PERFORMANCE-BASED SEISMIC DESIGN OF TALL BUILDINGS IN THE U.S.

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ABSTRACT:

Building codes in the United States contain prescriptive requirements for seismic design as well as an option for use of alternative provisions. Increasingly these alternative provisions are being applied for the performance-based design of tall buildings. Application of performance-based procedures requires: An understanding of the relation between performance and nonlinear response; selection and manipulation of ground motions appropriate to the seismic hazard; selection of appropriate nonlinear models and analysis procedures; interpretation of results to determine design quantities based on nonlinear dynamic analysis procedures; appropriate structural details; and peer review by independent qualified experts to help assure the building official that the proposed materials and system are acceptable. Both practice- and research-oriented aspects of performance-based seismic design of tall buildings are presented.

1 INTRODUCTION

The west coast of the United States, a highly seismic region, is seeing a surge in the design and construction of tall buildings (defined here as buildings 240 feet (73 meters) or taller). Many of these buildings use high-performance materials and framing systems that are not commonly used for building construction or that fall outside the height limits of current buildings codes. In many cases, prescriptive provisions of governing building codes are found to be overly restrictive, leading to pressures to design outside the limits of the code prescriptive provisions. This is allowable through the alternative provisions clause of building codes. An example of the alternative provisions clause of the San Francisco Building Code [SFBC, 2001] is reproduced below:

> ***104.2.8 Alternate materials, alternate design and methods of construction.*** *The provisions of this code are not intended to prevent the use of any material, alternate design or method of construction not specifically prescribed by this code, provided any alternate has been approved and its use authorized by the building official.*
>
> *The building official may approve any such alternate, provided the building official finds that the proposed design is satisfactory and complies with 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 suitability, strength, effectiveness, fire resistance, durability, safety and sanitation.*
>
> *The building official shall require that sufficient evidence or proof be submitted to substantiate any claims that may be made regarding its use. The details of any action granting approval of an alternate shall be recorded and entered in the files of the code enforcement agency.*

Increasingly, nonlinear dynamic analysis is being used in a performance-based approach to demonstrate the seismic performance of buildings that do not satisfy all the prescriptive provisions of the building code. While guidelines [FEMA 356, 2000; LATB 2006; SEAONC 2007] and code requirements [ASCE 7, 2005; IBC, 2003; UBC, 1997] exist, there still remain many undefined aspects of nonlinear performance-based design for which additional guidance would be helpful. Recent and ongoing research is providing some of the answers, but many issues in performance-based design remain to be resolved. This paper describes performance-based design of tall buildings as it is commonly practiced and highlights research opportunities.

2 THE NEW GENERATION OF TALL BUILDINGS IN THE WESTERN US

Figure 1 - San Francisco buildings built since 1999, under construction, approved, and proposed (after Steve Boland).

Urban regions along the west coast of the United States are seeing a boom in tall building construction. Figure 1 illustrates recent developments and plans in San Francisco, where buildings as tall as 1200 ft (370 m) are under study. Many of the buildings are residential or mixed-use (including residential) occupancy. Functional requirements for tall residential buildings have led to new building configurations and systems that do not meet the prescriptive definitions and requirements of current buildings codes. These include efficient framing systems whose redundancy may be reduced compared with more conventional buildings. High-strength materials and specialized products are also being proposed to help meet the unique challenges introduced by these structures.

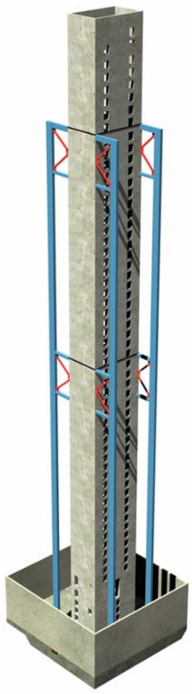


Figure 2 – Schematic of seismic-force- resisting system of 60-story building in San Francisco (Magnusson - Klemencic)

Seismic design of tall buildings introduces many design challenges. As an example, Figure 2 illustrates the “seismic” system of a 60-story building currently under construction in San Francisco, in which the seismic force-resisting system is reinforced concrete core walls with buckling-restrained steel outrigger braces along one axis. The building exceeds prescriptive building code height limits for core-wall-only systems by a factor of about three. In this building the gravity framing comprises unbonded post-tensioned flat-plate framing (not shown). Under earthquake ground shaking, this gravity framing, while not designed as part of the primary seismic-force-resisting system, will undergo lateral displacements and will accumulate over its height significant axial forces owing to unintended outrigger action. The unintended frame action also will result in wall shears higher than those expected for a wall-only system. These effects should be considered in design. Other framing systems including moment frames, steel-plate walls, and steel gravity frames mixed with concrete walls are being considered for various buildings, each with its own special design needs.

Other special conditions may apply. High occupancy levels and interest in re-occupancy following an earthquake are leading to rethinking of performance objectives. As a minimum, a building must be safe for rare ground shaking demands, and must remain safe for significant aftershocks. However, there is increasing concern that serviceability for more frequent events should be considered as well. In view of the very long vibration periods of tall buildings, special treatment of design ground motions is needed to ensure they are representative in their damage potential, including proper duration and long-period energy content. While equivalence to building code minimum performance requirements generally is the basic objective, there is no consensus on how to translate that objective into specific engineering demands and capacity checks in a performance-based procedure.

3 ANTICIPATED BUILDING RESPONSE

With the exception of special high-performance buildings and buildings with special protective systems, it usually is not economically feasible to design a building to remain fully elastic for ground motions representative of the maximum considered hazard level in regions of high seismicity. Therefore, some nonlinear behavior should be anticipated during design and analysis.

The design strength of a building satisfying the U.S. code prescriptive provisions is established based on the forces that would occur for linear seismic response divided by a force reduction factor *R*. Thus, the value of *R* used in design provides a measure of the degree of nonlinearity expected during a design earthquake. For a tall building, however, the required strength generally is controlled by minimum base-shear requirements. Consequently, the effective *R* value is reduced from the value specified in the building code for that framing system to a smaller value dependent on building period and other factors. Figure 3 illustrates design base shears from the Uniform Building Code [UBC, 1997] for a typical site under development in San Francisco.

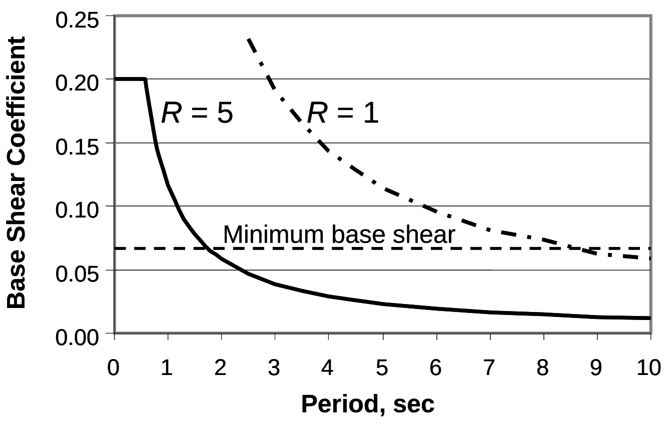


Figure 3 – UBC-97 required design base shear for typical development site in San Francisco, site class C. The specified value of *R* is around 5, but the effective reduction from linear response (*R* = 1) is limited by the minimum base shear.

The Structural Engineers Association of Northern California has issued a guideline for the performance-based seismic design of buildings designed to the "alternative provisions" clause of the San Francisco Building Code [SEAONC 2007]. As adopted by the City of San Francisco, this guideline requires a serviceability evaluation for ground motion levels having 43-year mean return period; at this level minor nonlinear building response is acceptable provided damage will be cosmetic and not require major repairs. Structural and nonstructural systems must be designed to be safe for ground motion levels having 475-year mean return period (10% probability of exceedance in 50 years), and the structural system is required to remain stable with maximum interstory drifts not exceeding 0.03 for ground motions corresponding to the larger of 10% probability of exceedance in 100 years or 1.5 times median deterministic motions.

4 GROUND MOTIONS FOR NONLINEAR ANALYSIS

Seismic ground shaking for performance-based design of tall buildings usually is represented using site-specific seismic hazard analysis. Seismic hazard due to ground shaking should be determined considering the location of the building with respect to causative faults, the regional and local site-specific geologic characteristics, and the selected earthquake hazard level. In general, the seismic hazard should include earthquake-induced geologic site hazards in addition to ground shaking. The discussion here is limited to ground shaking hazard.

Where nonlinear dynamic analysis is used, representative ground motion records are required. The predominant current practice is to select records from actual earthquake databases (e.g., [PEER GM, 2007]) considering magnitude, distance, site condition, and other parameters that control the ground motion characteristics. To help guide selection of ground motion records, the seismic hazard at the periods of interest is deaggregated for each hazard level to determine the contributions to the hazard from earthquakes of various magnitudes and distances from the site. Because magnitude strongly influences frequency content and duration of ground motion, it is desirable to use records from earthquakes within 0.25 magnitude units of the target magnitude [Stewart, et al., 2001]. Duration can be especially important for tall buildings because of the time required to build up energy in long-period structures. For sites close to active faults, selected motions should contain an appropriate mix of forward, backward, and neutral directivity consistent with the site [Bray and Rodriguez-Marek, 2004].

Once a suite of ground motions has been selected, these commonly are manipulated to represent the target linear response spectrum using either *scaling* or *spectrum matching*. *Scaling* involves applying a constant factor to individual pairs of horizontal ground motion records to make their response match the design spectrum at a single period or over a range of periods. *Spectrum matching* is a process whereby individual ground motion records are manipulated (usually in the time domain by addition of wave packets) to adjust the linear response spectrum of the motion so it matches the target design response spectrum. Resulting motions should be compared with original motions to ensure the original character of the motion is not modified excessively.

There currently is no consensus on which approach, scaling or spectrum matching, is preferable for nonlinear dynamic analysis. The advantage of scaling is that individual ground motion records retain their original character including peaks and valleys in the response spectrum. However, given the long fundamental periods characteristic of tall buildings, it can be difficult to find records with sufficient energy in the long-period range, therefore requiring relatively large scaling factors that may result in unrealistic short-period response. This procedure, therefore, if is not done appropriately, may excessively excite higher-mode responses. Spectrum matching can alleviate the aforementioned problem with scaling, but matching the uniform hazard spectrum at every period also produces ground motions that are unrealistic and may be excessively demanding.

Alternative approaches to ground motion selection and scaling have been proposed. Abrahamson [2006] recommended that the ground motion should represent a scenario earthquake, that is, just one of the many earthquakes that contribute to the uniform hazard spectrum at the site. In this case, the selected motion is matched to the response spectrum of the scenario earthquake, which is less broadband than the uniform hazard spectrum (Figure 4). Baker [2006] and Cornell [2006] have recommended that the scenario spectrum should be modified to represent the conditional mean response spectrum, which takes into account the lack of correlation between response spectral amplitudes at different periods. These methods imply a deaggregation of the seismic hazard to identify which earthquakes have the highest contribution to the seismic hazard at a key vibration period of the building. A problem that arises with a tall building is that different engineering demands are controlled by different periods (for example, overturning moment might be controlled by the fundamental period while link beam demands are controlled by higher "modes"). Scenario-based ground motions currently are not widely used for tall building designs because of these uncertainties.

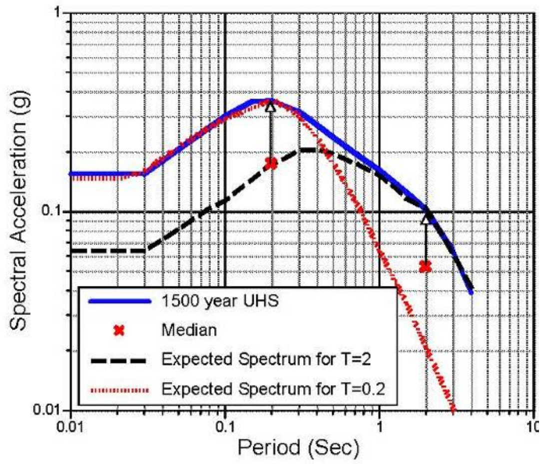


Figure 4 – Uniform hazard spectrum and scenario spectra that combine to create the envelope of the uniform hazard spectrum [Abrahamson, 2006].

5 ANALYSIS FOR TALL BUILDINGS

Performance-based seismic analysis of tall buildings in the US increasingly uses nonlinear analysis of a three-dimensional model of the building. Lateral-force-resisting components of the building are modeled as discrete elements with lumped plasticity or fiber models that represent material nonlinearity and integrate it across the component section and length. Gravity framing elements (those components usually not designated as part of the lateral force resisting system) increasingly are being included in the nonlinear models so that effects of building deformations on the gravity framing (imposed deformations and the accumulation of overturning forces over the building height) as well as effects of the gravity framing on the seismic system (for example, increased stiffness and core-wall shears) may be represented directly.

Because the behavior is nonlinear, behavior at one hazard level cannot be scaled from nonlinear results at another hazard level. Furthermore, conventional capacity design approaches can underestimate internal forces in some structural systems (and overestimate them in others) because lateral force profiles and deformation patterns change as the intensity of ground shaking increases [Kabeyasawa, 1993; Eberhard and Sozen, 1993]. Figure 5 shows statistics of story shear profiles for a tall core-wall building subjected to different levels of earthquake ground motion. (Twenty-seven records from earthquakes having ~M7 at ~10km were scaled to the first-mode spectral acceleration of the uniform hazard spectrum for 475-year (DBE) and 2475-yr (MCE) mean return periods.) The story shear has considerable dispersion for any level of ground shaking, and the shear increases as the intensity of shaking increases even though total overturning moment is limited by the wall yielding mechanism. Similar patterns can be observed for upper-story wall moments and other design parameters.

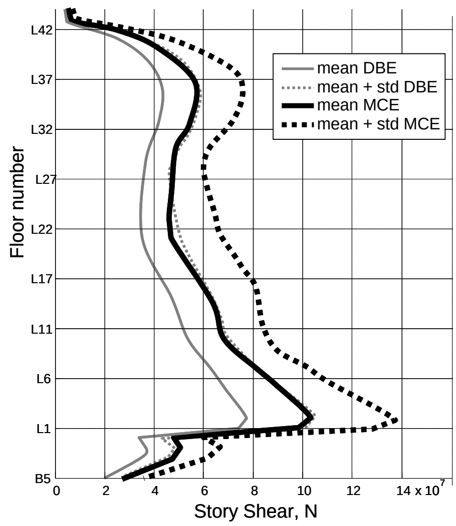


Figure 5 – Calculated story shears for 48-story tall core-wall building. Values shown are mean and mean plus one standard deviation values.

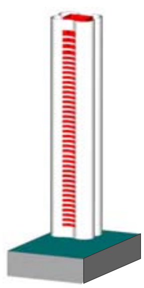
Results of nonlinear dynamic analysis are sensitive to modeling of component load-deformation properties. In reinforced concrete construction, component effective stiffnesses should consider effects of cracking on stiffness and slip of reinforcement from anchorage zones. Some guidance is provided by FEMA 356 [2000] and elsewhere [e.g., Paulay and Priestley, 1992], but the level of guidance is incomplete and commonly aimed at buildings of moderate or lower height. Usual practice is to base nonlinear component strengths on expected material properties. By so doing, the computer model response is likely to be closer to best-estimate response and internal actions (e.g., axial forces, shears, and moments) on components expected to remain elastic will be more conservatively estimated. Based on mill tests of Grade 60 A615 bars in North America, Mirza and MacGregor [1979] report mean yield strength is approximately 1.15 times the nominal value. Strain-hardening commonly results in ultimate strength approximately 1.5 times the actual yield value. Values close to these probably should be used unless specific information regarding reinforcement properties indicates other values are more appropriate.

Damping properties for concrete buildings normally are set around five percent of critical damping for the periods (vibration modes) likely to dominate response. Lower damping values, in the range of two to three percent of critical damping, commonly are used for steel buildings.

6 DESIGN VALUES

For buildings designed using nonlinear dynamic analysis approaches, building codes and design guides [e.g., ASCE 7-05, 2005; LATB, 2006; SEAONC, 2007] require/recommend that a building design be subjected to a series of design-level earthquake time series to determine the building design-level response. A key question, then, is how to define the design-level response when each of the multiple design ground motions produces its own distinct results. SEAONC [2007] recommends that demands for ductile actions be taken not less than the mean value obtained from the nonlinear response history analysis, whereas demands for low-ductility actions (e.g., axial and shear response of columns and shear response of walls) should consider the dispersion of the values. The commentary to the SEAONC guidelines notes that in typical cases design actions for low-ductility actions can be defined as the mean plus one standard deviation of the values obtained from the nonlinear response history analysis.

The choice of design value (mean, median, or one standard deviation higher) has a significant impact on final design quantities. Figure 6 illustrates this for the 48-story core-wall building discussed previously, listing sample statistics for interstory drift in the 37th story, base shear, and wall shear at the 22nd story, determined for ground motions scaled to the MCE uni-



	IDR_{37}	V_{base}	V_{22}
Max	2.25	1.74	2.31
Min	0.44	0.53	0.47
mean (m)	1.00	1.00	1.00
m + σ	1.37	1.32	1.42

Figure 6 – Building elevation and summary of nonlinear dynamic analysis results.

form hazard spectral acceleration at the fundamental period. All response quantities are normalized to the mean value. The coefficients of variation (in this case from 0.32 to 0.42) are typical of values obtained from dynamic analyses with ground motions scaled as described above. Similar values have been obtained for DBE loadings (not shown).

The results of studies such as the one reported in the preceding paragraph can be useful to understand the degree of safety in a building design. For example, one can determine the probability of exceeding the wall design shear for different shaking intensities. In the example described above, if the design shear is set equal to the mean plus one standard deviation value at the MCE level, as suggested by the SEAONC commentary [2007], the probability of exceeding the design shear for ground motions having 475-year mean return periods is approximately five percent. Whether this is an acceptable result has not been widely considered at this time.

Results such as those presented in the preceding paragraphs are promising, but they convey a specious sense of precision that cannot be fully supported by the procedures used. For one, the procedures for selecting and scaling ground motions were essentially arbitrary. For another, uncertainties in the analysis model and in the wall capacity have not been considered. Thus, while these procedures are an improvement over linear prescriptive methods, further improvements are required.

7 STRUCTURAL DETAILING

A significant percentage of high-rise building construction in the western US will be for residential or mixed-use occupancies. As such, much of it will be constructed of reinforced concrete, and most of those will use reinforced concrete core walls.

Under design-level earthquake ground motions, the core wall may undergo inelastic deformations near the base (and elsewhere) in the presence of high shear. Ductile performance requires an effectively continuous tension chord, adequately confined compression zone, and adequate proportions and details for shear resistance. In locations where yielding is anticipated, ACI 318-2005 requires use of Type 2 mechanical splices or tension lap splices designed for $1.25f_y$, where f_y is the nominal yield strength of the longitudinal reinforcement. Furthermore, longitudinal reinforcement is to be extended a distance $0.8l_w$ past the point where it is no longer required for flexure based on conventional section flexural analysis, where l_w is the (horizontal) wall length. ACI 318-2005 specifies minimum requirements for confinement of the compression zone. It is recommended that not less than this amount of confinement be used within potential plastic hinge zones, unless analysis using models validated by experimental data indicate compression strains well below 0.003; computer software for nonlinear earthquake response analysis may produce strain as one of the output quantities, but such outputs can be strongly dependent on modeling assumptions; modeling procedures should be validated (by the engineer of record) against strains measured in laboratory tests. Because ACI 318-05 was written with moderate-rise buildings in mind, it does not include requirements for confinement outside the intended plastic hinge zone at the base. Nonetheless, some confinement (perhaps half that specified in the fully confined zone) usually is provided above the base to accommodate nonlinear demands that may occur due to "higher-mode" effects under severe loading.

Details of transverse reinforcement for shear should include development of the horizontal reinforcement to the far face of the confined boundary zone; otherwise, the full length of the wall is not effective in resisting shear. Figure 7 shows an example detail for boundary element confinement and anchorage of shear reinforcement using headed bars. Another accepted detail is to lap the horizontal shear reinforcement with an equal area of hoops or U-bars inserted into the boundary. Hooks on the horizontal reinforcement may not be feasible given the large diameter of the horizontal bars.

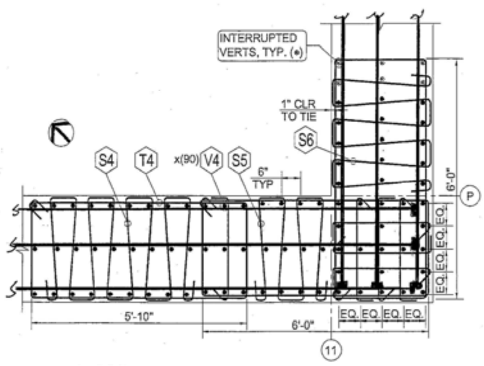


Figure 7 – Wall reinforcement detail.

Coupled core walls require ductile link beams that can undergo large inelastic rotations. In typical cases, the small aspect ratio and high nominal shear stress dictate use of diagonally reinforced coupling beams. To facilitate construction, link beams are now constructed using full cross section confinement rather than individual diagonal confinement (Figure 8). Recent tests have demonstrated that full beam confinement is at least as effective as diagonal-only confinement [ENR, 2007].

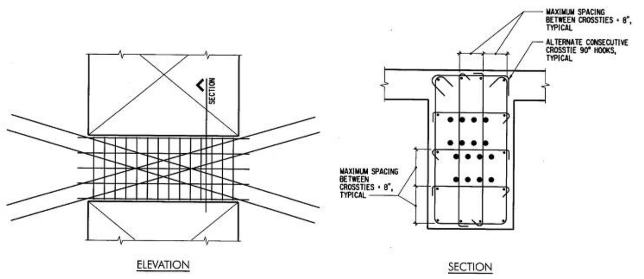


Figure 8 – Confinement alternative for diagonally reinforced beams.

Away from the core walls, gravity loads commonly are supported by post-tensioned floor slabs supported by columns. Slab-column connections are designed considering effect of vertical slab shears on the lateral drift capacity of the connection according to ACI 318 [2005]. In most cases, stud rails or other systems are used to reduce the likelihood of punching around the columns. At least two of the strands in each direction must pass through the column cage to provide post-punching resistance.

8 BUILDING DESIGN REVIEW

Few building departments have the expertise to understand and approve the code exceptions and alternative means proposed in a performance-based design. Questions invariably arise regarding use and performance of new materials and systems, selection of appropriate hazard levels and representative ground motions, nonlinear dynamic analysis models and results interpretations, acceptance criteria, and quality assurance in design and construction. Peer review by independent qualified experts helps assure the building official that the proposed materials and system are acceptable.

9 CONCLUSIONS

Performance-based earthquake engineering increasingly is being used as an approach to the design of tall buildings in the U.S. Available software, research results, and experience gained through real building applications are providing a basis for effective application of nonlinear analysis procedures. Important considerations include definition of performance objectives, selection of input ground motions, construction of an appropriate nonlinear analysis model, and judicious interpretation of the results. Implemented properly, nonlinear dynamic analysis specific to the structural system and seismic environment is the best way to identify nonlinear dynamic response characteristics, including yielding mechanisms, associated internal forces, deformation demands, and detailing requirements. Proportions and details superior to those obtained using the prescriptive requirements of the building code can be determined by such analysis, leading to greater confidence in building performance characteristics including safety. Although performance-based designs already are under way and are leading to improved designs, several research needs have been identified, the study of which can further improve design practices.

10 ACKNOWLEDGMENTS

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