeles (see Figures 1.1 and 1.2), San Francisco, Seattle, San Diego, and Salt Lake City. The current version of the American Society of Civil Engineers loads standard (ASCE 7–16) includes a detailed framework for PBSD, making it possible to use PBSD methods in all US jurisdictions adopting this standard. The broad acceptance of this methodology in the US will lead to a more detailed understanding of building response in seismic events and allow for further innovations in seismic design.

This publication provides structural engineers, developers, and contractors – in the US and internationally – a general understanding of the PBSD process and examples from leading structural engineering firms with a history of designing tall buildings in high seismic zones. This publication is not intended as a standard such as ASCE 7, or as a group of guidelines such as PEER/TBI and LATBSDC. Instead, this is a bridging document to introduce PBSD methods to an international audience. Structural engineers should look to develop a project-specific basis of design founded on the references provided and engage their local jurisdictions for appropriate steps needed for project approval. The PBSD process is regularly evolving and the latest standards and guidelines should be referenced.

1.1 Overview of Performance-Based Seismic Design

Performance-based seismic design is a highly developed design methodology that provides greater design flexibility to structural engineers than that afforded by prescriptive code-based approaches. However, the methodology also involves significantly more effort in the analysis and design stages, with verification of building performance required at multiple seismic demand levels using linear and advanced nonlinear analysis techniques. PBSD uses first principles of engineering to proportion and detail structural systems and components to meet specific performance objectives.

Using PBSD methodology, the focus of the structural engineer changes from a prescriptive "check list" approach of code provisions to requiring the designer to more fully understand building performance and the code's intent. Finding solutions through a more detailed knowledge of building behavior in a seismic event often results in cost-efficient solutions that satisfy the targeted performance levels more efficiently. Although PBSD requires additional design effort, the benefits can be significant: reduced construction costs, improved lease spaces, and enhanced seismic performance.

▲ Figure 1.2: Wilshire Grand Center, Los Angeles, designed using PBSD principles. © Gary Leonard/AC Martin

1.2 Goals of PBSD

Developers and structural engineers will incorporate PBSD for a variety of reasons. Common goals of PBSD include:

- the ability to make exceptions to specific code requirements, such as height limits for select seismic force-resisting systems;
- the use of seismic force-resisting systems and innovative designs not prescribed by code;
- the use of high-strength materials and mechanical devices not prescribed by code; and
- the reduction of structural and non-structural damage through enhanced seismic damage performance objectives at specified levels of seismic intensity.

A common example of a seismic force-resisting system not recognized by code is a core-and-outrigger seismic force-resisting system. In the United States, this is not one of the 82 seismic force-resisting systems recognized in ASCE 7. The use of PBSD methods facilitates a method to evaluate and design such seismic force-resisting systems.

1.3 Historical Development of PBSD Provisions

Historically significant earthquake events (e.g., 1971 San Fernando, 1989 Loma Prieta, and 1994 Northridge) caused significant damage and down-time to large manufacturing businesses in California. As a result of these seismic events, major market sectors like the airline industry in the Los Angeles area and the computing industry in Silicon Valley desired to enhance the performance of their buildings to minimize the risk of casualties, damage to facilities, and down-time of their existing and new facilities should a more frequent event occur. This demand served as a catalyst to the engineering community in the US to develop design methods to assess performance of existing structures and to develop design methodologies to enhance the performance of these systems, as well as ways to quantify the impact of these enhancements. Performance of existing structures is quantified by the development of performance objectives that are defined for structural systems and components of the system.

Principles central to PBSD were developed to rationally and efficiently guide the design of seismic retrofits to enhance the performance of existing structures. These provisions ultimately resulted in ASCE 41. The current ASCE 41-13 (Seismic Evaluation and Retrofit of Existing Buildings) outlines a series of evaluation levels for existing buildings. Some levels of these types of retrofits are defined as Tier 1 and 2, which involve more prescriptive procedures. The Tier 3 methodology utilizes PBSD principles and includes performance objectives that are implied in the code and illustrated in Figure 1.3.

Current PBSD documents such as PEER/TBI (The Pacific Earthquake Engineering Research Center/Tall Buildings Initiative) and LATBSDC (Los Angeles Tall Buildings Structural Design Center) refer to ASCE 41-13 for acceptance criteria at performance levels described in Section 1.6 of deformation-controlled elements

2.0 Site-Specific Seismic Hazard Assessment

2.1 Introduction

The seismic design of structures should include proper evaluation of seismic hazards. These hazards include the level of ground shaking for structural design and liquefaction, ground deformations, loss of bearing, and slope stability hazards that may impact the performance of foundations. PBSD guidelines typically recommend establishing site-specific ground motions, rather than using a prescriptive code spectrum, scaled up for the Risk-Targeted Maximum Considered Event or scaled down for the Service Level Earthquake (SLE).

Open communication between the geotechnical and structural engineers is critical in the development of site-specific ground motions for tall buildings. These communications ideally should be initiated at the outset of projects. Information regarding structural periods for the fundamental and higher modes is imperative for the development of appropriate ground motion criteria (e.g., time series) for structural evaluations and design. If a detailed Soil-Structure-Interaction (SSI) analysis is performed, additional information regarding location, spacing, and dimensions of structural elements (e.g., basement walls, basement floors, mat foundation, deep foundation elements, etc.) and properties of the structural elements (e.g., Poisson Ratio, Young's/Elastic Modulus, unconfined compressive strength for concrete elements, minimum yield strength for steel elements, moment of inertia, cross-sectional area, etc.) are needed.

Levels of shaking for design (i.e., earthquake-induced forces) are in general quantified by acceleration response spectra. A response spectrum is the maximum response of an elastic single-degree-of-freedom (SDOF) simple damped oscillator for a number of natural periods and a specified critical damping ratio. In recent building codes, starting with ASCE 7-10, the spectra for design are quantified in the maximum direction rather than the geometric mean (geo-mean). The quantification of the maximum-direction spectra are generally done by correcting the SDOF geometric mean spectrum via correction factors. However, the correction factors have been developed by computing the maximum response of an elastic two-degree-of-freedom (TDOF) simple damped oscillator and comparing it to the SDOF oscillator response. Walker et al. (2010) present a comprehensive discussion of the maximum-direction spectra.

2.2 Developing Site-Specific Target Response Spectra

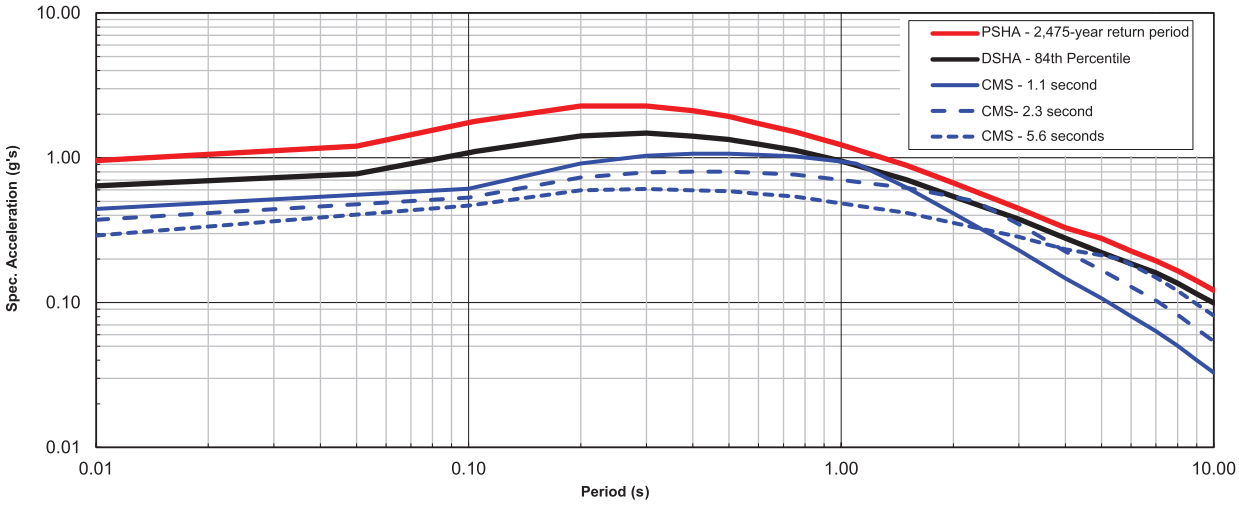
Generally, the level of shaking quantified as a Target Response Spectrum could be determined using probabilistic seismic hazard analysis (PSHA) (Cornell 1968 & McGuire 2004), deterministic seismic hazard analysis (DSHA), Conditional Mean Spectrum (Baker and Cornell 2006), and NIST (2011) or ground response analysis. All of these methods result in a target spectrum for ground motion scaling and matching.

Probabilistic Seismic Hazard Analysis (PSHA)
In a PSHA, a level of ground shaking is defined as a probability of exceedance in a given period of time, typically 50 years. The spectral values are developed for the same mean annual frequency of exceedance, which represents a uniform hazard, hence the term Uniform Hazard Spectrum (UHS). A UHS includes earthquake hazard from all considered sources in the area of study and does not represent a single earthquake. A typical level of hazard defined in US building codes is the Risk-Targeted Maximum Considered Earthquake (MCE_R), which corresponds to a two percent probability of exceedance in 50 years; however, in highly seismic active areas there is typically a deterministic cap for the MCE_R (see Section 2.5).

Deterministic Seismic Hazard Analysis (DSHA)
A DSHA represents a scenario earthquake approach. It is a relatively simple approach that considers the occurrence of an earthquake of a particular magnitude, typically a maximum earthquake on a particular fault and the closest distance to the fault. Uncertainty is considered through the use of standard deviation of the predictive relationships. The typical spectral levels considered are the median or 84th percentiles.

Conditional Mean Spectra (CMS)
CMS is an alternative approach to the target spectrum, determined either using PSHA or DSHA. It can be used as a target spectrum in the selection and scaling/matching of time series for nonlinear structural analysis. The goal of this approach is to address the conservatism in the UHS and the MCE_R deterministic spectra. CMS provides a methodology such that the expected mean response spectrum is conditioned on the occurrence of a target spectral acceleration value at the period of interest. Because the Uniform Hazard Spectrum (UHS) is a summation of hazards from all sources, it does not represent a scenario earthquake and provides higher spectral values than the CMS at all periods except the period of interest. Therefore, CMS can be used as the basis to develop and select an appropriate suite of time series for different spectral periods.

Figure 2.1 presents an example of the PSHA, DSHA, and CMS for three conditioning periods for the DSHA



▲ Figure 2.1: Example of Site-Specific Spectra. © Langan

spectral levels. CMS were developed for periods of 1.1, 2.3, and 5.6 seconds.

Ground Response Analysis

Ground response analysis is a computational technique based on the theory of wave propagation through the soil. For this analysis, an idealized soil column is shaken by an earthquake (input) time series at the base layer. The nonlinear soil behavior is modeled by an equivalent-linear approach (SHAKE-91; Idriss and Sun, 1992) and nonlinear approach (DEEPSOIL; Hashash et al., 2015 and D-MOD 2000, Matasovic and Ordonez, 2011, etc.). To quantify the interaction of the structure with the soil, SSI analyses are appropriate. While these types of evaluations are not routine and are not required, they are becoming more common in the development of site-specific ground motions for tall and supertall structures. SSI models, which use nonlinear ground-response finite-element or finite-difference models of the soil and structure, quantify the stress-strain behavior of soil material in a more direct fashion. It should be noted that for a two-layer site condition (e.g., 40 meters of fill and soft clay over rock), the use of NGA-West2 attenuation relationships, which are based on Vs30 values (average shear wave velocity in the top 30 meters, measured from the ground surface or below the basement level, see ASCE 7-16) could result in an overestimation of long-period spectral values, which are important for tall buildings. For these site conditions, it is suggested to develop ground motion at the surface of rock/firm soil depth and perform ground-response analysis to arrive at more reasonable ground-surface or basement-level spectral response values. Two- and three-dimensional nonlinear SSI analyses using computer programs – such as FLAC, Plaxis, LS-DYNA, SASSI, ADINA, OpenSees, and Midas – are some of the modeling techniques used by geotechnical practitioners. In addition, guidelines for incorporation of kinematic and inertia interaction effects are provided in NIST (2012).

2.3 Range of Structural Periods for Consideration

The use of CMS as a tool for selection and scaling/matching of time series has become the preferred method for the development of site-specific ground motions for structural evaluations and design. As presented in Figure 2.1, CMS is equal to the target spectrum at the conditioning periods and is less than the target spectrum for other periods. The CMS values presented in Figure 2.1 for the three conditioning periods are: 0.91g for a 1.1-second conditioning period, 0.50g for a 2.3-second conditioning period and 0.20g for a 5.6-second conditioning period. Therefore, it is imperative that an adequate number of CMS is developed such that the drop in the spectral values is not too severe. The provisions of ASCE 7-16 require that the envelope of the CMS does not fall below 75% of the target spectrum. Typically, two conditioning periods, one representing the fundamental mode of the structure and one short period representing the second or higher mode vibrations, are considered. However, three or more conditioning period(s) of CMS may be required in the event that the fundamental mode period and the higher mode period are too far apart. The determination of these spectral periods is the

3.0 Design Using Linear Analysis

The initial proportioning of a building consists of a complete design process whereby all members of the seismic force-resisting system are proportioned. Linear design can be done using SLE-, DE-, or MCE_R-level earthquake demands. Most engineers prefer using SLE demands, with design methods appropriately adjusted for the lower demand level. Some engineers have utilized DE or MCE_R level demands based on particular building types or preference. When this is done, additional verification at DE or SLE may be required to substantiate building performance and code equivalency. SLE-based design is primarily considered in this document and is described in detail in PEER/TBI and LATBSDC documents. The intent of designing using SLE-level demands is to inherently satisfy DE performance objectives by verifying performance under SLE and MCE_R demands. If specific performance objectives are targeted at SLE-, DE-, and MCE_R-level ground shaking, verification at each level may be required.

3.1 Modeling and Analysis

For initial design using linear analysis, modeling and design methods appropriate for the level of earthquake demands should be considered. Response spectrum analysis is typically used. As mentioned above, for this document SLE demands are used for initial linear design. Material strength and stiffness assumptions, section property modifiers, and material strength reduction factors appropriate for SLE demands should be used and differ from DE-level assumptions. Material and section property modifiers appropriate for SLE-level design are described in detail in PEER/TBI and LATBSDC, with key parameters in Tables 3.1 & 3.2.

The analysis model should include all lateral force-resisting elements, primary gravity system elements, and basements. P-Delta effects should be included. Slab openings affecting diaphragm stiffness should be included with semi-rigid diaphragm modeling.

Materials

For SLE-level design, expected material properties should be utilized for realistic estimates of stiffness. For projects using ASCE 7 criteria and specified ASTM material standards, unless more detailed justification can be produced, expected material properties as shown in Table 3.1 can be used. In jurisdictions not using ASCE 7 and associated ASTM standards, robust testing of local materials or historical information

Material		Expected Strength	
		Expected Yield Strength, f_{ye}, psi	Expected Ultimate Strength, f_{ue}, psi
Reinforcing Steel	A615 Grade 60	70,000	106,000
	A615 Grade 75	82,000	114,000
	A706 Grade 60	69,000	95,000
	A706 Grade 80	85,000	112,000
Structural Steel***	Hot-rolled structural shapes and bars		
	ASTM A36/A36M	1.5 f_y*	1.2 f_u**
	ASTM A572/A572M Grade 50	1.1 f_y*	1.1 f_u
	ASTM A913/A913M Grade 50, 60, 65 or 70	1.1 f_y*	1.1 f_u
	ASTM A992/A992M	1.1 f_y*	1.1 f_u
	Plates		
	ASTM A36/A36M	1.3 f_y	1.2 f_u
	ASTM A572/A572M Grade 50, 55	1.1 f_y	1.2 f_u
Concrete		$f'_{ce} = 1.3f'_c$ †	

*f_y is used to designate the specified (nominal) yield strength of steel materials in this Guideline. It is equivalent to f_y or f_{yr} used in ACI 318 and F_y used in AISC (2006) standards.
**f_u is used to designate the specified (nominal) ultimate strength of steel materials in this Guideline. It is equivalent to F_u used in AISC (2006) standards.
***For steel materials not listed, refer to Table A3.1 of ANSI/AISC 341-16
†f'_c = specified compressive strength. Expected strength f'_{ce} is strength expected at approximately one year or longer. Note that the multiplier on f'_c may be smaller for high-strength concrete, and can also be affected by (1) use of fly ash and other additives, and/or (2) local aggregates.

▲ **Table 3.1: Expected Material Strength.** Source: PEER/TBI

Component	Service-Level Linear Models			MCE_R-Level Nonlinear Models		
	Axial	Flexural	Shear	Axial	Flexural	Shear
Structural walls[1] (in-plane)	$1.0E_cA_g$	$0.75E_cI_g$	$0.4E_cA_g$	$1.0E_cA_g$	$0.35E_cI_g$	$0.2E_cA_g$
Structural walls (out-of-plane)	–	$0.25E_cI_g$	–	–	$0.25E_cI_g$	–
Basement walls (in-plane)	$1.0E_cA_g$	$1.0E_cI_g$	$0.4E_cA_g$	$1.0E_cA_g$	$0.8E_cI_g$	$0.2E_cA_g$
Basement walls (out-of-plane)	–	$0.25E_cI_g$	–	–	$0.25E_cI_g$	–
Coupling beams with conventional or diagonal reinforcement	$1.0E_cA_g$	$0.07\left(\frac{\ell'}{h}\right)E_cI_g$ $\leq 0.3E_cI_g$	$0.4E_cA_g$	$1.0E_cA_g$	$0.07\left(\frac{\ell'}{h}\right)E_cI_g$ $\leq 0.3E_cI_g$	$0.4E_cA_g$
Composite steel / reinforced concrete coupling beams [2]	$1.0(EA)_{trans}$	$0.07\left(\frac{\ell'}{h}\right)(EI)_{trans}$	$1.0E_sA_{sw}$	$1.0(EA)_{trans}$	$0.07\left(\frac{\ell'}{h}\right)(EI)_{trans}$	1.0EsAsw
Non-PT transfer diaphragms (in-plane only)[3]	$0.5E_cA_g$	$0.5E_cI_g$	$0.4E_cA_g$	$0.25E_cA_g$	$0.25E_cI_g$	$0.1E_cA_g$
PT transfer diaphragms (in-plane only)[3]	$0.8E_cA_g$	$0.8E_cI_g$	$0.4E_cA_g$	$0.5E_cA_g$	$0.5E_cI_g$	$0.2E_cA_g$
Beams	$1.0E_cA_g$	$0.5E_cI_g$	$0.4E_cA_g$	$1.0E_cA_g$	$0.3E_cI_g$	$0.4E_cA_g$
Columns	$1.0E_cA_g$	$0.7E_cI_g$	$0.4E_cA_g$	$1.0E_cA_g$	$0.7E_cI_g$	$0.4E_cA_g$
Mat (in-plane)	$0.8E_cA_g$	$0.8E_cI_g$	$0.8E_cA_g$	$0.5E_cA_g$	$0.5E_cI_g$	$0.5E_cA_g$
Mat[4] (out-of-plane)	–	$0.8E_cI_g$	–	–	$0.5E_cI_g$	–

[1] Values are relevant where walls are modeled as line elements. Where walls are modeled using fiber elements, the model should automatically account for cracking of concrete and the associated effects on member stiffness.
[2] *(EI)trans* is intended to represent the flexural rigidity of the cracked transformed section. It is acceptable to calculate the transformed section properties based on structural mechanics or to use $(EI)trans = E_cI_g/5 + E_sI_s$ per ACI 318.
[3] Specified stiffness values for diaphragms are intended to represent expected values. Alternative values may be suitable where bounding analyses are used to estimate bounds of force transfers at major transfer levels. For diaphragms that are not associated with major force transfers, common practice is to model the diaphragm as being rigid in its plane. Flexural rigidity of diaphragms out of plane is usually relatively low and is commonly ignored. The exception is where the diaphragm acts as a framing element to engage gravity columns as outrigger elements, in which case out-of-plane modeling may be required.
[4] Specified stiffness values for mat foundations pertain to the general condition of the mat. Where the walls or other vertical members impose sufficiently large forces, including local force reversals across stacked wall openings, the stiffness values may need to be reduced.

▲ **Table 3.2: Reinforced Concrete Effective Stiffness Values.** Source: PEER/TBI

is needed to estimate appropriate expected material parameters.

PEER/TBI recommends the use of expected material properties for analysis-model component stiffness, but specified material properties for component strength capacity.

LATBSDC recommends expected material properties for analysis-model component stiffness and strength capacity.

Since MCE_R evaluation using NRHA is also conducted, either method is valid, but the PEER/TBI method is more conservative.

Section Properties

In linear elastic analyses, section properties need to be reduced to account for cracking and damage to the components, through section property modifiers with reduced effective stiffness of the member. Property modifiers are based on experimental testing. Since SLE demands are often considered, LATBSDC and PEER/TBI have published concrete section property modifiers for use in SLE-and MCE_R-level events. The application of property modifiers can have a significant impact on member force levels and should be carefully considered for each project. Other resources that engineers should review include PEER/TBI (see Table 3.2), ASCE 41-13 Table 10-5 for all concrete elements, and ATC 72-1 Table 4-1 for link beams. For link beams reinforced with steel wide flanges, AISC 341-10 Commentary H4 can be consulted. It should be noted that there are inconsistencies between these documents, and engineers should use

In jurisdictions not using ASCE 7 and associated ASTM standards, robust testing of local materials or historical information is needed to estimate appropriate expected material parameters.

4.0 Verification of Response Under MCE_R Using NRHA

With the seismic force-resisting system fully proportioned using linear analysis, verification of performance under MCE_R level shaking using NRHA is conducted. The intent is to verify the design, but some design modifications can be made during this step to ensure design criteria and performance levels are satisfied. Generally, it is only required to repeat linear analysis checks if the MCE_R analysis and design leads to modifications to the non-negotiable dimensional proportioning of the structure.

MCE_R peak ground acceleration can be 4–8 times higher than SLE. It is expected that deformation-controlled actions of components of the structure will exhibit inelastic response, and as a result, a nonlinear analysis model that takes this inelastic response into account is necessary to appropriately evaluate the building performance objectives.

With appropriate modeling applied, global and component acceptance criteria need to be verified for conformance with specified performance objectives.

4.1 Modeling and Analysis

Modeling of elements is significantly more detailed with NRHA than RSA, and appropriate care should be given to all modeling assumptions. A much higher level of fidelity in modeling and results is possible with NRHA. This leads not only to a more refined understanding of building behavior, but also to opportunities for modest adjustments. For example, modeling methods in NRHA help identify with greater clarity the formation of a plastic hinge zone, or if a more distributed yielding is to be expected. This allows for more appropriate detailing in high-deformation areas, and more efficient reinforcement layouts elsewhere.

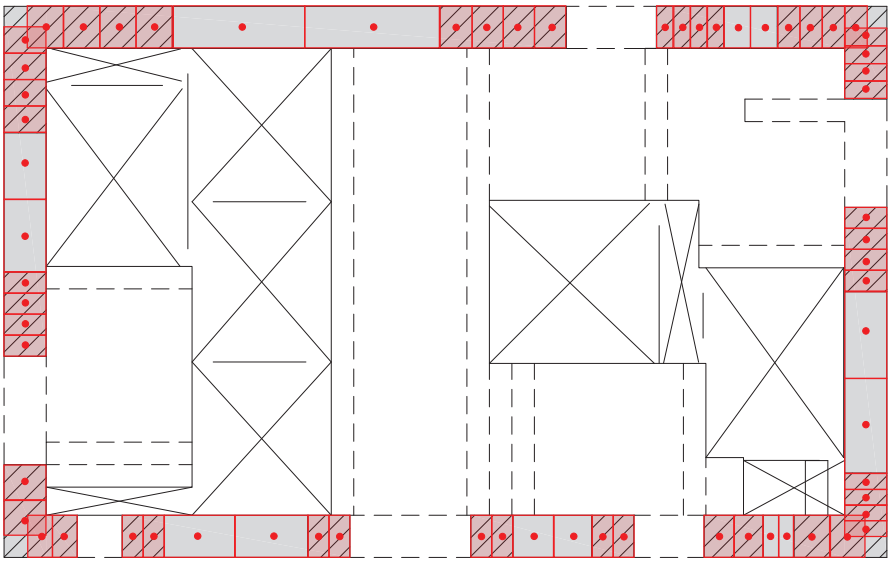
Component Modeling

All finite elements are composed of deformation and force-controlled actions identified in the linear analysis and design stage (see Table 3.3). For components with force- and deformation-controlled actions, elements are composed of linear and nonlinear responses. For components with only force-controlled actions, fully linear elastic assumptions are utilized.

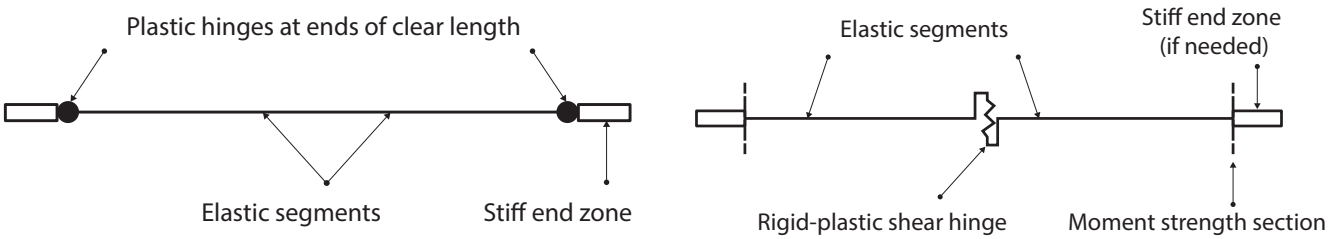
Although there are a variety of modeling methods available in a variety of software packages, commonly used component modeling methods are described in the following list. Further descriptions of these and other component modeling methods can be found in ATC-72 and ATC-114.

1. Wall Elements: Shear is a force-controlled action and modeled elastically. Axial and flexural behavior are represented with a series of nonlinear bar elements. Three types are typically included, and are referred to as "fibers" representing vertical wall reinforcement, confined concrete, and unconfined concrete. Each fiber has an independent backbone curve. A graphical example of a fiber arrangement in a core wall is shown in Figure 4.1. Here, each red dot represents a steel fiber and a concrete fiber. If the area is shaded red the concrete fiber is confined, if it is not shaded red it is unconfined. Fiber properties are determined by the material within each zone. Typically two to four fibers are located in boundary zones and only two are located in non-boundary areas between boundary zones. This allows for linear extrapolation of results outside of the fibers if desired.

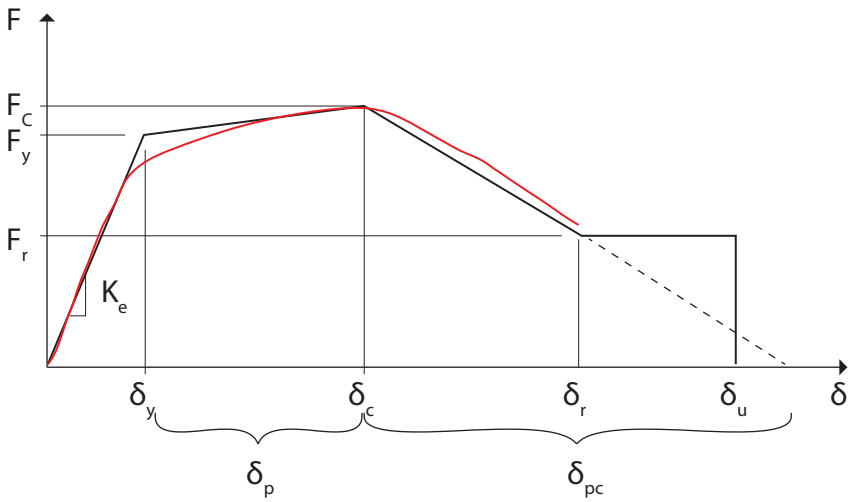
2. Coupling beams: These elements are often modeled with "lumped" plasticity as a mid-span hinge or as flexural hinges at each end. Behavior needs to be closely matched to representative physical testing, similar with material strengths, detailing, and span/depth ratios. Care should be taken to ensure the physical testing



▲ **Figure 4.1: Example Graphical Fiber Element Arrangement.** © Skidmore, Owings & Merrill LLP



▲ Figure 4.2: Element with Hinges at Each End (left) and Elements with a Hinge at Mid-Span (right). Source: CTBUH based on ATC-72



- Effective yield strength and deformation (F_y and δ_y)
- Effective elastic stiffness, $K_e=F_y/\delta_y$
- Strength cap and associated deformation for monotonic loading (F_c and δ_c)
- Pre-capping plastic deformation for monotonic loading, δ_p
- Effective post-yield tangent stiffness, $K_p=(F_c-F_y)/\delta p$
- Residual strength. $F_r=\kappa F_y$
- Ultimate deformation, δ_u

▲ Figure 4.3: Backbone Curve and Parameters (ATC-72) (Red = Tested, Black = Idealized). Source: CTBUH based on ATC-72

substantiating the proposed coupling beam is appropriate (see Figure 4.2). Refer to ASCE 41 and published physical testing results. In most cases, specific nonlinear properties are published. If only a hysteresis is available in a publication, engineers may need to overlay the hysteresis resulting from a proposed nonlinear backbone curve to verify their material model.

3. Gravity Columns: Axial compression is a force-controlled action and modeled elastically. If limited nonlinear behavior is permitted in the column, a flexural hinge at each end is appropriate. If flexure is to remain elastic, linear response is appropriate and reinforcement can be verified after analysis. If columns are modeled with nonlinear behavior, refer to ATC 114 for modeling information and ASCE 41 for rotation limits.

4. Slab Equivalent Frames: It is common to represent floor framing systems (beam or flat slab) using an equivalent frame with lumped plasticity at each end where yielding is anticipated. This is done to account for the "micro-outrigger" effect of flat slabs on columns, and to verify that framing rotation limits are satisfied. In tall buildings, floor framing systems will often yield in the upper portion of the tower, where the core lifts up and the gravity columns do not. Often flat-plate post-tensioned slabs are utilized and can yield significantly. Refer to ATC 114 for modeling information and ASCE 41 for rotation limits.

In some instances, deformation-controlled actions are modeled elastically. For example, while basement walls are categorized as deformation-controlled for flexure and force-controlled for shear, their non-linear behavior is not modeled, as they are not anticipated to yield.

Nonlinear Behavior Modeling with Lumped Plasticity

For deformation-controlled elements with a lumped plasticity, a nonlinear backbone curve is employed in the mathematical model using the general force deformation shape as shown in

5.0 Basis of Design Example

The purpose of a Basis of Design (BOD) document or design criteria document is to state deviations from governing code requirements, either exceptions or enhancements, and describe subsequent methods justifying these exceptions or enhancements. Content will often include descriptions of all structural systems, description of design procedure, performance objectives, analytical modeling methods, and acceptance criteria. It is not intended to contain all information used for the design of the building, but should be a standalone document with references to all needed information. No structural engineering results should be presented in the Basis of Design document. Typically, Basis of Design documents range from 10 to 25 pages in length. The BOD should be included in the design drawings for future reference by the building owner, especially if exceptions to code provisions are taken.

The Basis of Design document is generally submitted to the peer review panel and local governing jurisdictions involved in building permitting for review and comment early in the building design process. In some jurisdictions, the BOD is submitted with the architectural building site permit. Typically, the document can be updated and revised through the design process, as appropriate, to reflect the final design.

Engineers should review Appendix B of PEER/TBI for additional information.

5.1 General

Describe project location, structural system types used, and the most important building considerations.

Describe the primary load path if multiple systems are used, and, if so, their intended purposes. If higher occupancies require higher performance levels by the governing building code, specify.

Describe the site in terms of geographic coordinates and include a description of site considerations. Describe the relationship of local and national building code requirements to this project.

Describe locations of anticipated inelastic behavior and any enhanced seismic devices such as buckling restrained braces, isolation bearings, dampers, etc.

Representative design drawings should be included. This can be satisfied by placing the BOD on a drawing sheet as part of the set of building structural drawings.

5.2 Superstructure

This section will describe the superstructure, which includes the ground floor and above. For sloped sites, this would include all elements above grade.

If the building is connected to multiple buildings sharing a common basement, describe how they are interconnected (i.e., seismic joints, common transfer diaphragm, etc.). Include a schematic diagram showing their overall configuration.

Lateral System
Describe the lateral system in more detail, with typical dimensions and material strengths. Identify primary transfer diaphragms at- and above-grade.

Gravity System
Describe the gravity system in more detail, with typical dimensions and material strengths. Describe if gravity system components are intended to resist seismic actions as part of the seismic-force resisting system.

5.3 Substructure

Basement Levels
Describe basement levels in more detail, with typical dimensions and material strengths. For sloped sites, describe how the site slopes. Provide plans/sections that schematically describe unique considerations.

5.4 Foundation System

Describe the foundation system for the tower and podium including details, dimensions, and material strengths.

Geotechnical Investigations and Reports
Reference geotechnical investigations undertaken by the project geotechnical engineer and provide a reference to their report. Specify if site-specific information is being used in the design of the building.

5.5 Code Analysis and Design Criteria

Building Codes, Standards, Regulations and Computer Software

Building Codes, Standards and Regulations
List all codes progressing from local to national. Also, list non-code sources of information that are directly used in the design. Examples would include supporting publications of nonlinear material/component behavior and their acceptance criteria.

Computer Software

List all software, versions used, and structural elements designed with them.

Code Exceptions

State the specific section(s) of governing code that are excepted, if any. It is best to specifically quote the appropriate portions of code. Include a brief justification for how the exception is justified.

In jurisdictions where PBSD is accepted, there are typically clear design steps and criteria that are expected to be followed. Describe those requirements here and how they are satisfied. If multi-step sequencing is used, describe this sequence in general and how each step leads to the next.

Provide a description of the scope of peer review. This may already be specified by the governing jurisdiction if PBSD is already adopted.

Performance Objectives

Provide a table that describes the intended building performance. The table may specify actions, as in some cases a single element may have different objectives for different actions, such as shear walls. An example is found in Table 3.3.

If performance of non-structural components varies from the governing building code, specify their performance, including cladding, partitions, elevators, exit stairs, etc.

Structural Gravity Load Criteria

Include a summarized version of gravity loading criteria for typical floors and conditions. This helps the document stand alone. Not all gravity load criteria need to be stated, but the exterior wall should be included.

Structural Lateral Load Criteria

Provide a summary of seismic- and wind-code-based load criteria, listing all key code values. For wind, include all key parameters, such as exposure category and basic wind speed. For seismic, include all parameters used to calculate code base shear. Include response modification factors such as R and Ω_o.

Seismic Loads Utilizing Site-Specific Response Spectra and Ground Motions

Describe the site-specific seismic information provided by the geotechnical engineer and a brief description of their methods. The geotechnical engineer should help develop this text, or text should be adopted from the geotechnical engineer's report. Specify the level of damping assumed in the spectra.

Specify the target spectrum for the selection and development of ground motions. If target spectra are other than MCE_R, such as conditional mean spectrum (CMS), are used, then describe their development. Provide a plot of spectral acceleration versus period with code-based and seismic-specific MCE_R, DE, and SLE design spectra. Also provide a similar plot showing MCE_R and developed CMS target spectra. Examples of these two plots are found in Figure 5.1. Black-and-white should be used, as they may be placed on a drawing sheet.

Structural Materials

Provide a list of all typical materials used, and organize the list by concrete, reinforcement, and structural steel. State the grade, yield strength, and appropriate ASTM designation for reinforcement and steel. State the typical application for each.

5.6 Structural Analysis and Design

The following sections provide appropriate detail for each step in the structural system design and verification process. This information should be conveyed in a logical, sequential manner.

Typically, the initial design of the seismic force-resisting system is based on response spectrum analysis. Then, subsequent verification is conducted using NRHA. The specifics of this process can vary based on the requirements of the jurisdiction, project-specific requirements, and on-going advancements of the PBSD design process.

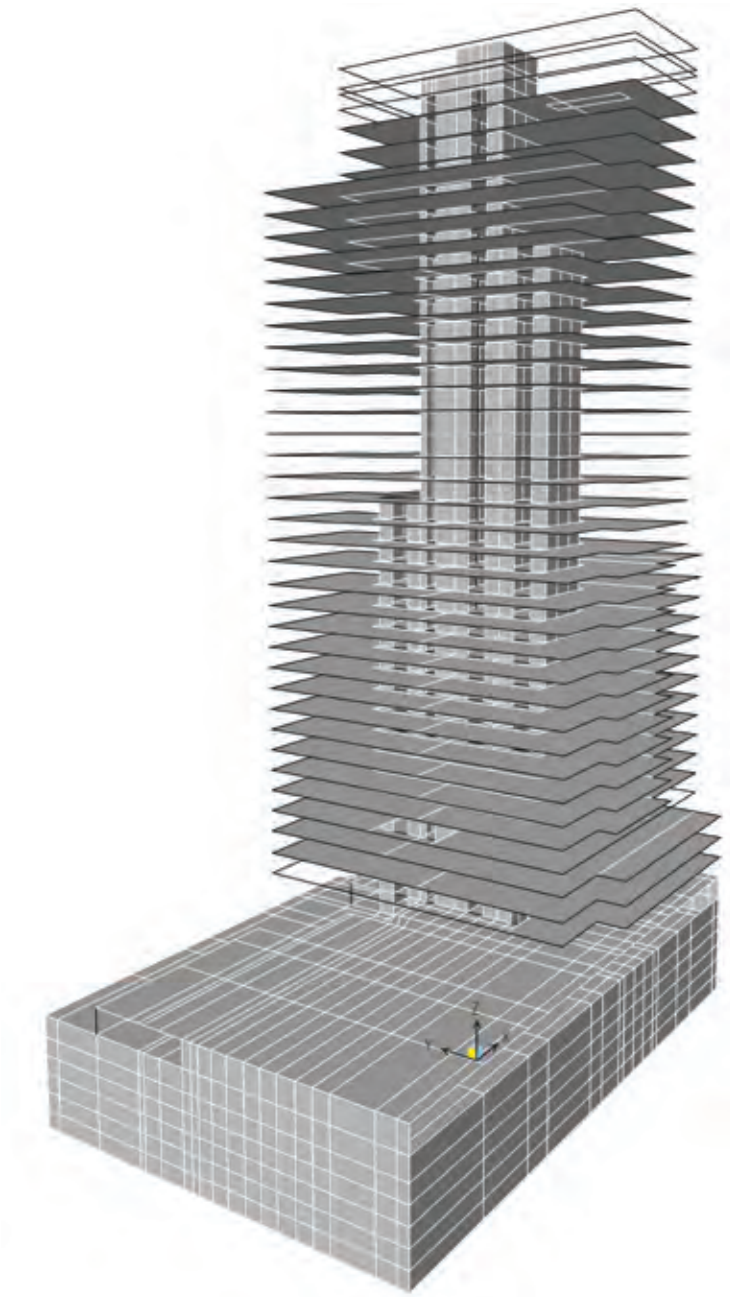
Each step in the described process should include common information such as performance objectives, design criteria, analysis model information (including particulars of nonlinear component modeling), and acceptance criteria of global performance and individual members. This common format will help convey differences between each step in the design and verification process.

The initial member designs using linear analyses are not always reviewed by the peer review panel, but could be very important to the governing jurisdiction looking to ensure a basic standard of care was utilized, similar to code-based buildings. Thus, including the entire design process in this document is advantageous.

Step 1: Design Using Linear Analysis

The purpose of this step is to describe the methods used for design of the seismic force-resisting system. Often, a service-level linear analysis is used with adjusted acceptance criteria to ensure appropriate performance under MCE_R

Design Example 1: Tall High-Occupancy Office Tower



▲ Figure 6.1: Service-Level Analysis Model. © Magnusson Klemencic Associates

Mode Number	Mode Description	SLE Model Period (Seconds)
1	X direction (translation)	4.12
2	Y-direction (translation)	2.45
3	Torsion	2.30

▲ Table 6.1: Service-Level Periods of Vibration.

This design example includes documentation to demonstrate the implementation of a performance-based design approach. A performance-based design approach should follow an approved Basis of Design document that establishes criteria for determining performance acceptability at both a service-level earthquake (SLE) and a maximum considered earthquake (MCE_R). The objectives of the performance in these events are further described in the Basis of Design, which is not provided with this document. The examples included are not intended to present complete or comprehensive results. The example presented is representative of the state of practice at the time of this project's design, and therefore the acceptance criteria or other metrics will vary from current practice depending on when the project was designed, project type, jurisdiction, design guidelines referenced, and peer reviewers. This example focuses on extracts from PBSD reports that highlight specific element types, show effective formats for presenting results, and display acceptance criteria evaluations that occurred for this design.

This design example is for a 525-foot (160-meter), 36-story office tower (see Figure 6.1). The design consists of a reinforced concrete central core, with floor framing consisting of steel floors with composite steel decking. This example highlights a building that is designed as Risk Category III due to the number occupants, and, therefore, with acceptance criteria modified accordingly.

6.1 Serviceability Event Analysis and Verification

Evaluation at the service-level earthquake (SLE) is required in order to quantify response characteristics that relate to the serviceability performance of the structure. These characteristics are

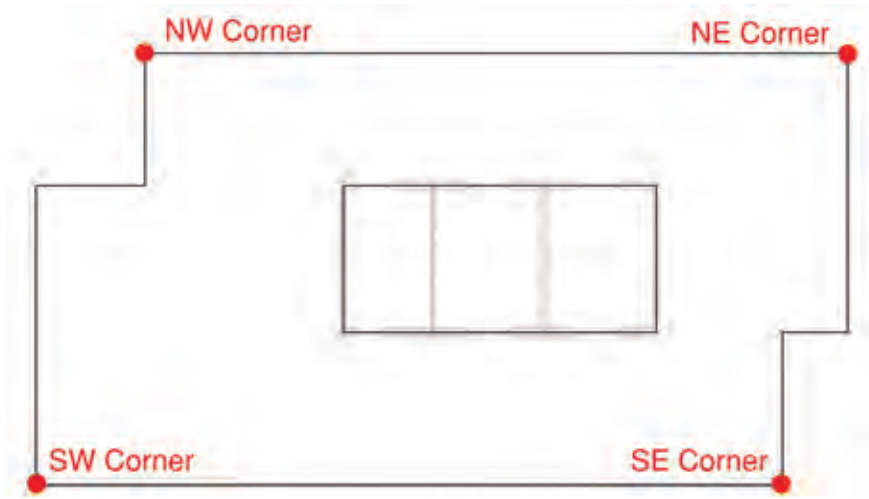
items such as story drift, coupling beam demands, and shear wall demands. The acceptance criteria for this serviceability-level event are selected to encourage essentially elastic behavior of the elements under consideration.

6.2 Elastic Model Description

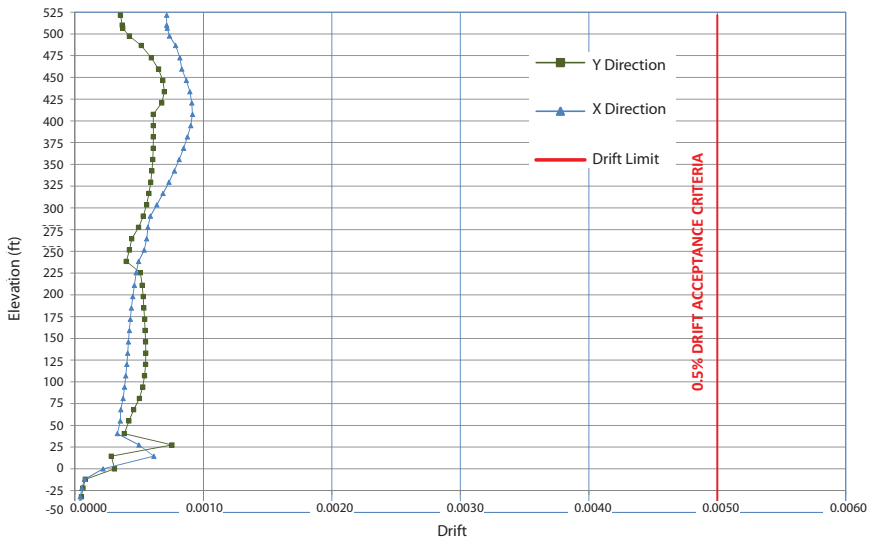
The analysis model used for the service-level verification is a three-dimensional, linear elastic, 2.5% damped mathematical model. Walls and slabs are modeled as elastic shell elements. Columns are modeled as frame elements. Torsion in the model is limited to inherent torsion; accidental torsion is not included. Cracked section properties are included in the model as described in the Basis of Design document. The analysis performed is a modal response spectrum analysis procedure, including a minimum mass participation of 90% of the seismic mass, with the results combined using the complete quadratic combination (CQC) method. The seismic mass includes the building's estimated self-weight, the superimposed dead load, and any live load required by ASCE 7 to be included, such as mechanical equipment and a portion of storage loads. Mass is only assigned above the seismic base (ground level in this case).

Acceptance Criteria: Story Drift
Story drift is a measure of the building deformations under the SLE event. By placing a limitation on building drift, damage of nonstructural elements (such as cladding, wall partitions, etc.) can be limited. The three-dimensional lateral analysis model includes the stiffness modification parameters identified above, which are consistent with the anticipated behavior at SLE.

For the design example, the acceptance criteria value for drift at SLE is 0.5 percent. The full SLE response spectrum is applied with no scaling and no accidental torsion. Story drift is calculated at each corner of the building by taking the difference in elastic displacement of adjacent floors divided by the story height. Story drift is calculated on a per-corner basis in order to correctly include the effects of inherent torsion and the rotational response. An example of the corner points considered is identified in Figure 6.2. Many software analysis tools have the ability to directly output story drift. The diagram in Figure 6.3 indicates the maximum story drift recorded for all four corners considered plotted over the building height. The story drift reported is substantially less than the acceptance criteria; therefore, the story drift is considered acceptable.

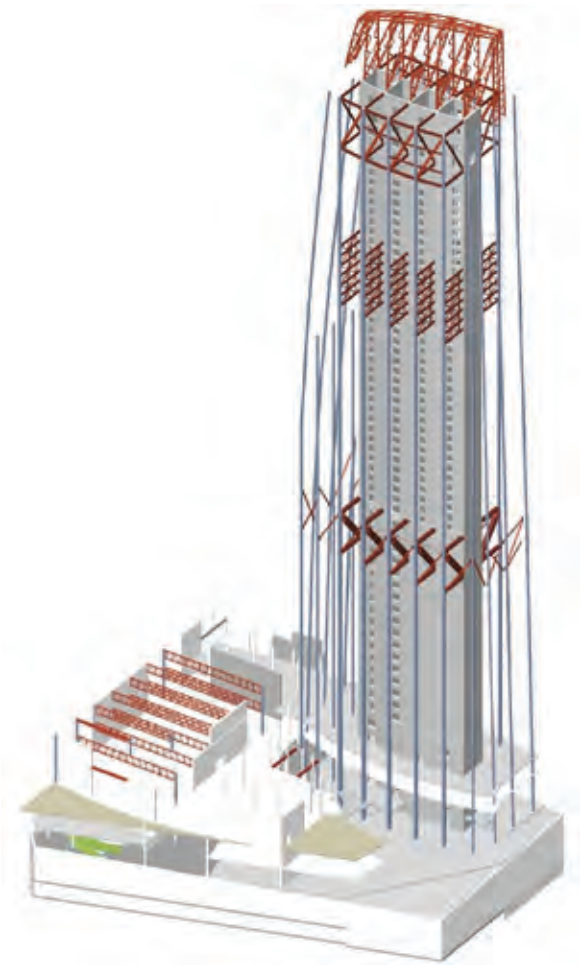


▲ **Figure 6.2: Example Corner Points for Drift Determination.** © Magnusson Klemencic Associates



▲ **Figure 6.3: Max SLE Seismic (Site-Specific) Story Drift.** © Magnusson Klemencic Associates

Design Example 2: Supertall Mixed-Use Tower



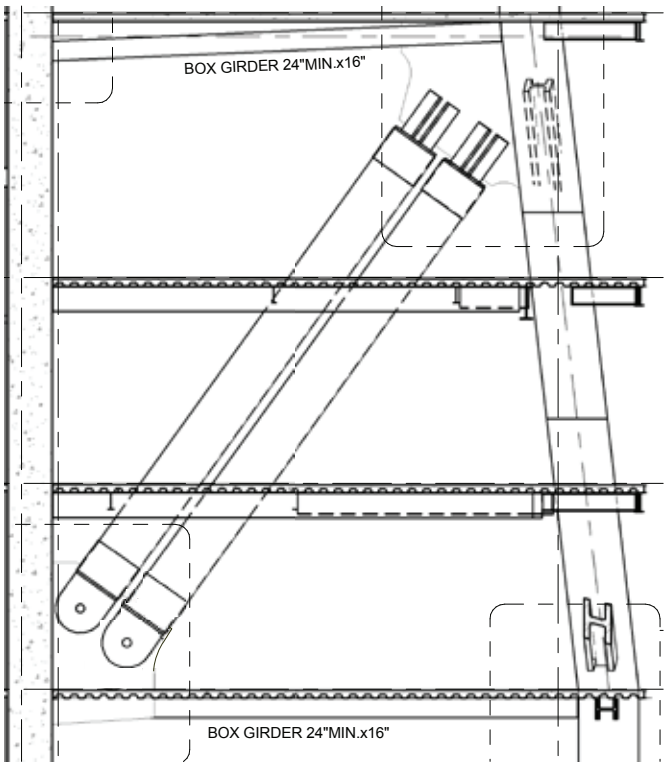
▲ Figure 6.33: Isometric 3D Model with Outriggers at Three Levels. © AC Martin

6.5 Project Overview

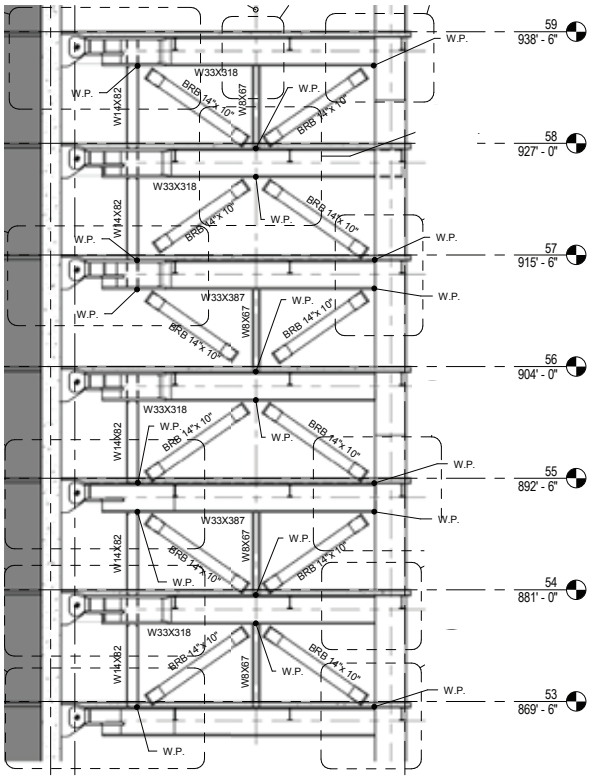
PBSD modeling approaches, acceptance criteria, and results summaries can be used for steel-framed structures, composite structures, and mixed-material structures. More economical and appropriate designs and details can be based on the better understanding of member demands from PBSD. For consistency with the previous example, another high-rise building is used to illustrate this point. Its structural framing incorporates both composite elements (concrete-filled steel box columns) and mixed construction. The tower has long, narrow floors and a slender central concrete core to suit its primary function as a hotel. Perimeter columns are composite, rectangular steel box columns with high-strength concrete fill to minimize impact on hotel room windows. Three sets of outrigger braces provide occupant comfort during windy conditions and safety during earthquakes (see Figure 6.33). A rooftop "sail" feature and a tapered spire (not shown) are other major steel elements.

PBSD using NRHA through the analysis program Perform-3D was selected to show acceptable performance of the proposed structural system. Prescriptive seismic code provisions do not include core-and-outrigger systems, and prescriptive seismic code provisions for buildings of this height would require a dual system (special core and special moment frames) that would not satisfy building performance criteria. Meeting the provisions of the Los Angeles Tall Buildings Structural Design Council (LATBSDC) PBSD guidelines was considered sufficient to demonstrate acceptability to the Los Angeles building department and its peer review panel. Buckling-restrained brace (BRB) diagonals at all outrigger levels provide stable cyclic nonlinear behavior and limit forces generated at columns, connections, and core walls. At the lowest set of outriggers, each diagonal is composed of four individual BRBs extending through three stories. The group of four BRBs provides exceptionally large capacity while using individual elements in sizes that have already been tested. The middle outriggers have an unusual "X-braced Vierendeel" configuration, discussed further in this section. The top three-story-tall outriggers are pre-loaded by jacks to address long-term differential shortening between the concrete core and concrete-filled steel perimeter box columns. The outrigger connection details are complex in order to handle large forces and deformations, but were developed with contractor input to enable practical construction (see Figures 6.34 and 6.35).

The middle outriggers occur roughly 2/3 of the way up the tower and extend vertically through six stories, from level 53 to level 59. These outriggers form "X-braced Vierendeel trusses," which are concealed within the walls separating hotel rooms. To keep the hotel corridor clear, a three-foot (0.9-meter) deep steel girder extends from the outrigger braces to a pinned connection at the concrete core wall at each floor level, shown in Figure 6.35. Steel posts share forces among girders, but some posts are omitted to minimize the restraint



▲ Figure 6.34: Bottom Outrigger Extending Through Three Stories. © Thornton Tomasetti



▲ Figure 6.35: Middle Outrigger Using Deep Girders to Cross Corridors. © Brandow & Johnston

of long-term shortening on core-wall concrete within the outrigger height.

Steel belt trusses at the bottom and top outrigger levels, visible in Figure 6.36, link the 10 outrigger columns to all perimeter tower columns. By engaging all 20 perimeter columns, the stiffness of the lateral load-resisting system is maximized, and differential vertical movements between columns are minimized. Belt trusses also act as "virtual (or indirect) outriggers," reducing tower deflections in the long direction. The load path to accomplish this is described later.

General Steps in the PBSD Process

Step one in the design process was sizing core walls, columns, and outriggers for strength-level forces based on a 1,700-year mean recurrence interval (MRI), ASCE 7-10 wind loads based on the building's high occupancy, and checking dynamic properties for occupant comfort using 10-year MRI wind informed by local wind climate data.

Step two was estimating structural-component seismic demands using linear elastic response-spectrum analysis (RSA) results, scaled up to provide base shear of at least 85% of the minimum base shear equations in the prescriptive code. In this case, equivalent lateral force for the design earthquake level, as used for conventional analysis and design, governed over spectrum-based shear. This was done by considering a building period of seven seconds in the transverse (north–south) direction and 3.5 seconds in the longitudinal, east–west direction, and local high seismicity. Note that in regions of low seismicity, the minimum base shear equations may not govern over the spectrum-based shear value. For overturning, scaled RSA results were used in preliminary checks. For shears, scaled RSA results were tripled because higher modes are major sources of story shear (but not overturning) in tall flexible buildings. That behavior is not addressed well by code methods that directly relate overturning and shear as appropriate for more common, shorter buildings, and because a 1.5 factor is applied to mean results for force-controlled shear checks. Subsequent NRHA results confirmed the reasonableness of this approach.

Design Example 3: Tall Flat-Plate Office Tower

▲ **Figure 6.62: Photos of Completed Building.** © Skidmore, Owings & Merrill LLP

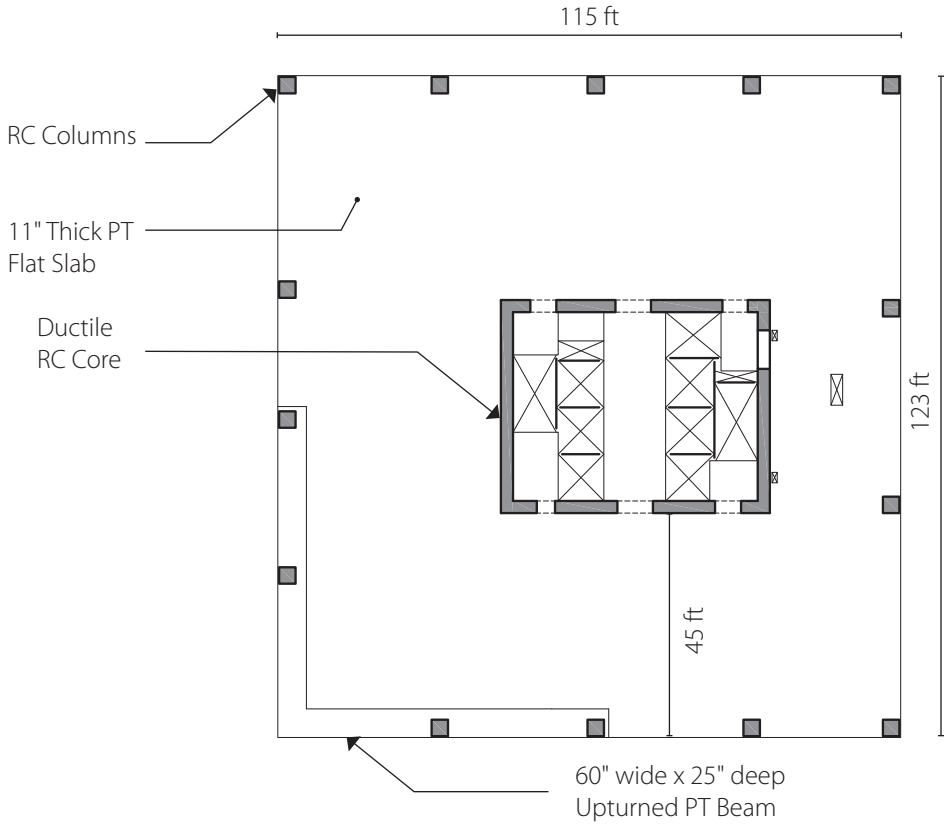
6.15 Project Description

The following is a structural system description for a tower situated in downtown San Francisco, California (see Figure 6.62). The occupancy consists of office above grade and parking below grade. The 30-story tower is 384 feet and two inches above grade with three basement levels below grade and a total building area of approximately 455,000 square feet. The seismic force-resisting structural system consists of reinforced concrete core walls from the foundation to roof. The gravity system is a long-span, flat plate, post-tensioned system. This combination of lateral and gravity systems is common in high-rise residential construction, but has been adapted in an efficient manner for high-rise office application.

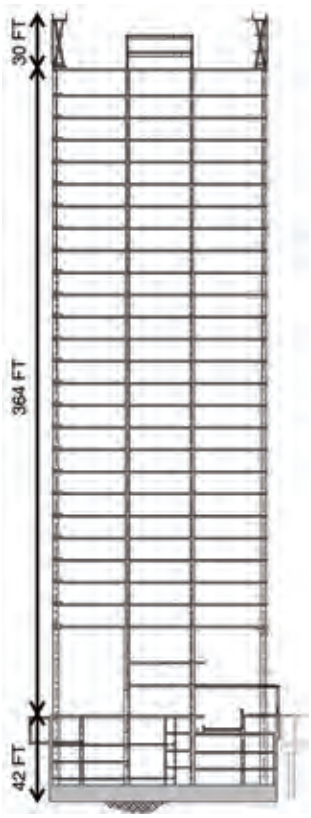
Superstructure
The superstructure consists of a central reinforced concrete shear-wall core, perimeter gravity columns, and two-way flat-plate slab framing. The tower is roughly square in plan with dimensions of 125′–0″ x 130′–0″. The typical office floor-to-floor height is 13′-2″.

Lateral System
The lateral system consists of a centrally located reinforced concrete shear-wall core. The shear wall core has an external plan area of 43′–0″ x 52′–6″ and is located around the service area of the structure, passenger and service elevators, and back-of-house areas. The height-to-depth ratio of the core is approximately 9:1. The shear wall core extends from foundation to roof. The shear walls vary in thickness from 24″ to 33″ and in concrete compressive strength from 6,000 psi to 8,000 psi. The shear wall core is interconnected with the use of ductile diagonally-reinforced link beams at openings required for doorways and corridors.

Gravity System
The gravity framing system both inside and outside the core consists of a two-way post-tensioned (PT) flat-plate slab. The slab clears spans from the core to the perimeter and has a uniform thickness of 11″. The



▲ **Figure 6.63: Typical Floor Plan.** © Skidmore, Owings & Merrill LLP



▲ **Figure 6.64: Building Section.** © Skidmore, Owings & Merrill LLP

long-span flat slab is post-tensioned to maintain uncracked section under service gravity loads, and cambered for a portion of the long-term deflection. The slab utilizes concrete with a compressive strength of 5,000 psi. Due to the long-span condition, a digitally mapped camber program was used to ensure deflections would meet tenant requirements. The perimeter vertical gravity columns are typically composed of conventional reinforced concrete sections, varying in size from 42″ x 42″ square to 26″ x 26″ square. The columns utilize concrete with compressive strengths ranging from 6,000 psi to 8,000 psi. The tall columns at the entry lobby consist of 42″ x 42″ composite members utilizing steel cruciform shapes embedded within the concrete column. The southwest corner is double-cantilevered 30 feet with the use of upturned post-tensioned beams (see Figure 6.63).

Substructure

Vertical elements of the superstructure continue down through the substructure to the foundation. Shear walls are 24″–33″ thick, with a compressive strength of 8,000 psi. The columns are typically 36″ x 48″ and consist of 8,000 psi concrete. The gravity system in the substructure consists of a 10″ thick, conventionally reinforced, two-way flat-plate slab that utilizes concrete with a compressive strength of 5,000 psi.

Foundations

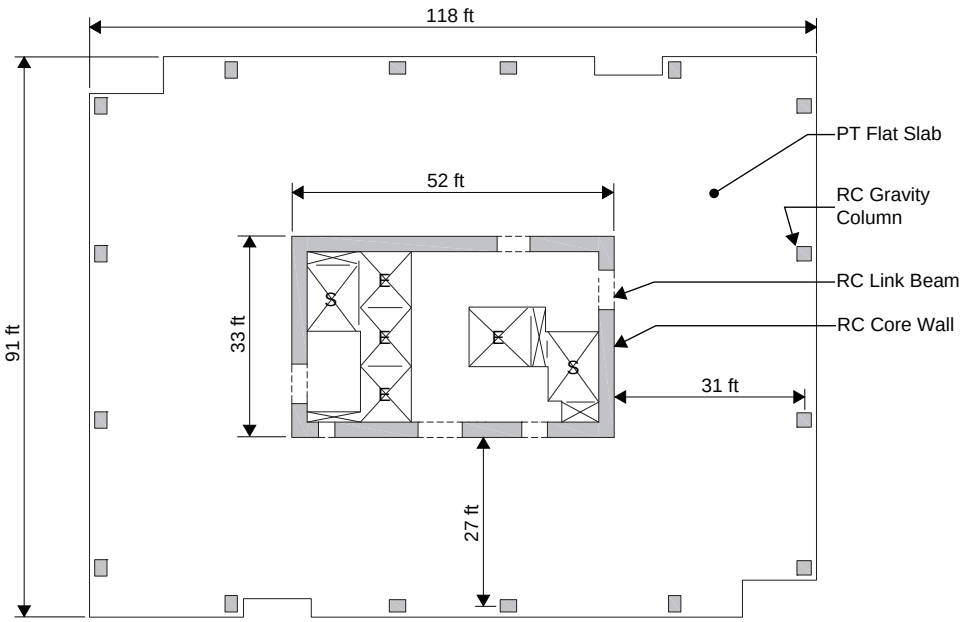
The foundation system consists of a 10′–0″ thick, conventionally-reinforced concrete mat foundation (see Figure 6.64). A perimeter reinforced concrete foundation wall system consists of conventional 16″–22″ thick cast-in-place concrete walls. The substrate consists of dense sands over pre-consolidated clay.

6.16 Design Criteria

The building was designed under the San Francisco Building Code SFBC 2010, which refers to the California Building Code (2010) and ASCE 7 (2005). The CBC is adopted from previous IBC publications with amendments specific to California.

Design Example 4: Tall Residential Tower with Podium

▲ **Figure 6.77: Rendering.** © Skidmore, Owings & Merrill LLP



▲ **Figure 6.78: Typical Floor Plan.** © Skidmore, Owings & Merrill LLP

6.20 Project Description

The following is a structural system description for a residential tower situated in downtown San Francisco, California (see Figure 6.77). The occupancy consists of apartment residential units above grade, and parking below grade. The 42-story tower is 420′–0″ tall above grade with six basement levels below grade and a total building area of approximately 743,500 square feet. The seismic force-resisting structural system consists of reinforced concrete core walls from the foundation to roof. The gravity system is a long-span, flat-plate, post-tensioned system. This combination of lateral and gravity systems is common in high-rise residential construction.

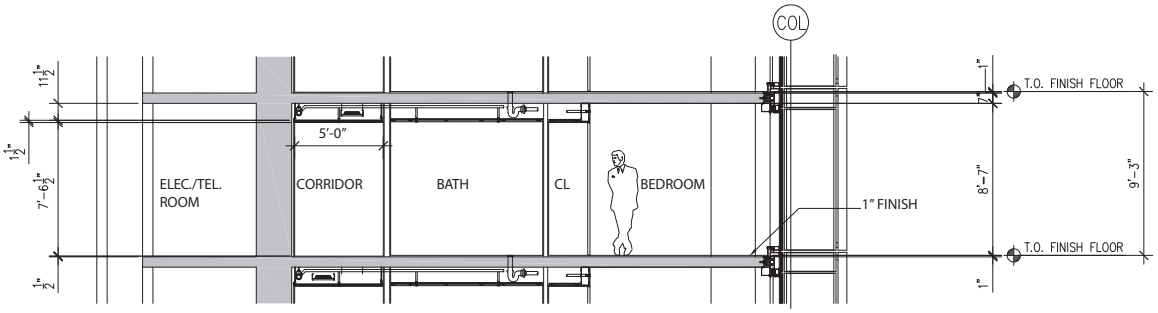
Superstructure
The superstructure consists of a central reinforced concrete shear wall core, perimeter gravity columns, and two-way flat-plate slab framing (see Figure 6.77). The tower is roughly square in plan with dimensions of 91′–0″ x 118′–0″ and a larger podium. The typical residential floor-to-floor height is 9′–3″ (see Figure 6.78). The podium is exceptionally large for similar buildings in San Francisco at nine floors, and plays a significant role in building's response to lateral loading.

With the removal of the code-prescribed moment frame, the improvement of the typical floor section is demonstrated in Figures 6.79 and 6.80. By removing the moment frame, inefficient material is removed, floor-to-floor height is reduced, and constructability is improved.

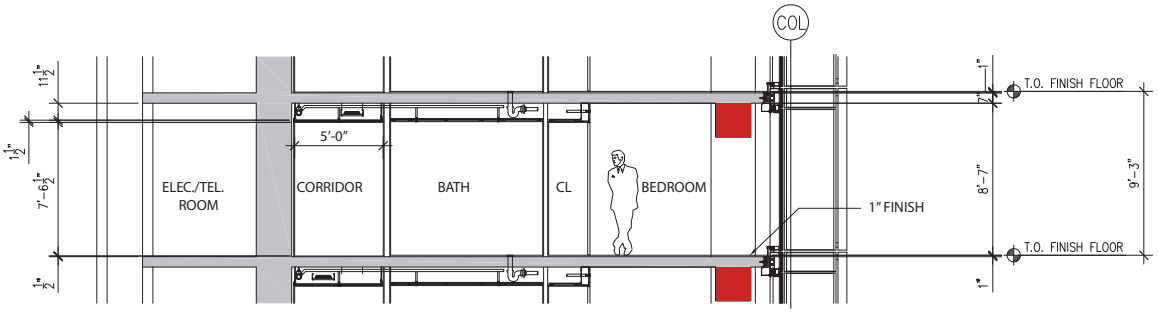
Lateral System
The lateral system consists of a centrally-located reinforced concrete shear wall core. The shear wall core has an external plan area of 33′–0″ x 52′–0″ and is located around the service area of the structure, passenger and service elevators, and back-of-house areas. The height-to-depth ratio of the core is 12.1 to 1. This is a very slender application of a core-only lateral system approach. The shear wall core extends from foundation to roof. At podium levels, additional shear walls are included to assist the core, due to the added mass and eccentricity. The shear walls vary in thickness from 36″ to 24″, and in concrete compressive strength of 8,000 psi at the core to 6,000 psi at the podium. The shear wall core is interconnected with the use of ductile diagonally-reinforced link beams at openings required for doorways and corridors.

Core shear wall elevations are shown in Figure 6.81, where the darker shading indicates confined boundary zones.

Gravity System
The gravity framing system both inside and outside the core consists of a two-way post-tensioned (PT) flat-plate slab. The slab clear-spans from the core to the perimeter and has a uniform thickness of seven inches. The long-span flat slab is post-tensioned to maintain uncracked



▲ Figure 6.79: PBSD Core-Only Approach. © Skidmore, Owings & Merrill LLP



▲ Figure 6.80: Code-Prescribed Dual System Approach. © Skidmore, Owings & Merrill LLP

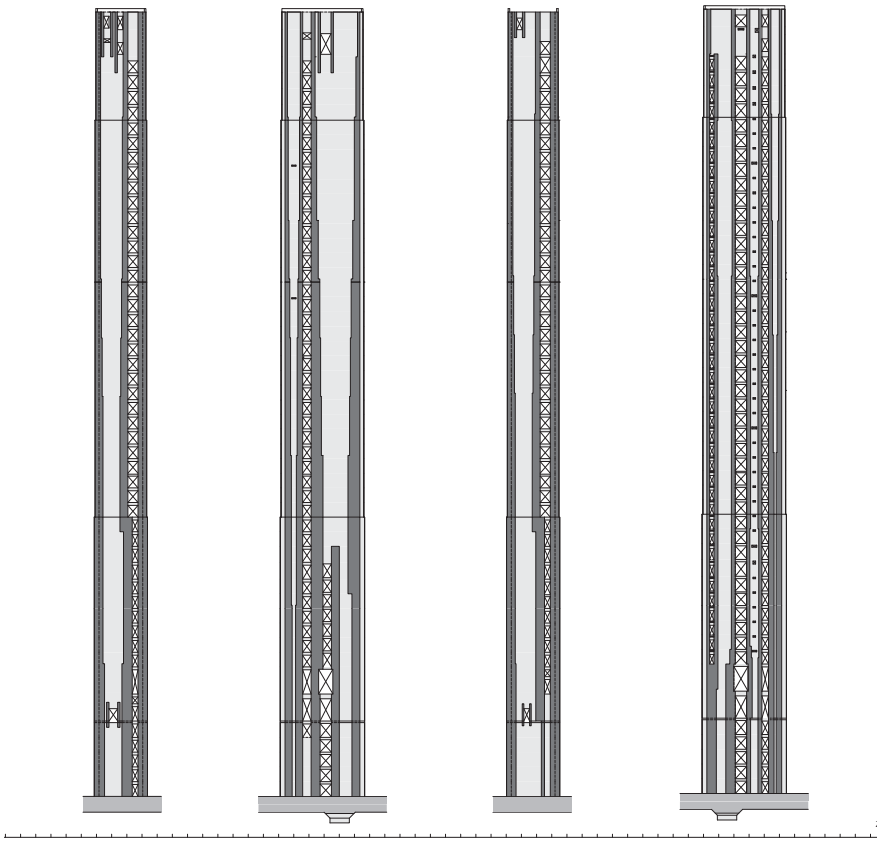
section under service gravity loads. The slab utilizes concrete with a compressive strength of 6,000 psi.

Substructure

The vertical elements of the superstructure continue down through the substructure to the foundation. The gravity system in the substructure consists of a 10" thick, conventionally reinforced, two-way flat-plate slab that utilizes concrete with a compressive strength of 6,000 psi.

Foundations

The foundation system consists of a 10'–0" thick conventional reinforced concrete mat foundation. A perimeter-reinforced concrete foundation wall system consists of conventional cast-in-place concrete walls. The substrate consists of dense sands over pre-consolidated clay. Due to soils that could potentially "liquefy" in a seismic event, jet-grouted soil improvement columns were specified by the geotechnical engineer.



▲ Figure 6.81: Shear Wall Elevations: West, North, East, and South (from left to right). © Skidmore, Owings & Merrill LLP

Performance-Based Seismic Design (PBSD) is a structural design methodology that has become more common in urban centers around the world, particularly for the design of high-rise buildings. The primary benefit of PBSD is that it substantiates exceptions to prescribed code requirements, such as height limits applied to specific structural systems, and allows project teams to demonstrate higher performance levels for structures during a seismic event.

However, the methodology also involves significantly more effort in the analysis and design stages, with verification of building performance required at multiple seismic demand levels using Nonlinear Response History Analysis (NRHA). The design process also requires substantial knowledge of overall building performance and analytical modeling, in order to proportion and detail structural systems to meet specific performance objectives.

This CTBUH Technical Guide provides structural engineers, developers, and contractors with a general understanding of the PBSD process by presenting case studies that demonstrate the issues commonly encountered when using the methodology, along with their corresponding solutions. The guide also provides references to the latest industry guidelines, as applied in the western United States, with the goal of disseminating these methods to an international audience for the advancement and expansion of PBSD principles worldwide.

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