THE STRUCTURAL DESIGN OF TALL AND SPECIAL BUILDINGS
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SEISMIC ANALYSIS, DESIGN, AND REVIEW FOR TALL BUILDINGS

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SUMMARY

Whereas current building codes legally apply to seismic design of tall buildings, their prescriptive provisions do not adequately address many critical aspects. Performance-based engineering provides a desirable alternative. Application of performance-based procedures requires: an understanding of the relation between performance and nonlinear modeling; selection and manipulation of ground motions appropriate to design hazard levels; selection of appropriate nonlinear models and analysis procedures; interpretation of results to determine design quantities based on nonlinear dynamic analysis procedures; careful attention to structural details; and peer review by independent qualified experts to help assure the building official that the proposed materials and system are acceptable. These topics are discussed with an emphasis on tall buildings.

1. INTRODUCTION

A trend in the seismic design of tall buildings is to use performance-based approaches that rely on nonlinear dynamic analysis to simulate expected earthquake response. While guidelines (FEMA 356, 2000; LATB, 2006) and code requirements (ASCE, 2002; IBC, 2003; UBC, 1997) exist, there still remain many undefined aspects for which additional guidance would be helpful. Providing such guidance must be done tentatively, as much of nonlinear analysis is still an art rather than a strict science. Ongoing studies will continue to improve our understanding of the requirements for nonlinear analysis in support of performance-based earthquake engineering in the years ahead, but even if the field of nonlinear analysis was fully studied there still would remain necessary judgments about the acceptable risk of exceeding various performance states. This paper is written, therefore, not as a final word on the subject of nonlinear analysis in support of performance-based earthquake engineering, but instead as a status report on a limited subset of the problem.

The discussion begins with a brief overview of why nonlinear analysis is important in seismic performance assessment. This is followed by discussion of seismic hazard and ground motion selection and manipulation. The roles of nonlinear static analysis, simplified dynamic analysis, and 'complete' nonlinear dynamic analysis are then described and compared. Some ideas related to the use of nonlinear dynamic analysis as an alternative to prescriptive code procedures follow. The paper concludes with some discussion of key detailing issues and the role of peer review in performance-based seismic design of tall buildings.

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2. PERFORMANCE AND NONLINEAR RESPONSE

With the exception of special high-performance structures and structures with special protective systems, it is usually not economically feasible to design a structure 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. For a yielding structure, the occurrence of structural damage is more directly related to deformation than it is to lateral force level. This concept has been effectively illustrated by graphics such as that in Figure 1. Recognition of the importance of lateral drift as a basis for design was noted decades ago (Muto, 1960) and has been subsequently noted in several documents (e.g., Sozen, 1980; Moehle, 1992; FEMA 356, 2000).

Modern designs usually concentrate the lateral-force resistance in only a portion of the building framing elements. The remainder of the building, commonly known as the 'non-participating' framing or 'gravity-only' framing, is not included as part of the design lateral resistance. Nonetheless, this 'gravity-only' framing must be designed to remain stable under lateral drifts anticipated for future earthquakes. Displacement-based design approaches provide a direct means of checking the stability of these systems.

Ability to support combined lateral and vertical forces in a yielding building requires both strength and deformation capacity. For example, a reinforced concrete column at the base of a building requires sufficient axial strength to support gravity loads and some portion of building overturning action. Furthermore, if that column yields flexurally during strong earthquake shaking, it requires inelastic deformation capacity, which depends on column reinforcement detailing and axial load (Bayrak and Sheikh, 1997). Linear analysis procedures generally provide poor indications both of the level of axial load and the degree of nonlinear action required. For significant buildings, nonlinear analysis procedures are preferred.

While the importance of deformation is paramount in performance-based assessment and design of yielding structures, the design should not overlook aspects of performance that may be affected by other demand parameters. For example, the performance of building contents and some nonstructural components (e.g., suspended ceilings) is controlled more by floor acceleration than by building deformation. Performance-based design should consider these components as well, providing a balanced design that weighs the relative importance and sensitivity of different components to different seismic response demand parameters.

3. GROUND MOTIONS FOR NONLINEAR ANALYSIS

Seismic hazard due to ground shaking should be determined considering the location of the building with respect to causative faults, the regional site-specific geological characteristics, and the selected

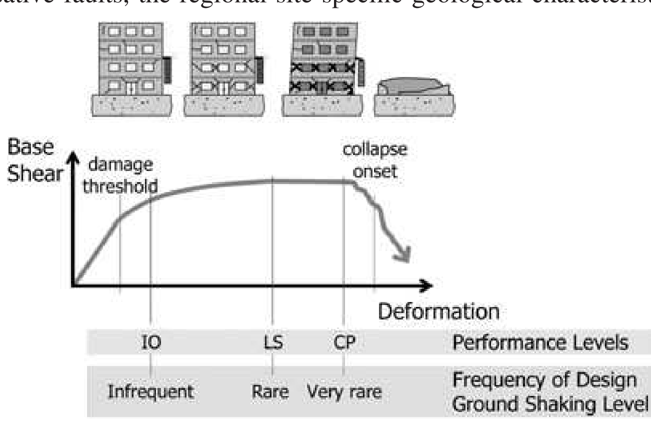

Figure 1. Relation between performance and nonlinear response

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earthquake hazard level. In general, the seismic hazard should include earthquake-induced geological site hazards in addition to ground shaking. The discussion here is limited to ground-shaking hazard.

Seismic ground-shaking levels for performance-based earthquake engineering can be defined using either a *general procedure* based on approved contour maps and standard response spectrum shapes (e.g., ASCE 7, 2002; FEMA 356, 2000) or *site-specific seismic hazard analysis*. For 'significant' structures, the latter approach is commonly used. Regardless of the approach, common US practice is to define both a design basis earthquake (DBE) and a maximum considered earthquake (MCE). In most cases, the DBE is at a hazard level consistent with the design basis for new buildings. When used in performance-based design applications for new building designs, the DBE may be used in conjunction with (some or all of) the prescriptive provisions of the building code as the basis for establishing initial building strength and stiffness requirements, while for existing buildings the DBE is sometimes used to check for life safety (a margin against collapse and protection against falling hazards) (FEMA 356, 2000). The MCE is usually used to check safety against local or global collapse. While details vary from case to case, the DBE usually corresponds to ground shaking having 10% probability of exceedance in 50 years (10%/50 yr); an alternative is to define DBE as two-thirds of MCE shaking (IBC, 2003). MCE shaking levels vary, usually corresponding to either 5%/50 yr (10%/100 yr) or 2%/50 yr (10%/250 yr) levels, perhaps capped by shaking associated with attenuation of characteristic earthquakes in regions with relatively well-defined active faults. Performance-based earthquake engineering practice for tall buildings also might consider damage control for earthquakes associated with higher recurrence intervals, though this has not been common practice in the USA. For example, common Japanese practice for high-rise buildings is to restrict structural response to the linear domain and to check performance of key nonstructural components for ground motions having peak ground velocity of around 10 in./s (Otani, 2004).

Where nonlinear dynamic analysis is used, representative ground motion records are required. Records should be selected from actual earthquakes 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 can be deaggregated for each hazard level to determine the contributions to the hazard from earthquakes of various magnitudes and distances from the site. Figure 2 illustrates deaggregation of the seismic hazard at one site in Los Angeles for a vibration period of 1 s and 2%/50 yr (2475-year return period) hazard level. For this case, it is clear that the seismic hazard is dominated by M6·5 to M7·0 events within about 10 km. Because magnitude strongly influences frequency content and duration of ground motion, it is desirable to use earthquake magnitudes 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 are commonly 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 spectrum 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. Figure 3 shows an example of spectrum-matched motions (Stewart *et al.*, 2001). Resulting motions should be compared with original motions to ensure the original character of the motion is not modified excessively. Amplitudes and shapes of waveforms may become modified by the process, but addition of a significant number of new velocity pulses that change the signature of the original motion in general should be avoided.

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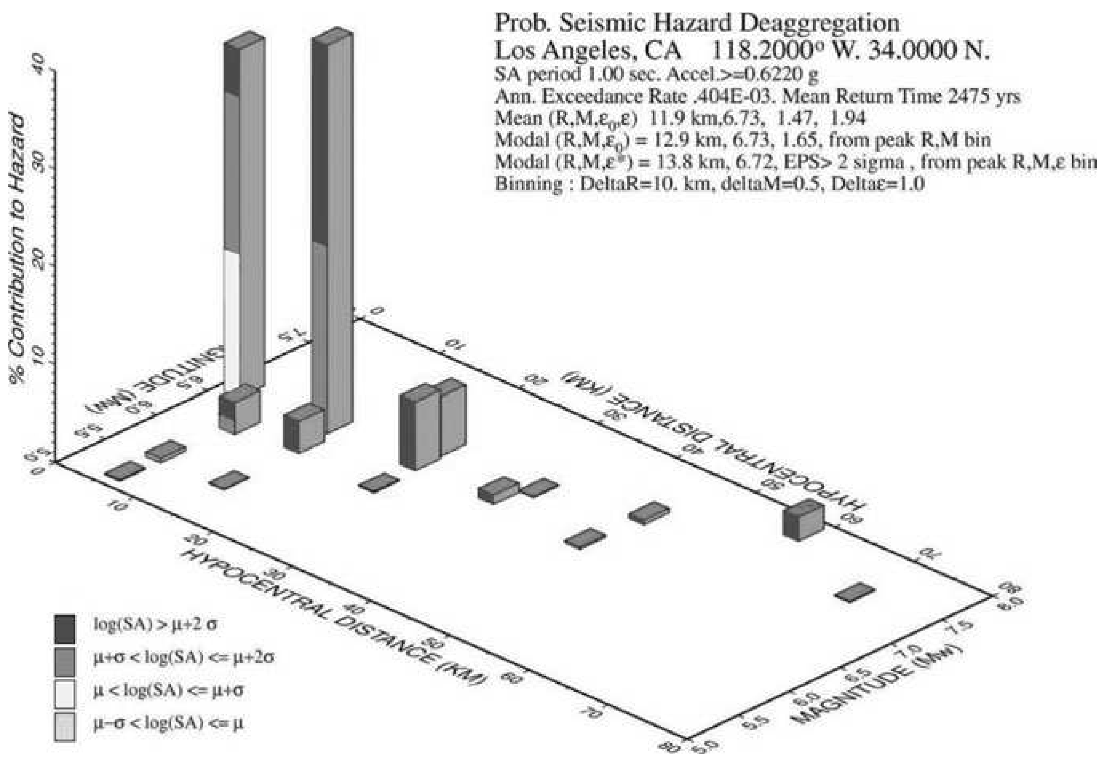

Figure 2. Deaggregation of the seismic hazard at a site in Los Angeles for 1 s period at 2%/50 yr hazard level (Stewart *et al.*, 2001)

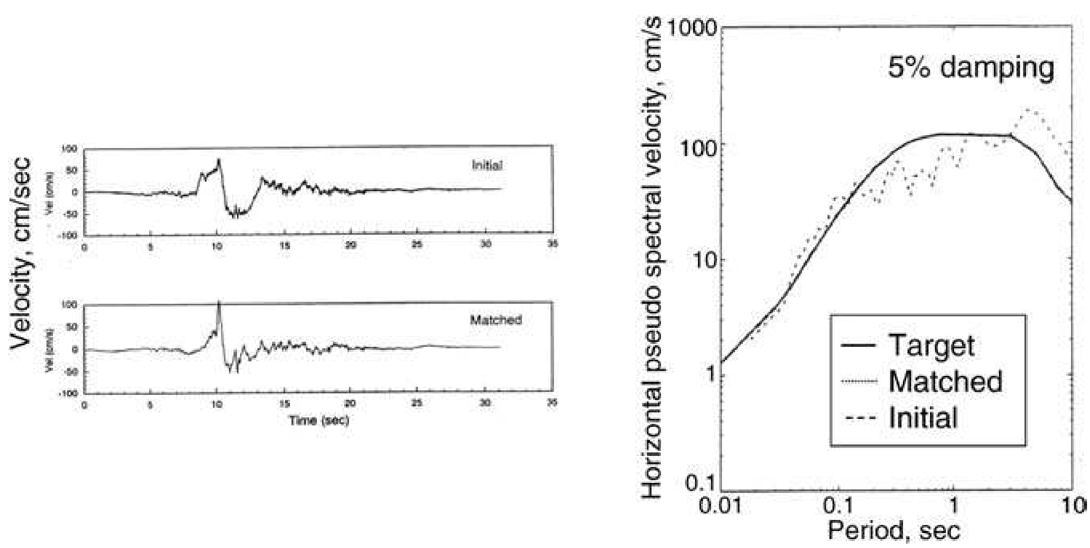

Figure 3. Example of spectrum-matched ground motion (Stewart *et al.*, 2001)

Ground motion scaling procedures are defined by codes and guidelines (e.g., ASCE-7, 2002). Given a suite of ground motion records, a scale factor is applied to each record to increase or decrease its intensity. The scale factors are selected through trial and error such that the average of response spectra from the scaled motions does not fall below the target design response spectrum over the period range $0{\cdot}2\,T$ through $1{\cdot}5\,T$, where T is the fundamental vibration period of the building. Where pairs of horizontal ground motions are considered for three-dimensional analysis, a single scaling factor is applied to both motions of the pair. The vector of the response spectra is calculated as the square root of the sum of the squares (SSRS) of the response spectra for the two different directions, and this vector spectrum should not fall below 1·4 times the target spectrum over the specified period range (the factor 1·4 is simply an approximation of $\sqrt{2}$ as required for the SRSS combination). Figure 4 shows an example set of ground motions scaled so the average of the SRSS spectra falls above 1·4 times the smooth design response spectrum over a long range of periods. Although not specified by most codes or guidelines, when ground motions are scaled to response spectra with significant differences in fault normal and fault parallel directions, it is preferable to scale the individual components to their individual target spectra rather than the vectors of the two components (Stewart *et al.*, 2001).

There is currently 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, to avoid response being uncharacteristically dominated by the peaks and valleys of any one ground motion, it is recommended to use not less than seven ground motion records. Spectrum matching may be more appropriate where fewer ground motions are used. However, effects of spectrum matching on nonlinear response are not well understood at this time; some engineers are concerned about skewing the energy content of ground motions through spectrum matching, which may have an unknown effect on nonlinear response.

When the scaling approach is used, scale factors should not be too large (approximately 2). Large scaling factors tend to bias nonlinear response toward the high side (that is, increase nonlinear response relative to that which would be obtained using records whose peaks naturally match the target spectrum) (Luco and Bazzurro, 2004). Response spectra associated with MCE hazard levels (e.g., 2%/50 yr) are often the result of the combination of a large event and an unusually large motion at the period of interest for that event (sometimes referred to as positive epsilon, where epsilon is defined as the number of standard deviations above or below the median ground motion level for the magnitude and distance that is required to match the probabilistic spectrum). If a motion is selected without approximately the same value of epsilon, and it is subsequently scaled up to the MCE spectrum, it will tend to overestimate nonlinear response (Baker, 2005).

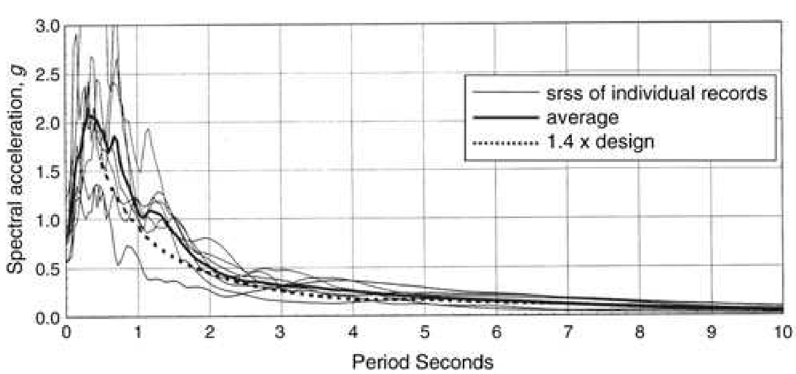

Figure 4. Example of scaled ground motions

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Selection of ground motions for tall buildings is complicated by the long fundamental period of the building. It may be difficult to find records with sufficient energy in the long-period range, therefore requiring relatively large scaling factors for records deficient in long-period energy. Application of a large scaling factor may result in unnaturally large spectral ordinates for shorter periods, a consequence being exaggerated higher-mode response. For such cases, spectrum matching may be preferred.

4. NONLINEAR STATIC ANALYSIS

Nonlinear static analysis is a simplified analysis procedure that can be useful for obtaining approximations of earthquake demands on buildings. The procedure is based on the assumption that the response quantity of interest is driven primarily by response in a single mode. This assumption is valid only for a subset of buildings and response quantities. Where the response quantity of interest is influenced by more than a single vibration mode, multi-mode procedures can be used to improve the approximation (Chopra *et al.*, 2004). Except where specifically noted below, this discussion is limited to single-mode nonlinear static analysis.

Nonlinear static analysis uses an analytical model whose component properties represent the nonlinear response of the components. The loading is accomplished by first applying gravity loads, then applying monotonically increasing lateral forces acting in a constant or time-varying profile over the height of the building (Figure 5). Behavior of the components and overall structural system is monitored to identify deformations at which key performance points (e.g., yielding, spalling, fracture, and instability) are reached.

To estimate the deformation demands corresponding to design-basis earthquakes, the building model is simplified to a single-degree-of-freedom (sdof) model. This is accomplished by assuming the deformed shape at some point during the nonlinear static analysis corresponds to a linear mode shape from which effective mass (M^*), sdof acceleration (S_{sdof}), and sdof displacement (Δ_{sdof}) can be obtained (ATC 40, 1996; Chopra and Goel, 2001; Qi and Moehle, 1991; Saiidi and Sozen, 1981) as

$$M^* = \frac{\left(\sum m_i \phi_i\right)^2}{\sum m_i \phi_i^{\,2}},\ S_{\text{sdof}} = \frac{V_{\text{base}}}{M^*},\ \Delta_{\text{sdof}} = \frac{\Delta_{\text{roof}}}{\Gamma \phi_{\text{roof}}},\ \Gamma = \frac{\left(\sum m_i \phi_i\right)}{\sum m_i \phi_i^{\,2}}$$

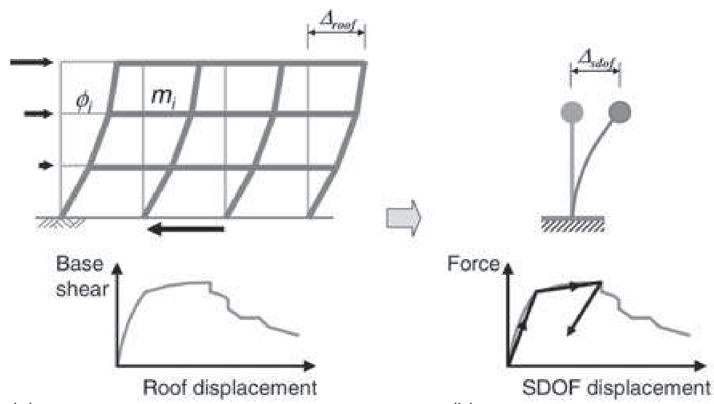

Figure 5. Static nonlinear analysis and conversion to equivalent sdof system: (a) static nonlinear analysis; (b) equivalent sdof system

in which m_i = reactive mass at level i, ϕ_i = mode shape value at level i, V_{base} = base shear of actual building loaded to deform in first mode shape, and Δ_{roof} = displacement at roof. Given properties of the equivalent sdof system, various techniques can be used to estimate the seismic response.

One approach is to define a hysteresis rule for the sdof oscillator representative of the overall load-deformation behavior of the building and compute a response history using appropriate software (e.g., Hachem, 2004). Dynamic analysis of the oscillator under the design ground motions can provide insight into maximum displacement response as well as the number of cycles to be anticipated. Figure 6 illustrates application of this approach for a 10-story frame tested on an earthquake simulator (Saiidi and Sozen, 1981). Response of the sdof oscillator was scaled by coefficient C_0, based on the assumed mode shape to obtain the response at the roof level:

$$C_0 = \frac{\phi_{roof} \sum m_i \phi_i}{\sum m_i \phi_i^2}$$

in which ϕ_{roof} = mode shape value at the roof.

As an alternative to dynamic response history analysis, maximum response can be estimated using procedures based on the initial stiffness or on equivalent linearization at a reduced stiffness. FEMA 440 (2005) provides guidance on use of these procedures. The procedure based on initial stiffness defines the expected peak displacement by the following formula:

$$\Delta_{roof} = C_0 C_1 C_2 \frac{T_e^2}{4\pi^2} S_a$$

in which Δ_{roof} = roof displacement, C_1 = a coefficient based on strength and period, C_2 = a coefficient based on stiffness and strength degradation, T_e = initial vibration period (considering cracked sections for concrete), and S_a = the spectral acceleration in units of length per second^2. Coefficient C_2 can be taken equal to 1·0 for most structures. Coefficient C_1 is given by the following equation, with typical results in Figure 7:

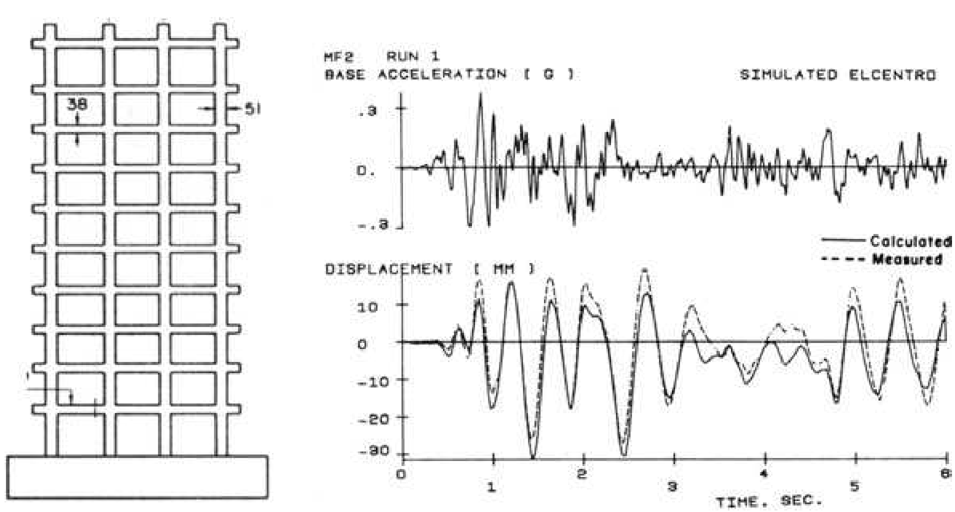

Figure 6. sdof analysis of a multistory test structure: (a) structure elevation; (b) input motion and response history

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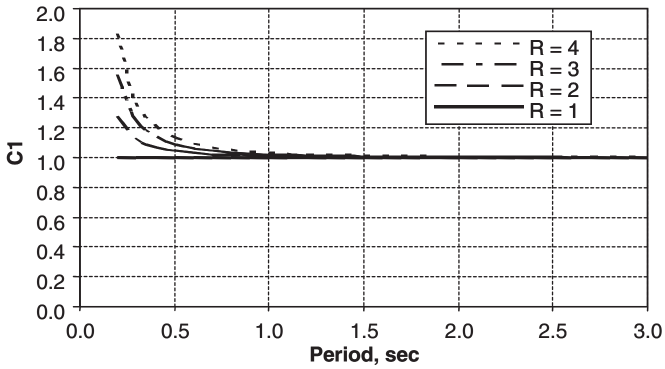

Figure 7. Effect of strength and period on displacement amplification relative to elastic response for NEHRP site class C

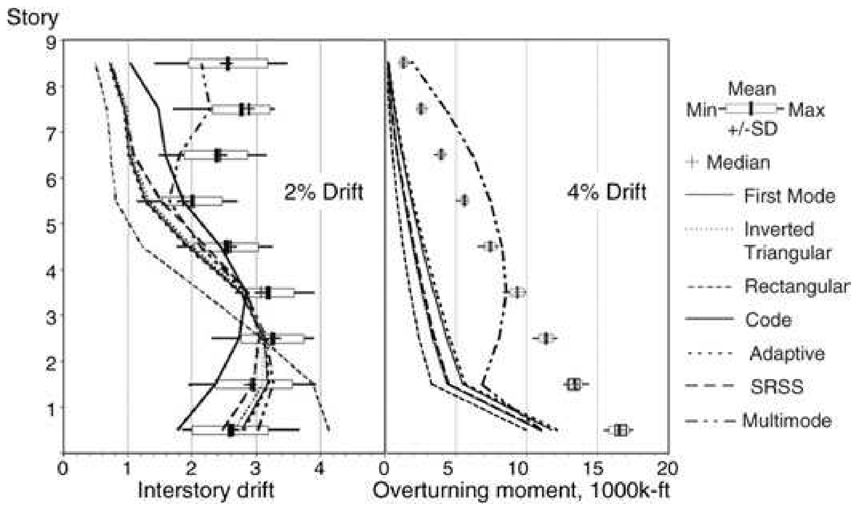

Figure 8. Comparison of responses calculated by nonlinear dynamic analysis and nonlinear static analysis (FEMA 440, 2005)

$$C_1 = 1 + \frac{R-1}{aT_e^2}$$

in which R = the ratio of elastic strength demand to actual strength and a = 130, 90, or 60 for NEHRP site classes B, C, and D, respectively.

Conventional nonlinear static analysis is limited by its basic premise—that response is dominated by a fundamental mode. For more flexible structures (including taller buildings) and response quantities more strongly influenced by higher modes (such as interstory drift, component plastic-hinge rotations, and accelerations), nonlinear static analysis can be highly inaccurate. Figure 8 presents sample results for a nine-story frame structure, demonstrating the potential inaccuracies. Several different lateral load profiles are presented. A conclusion of the FEMA 440 (2005) study is that lateral displacement was reasonably estimated by a lateral load profile corresponding to the first-mode shape without significant improvement using other (including adaptive) load profiles. Interstory drifts were better estimated using multi-mode loading methods (FEMA, 2005; Chopra *et al.*, 2004), though these are more complicated to apply. None of the methods was consistently reliable for estimating internal forces.

5. SIMPLIFIED MULTISTORY MODELS

Shortcomings in sdof nonlinear static analyses on the one hand, and complications of complete three-dimensional nonlinear dynamic analyses on the other, have led to some applications of simplified multi-degree-of-freedom (mdof) models as a compromise. In Japan, where high-rise buildings usually comprise frames (rather than core walls), the nonlinear dynamic analysis is usually done using a simplified model comprising a single mass, spring, and dashpot per floor (Otani, 2004). These models can be sufficiently accurate in cases where the degree of nonlinearity is relatively small, as is intended for buildings in Japan.

The 'fish-bone' model (Figure 9) is a slightly more complicated version of this basic concept that can result in improved estimates for nonlinear analysis of frame buildings. By this model, the flexural and shear properties of beams and columns are 'averaged' across the building and lumped in a stick frame. Sample results from this model are presented by Nakashima *et al.* (2002) (Figure 10), demon-

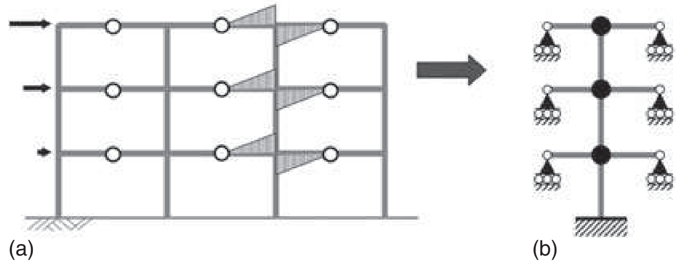

Figure 9. Fishbone model

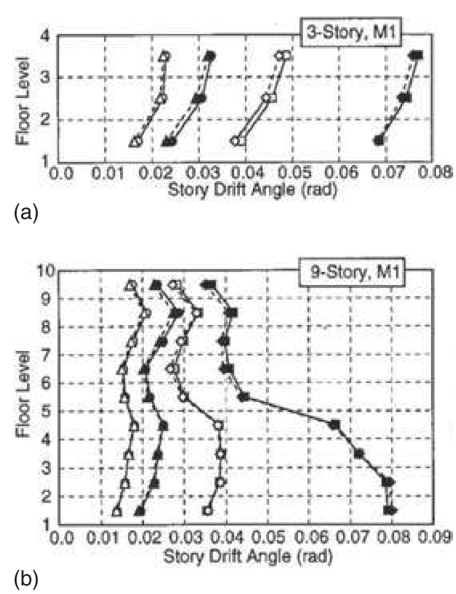

Figure 10. Comparison of peak interstory drifts computed by fishbone and complete nonlinear models: (a) three-story frame; (b) nine-story frame

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strating its suitability, if properly constituted, to represent various building responses with reduced computational expense. This approach is sometimes used as a research tool but is seldom used in design.

Such models are not commonly used today in the USA.

6. ANALYSIS FOR TALL BUILDINGS

Performance-based seismic analysis of tall buildings in the USA 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 not designed as part of the lateral-force-resisting system) may or may not be directly modeled; if their contribution to seismic resistance and their interaction with lateral-force-resisting parts of the building are negligible, it is not necessary that they be included. However, effective mass and *P*-delta effects associated with 'non-participating' parts of the building must be included in the overall analytical model; and non-participating components that support gravity loads need to be checked for performance at anticipated force and deformation demands associated with MCE loadings. Gravity systems in tall buildings can act as 'unintentional' outriggers, developing significant axial force over height, an effect that should be checked.

Experimental and numerical studies of nonlinear dynamic response demonstrate that, because the behavior is nonlinear, internal actions cannot be scaled directly from linear analysis results; similarly, nonlinear behavior at one hazard level cannot be scaled from nonlinear results at another hazard level. Additionally, 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; Priestley and Amaris, 2003). Figure 11 illustrates this for a multistory wall building subjected to different levels of earthquake ground motion. According to this analytical result, the wall develops its plastic moment strength at the base, as intended in design, and wall base moment remains at the plastic moment capacity as the intensity of ground motion increases. Wall moments above the base, and wall shears at all levels, however, continue to increase with increasing ground motion intensity even though

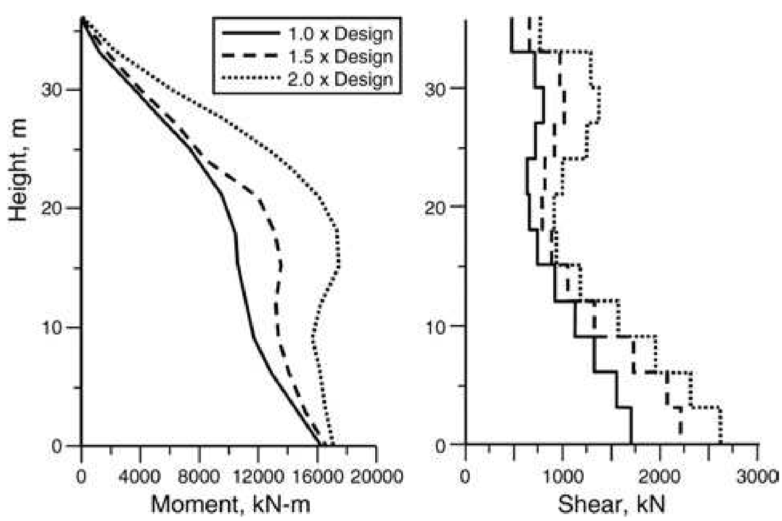

Figure 11. Wall moments and shears for increasing intensity of ground shaking (after Priestley and Amaris, 2003)

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the base has reached its plastic moment capacity. This is because lateral deformations in various 'modes' and associated internal forces continue to increase as shaking intensity increases. Design studies of very tall wall buildings suggest that this behavior can lead to formation of secondary wall plastic hinges near mid-height. Only by analyzing the building for the target hazard level can these internal deformations and forces be identified.

Various empirical approaches have been proposed to estimate internal actions associated with nonlinear dynamic response (e.g., Kabeyasawa, 1993; Paulay and Priestley, 1992; Eberhard and Sozen, 1993; Priestley and Amaris, 2003). Most such procedures were derived for a limited set of ground motion types, framing types, and building heights, and applying these empirical approaches to different conditions can be shown to produce erroneous results.

The preferred approach is to study the response of the actual building design to representative (scaled or spectrum-matched) ground motions at the target hazard level using nonlinear dynamic analysis. For this purpose, a detailed nonlinear model of the building is assembled using appropriate computer software. A first step in the development of the nonlinear model is to apply the capacity design concept to decide which parts of the building are intended to yield during earthquake shaking and which parts (or actions) are intended to remain in the essentially linear response domain. This step is important not only as a design tool to help restrict yielding to ductile components; it also enables the development of a reasonably accurate nonlinear computer model without requiring full nonlinear capabilities for all components or all modes of behavior. The computer analysis results can subsequently be used to help determine design demands and required capacities for the capacity-protected components of the building that are intended to remain in the linear response range.

While a full discussion on nonlinear modeling is beyond the scope of this paper, it is noted that selection of component load-deformation properties is important to the final result. In reinforced concrete building 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; Adebar *et al.*, 2004; Elwood and Eberhard, 2006). When using resources such as these, be certain the expected applications are well understood. For example, FEMA 356, Paulay and Priestley, and Adebar each provide recommendations for wall stiffness, but only one (Adebar) is specific to high-rise walls with relatively high axial stresses. 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. Damping properties for concrete buildings are normally set around 5% of critical damping for the periods (vibration modes) likely to dominate response. Damping for steel buildings should be in the range of 2–3% of critical damping.

Figure 12 shows summary results for a 40-story concrete wall building located in a region of high seismicity in the western USA (Maffei, 2005). Presented results include minimum, maximum, mean, mean plus one standard deviation, and coefficient of variation of roof drift, wall base shear, and wall moment at the 13th floor. Partly because of the relatively significant irregularity of the building, some of the response results show significant dispersion.

A key question is how this dispersion should be taken into account in design. It is tempting to assume the response dispersion follows a lognormal distribution, and from these statistics to derive design quantities having probabilities of exceedance conditional on the hazard level (for example, a shear design level corresponding to 10% probability of exceedance for the selected hazard level). However, it must be recognized that the selection and scaling of individual ground motion records from different earthquakes and sites introduce additional dispersion to the results that is not representative of true dispersion for the building at its construction site (Der Kiureghian, 2005). Therefore, design values obtained from such exercises may be overly conservative.

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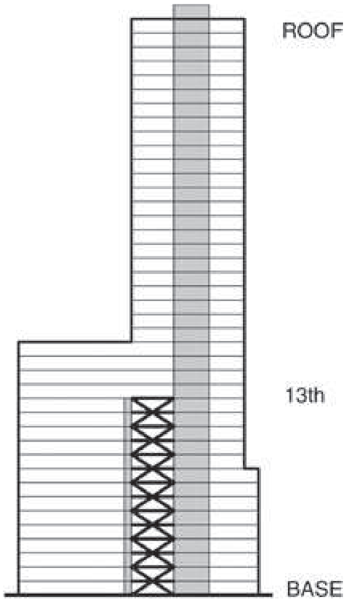

	Roof drift, ft	Wall base shear, k	Wall moment at 13th floor, 1000 x k-ft
Minimum	2.1	7600	513
Maximum	6.7	29700	1080
Mean (m)	4.2	15500	900
m + σ	5.4	22200	1090
c.o.v.	0.23	0.43	0.21
Nonlinear static		5500	760

Figure 12. Building elevation and summary of nonlinear dynamic and nonlinear static analysis results. Nonlinear static analysis results are for inverted triangular floor acceleration pattern: (a) building elevation; (b) summary of results

7. NONLINEAR ANALYSIS AS A PERFORMANCE-BASED ALTERNATIVE TO CODE PRESCRIPTIVE PROVISIONS

Buildings codes may contain provisions that allow for use of alternative materials, systems, or means of verification, so long as it is demonstrated that the alternative results in performance at least equal to that of the materials, systems, and means prescribed in the code. As an example, UBC (1997) states in 1629.10.1:

> Alternative lateral-force procedures using rational analyses based on well-established principles of mechanics may be used in lieu of those prescribed in these provisions.

Nonlinear dynamic analysis, properly executed and interpreted, can be suitable as an alternative procedure. It has been used on numerous occasions in actual building designs to demonstrate through analysis (or analysis combined with experimental evidence) that building system performance will equal or exceed performance intent of the prescriptive provisions of the code. Examples (based on actual construction projects) include:

- Innovative structural framing systems—in these cases test data on representative large-scale models may be required to demonstrate constructability, quality control requirements, and structural performance, with nonlinear analysis demonstrating earthquake demands do not exceed test capacities.
- Conventional framing systems built to greater heights than permitted by code provisions—in these cases nonlinear dynamic analysis is used to demonstrate reliable performance within accepted drift, inelastic deformation, and force demands.
- More effective conventional designs—in these cases an objective may be to except certain code provisions (e.g., minimum strength requirements) to achieve greater economy; another objective

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may be to revise internal distributions of reinforcement to achieve improved performance at fixed cost.

The preceding discussion suggests some minimum considerations for alternative design provisions for high-rise buildings, as discussed in the following paragraphs.

Buildings are expected to respond nonlinearly under MCE level ground motions. Therefore, nonlinear dynamic analysis is encouraged as a tool for performance-based designs. 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. Details superior to the minimum requirements of the prescriptive building code provisions can be determined by such analysis, leading to greater confidence in building performance characteristics including safety.

Building nonlinear response is highly sensitive to the unique character of individual ground motions. Nonlinear analysis verification studies, therefore, should use several ground motion records for each considered hazard level to obtain a representative sample of expected performance. ASCE-7 (2002) specifies that design values be taken as the maximum values obtained from calculated response when three ground motion records are used or the average values when seven or more records are used. Records should be selected from actual earthquakes considering magnitude, distance, site condition, and other parameters that control the ground motion characteristics. Where record scaling is used, scale factors should not be excessive. Spectrum-matching approaches should not result in extraordinary changes in the signature of the original recorded ground motion record. Ground motion durations, which can be important for long-period structures, should be consistent with magnitude and distance of earthquakes that dominate the seismic hazard.

Nonlinear dynamic response is, by its very nature, nonlinear, such that linear scaling of results from one shaking intensity to another is unreliable. Dynamic response studies should verify performance at multiple hazard levels that are deemed important. Linear response verification (including serviceable performance of structural and nonstructural systems) may be appropriate for low hazard levels (though this has not been common practice in the USA). Drift and damage control, as well as life-safety protection, may be considered for DBE shaking levels (possibly 10%/50 yr hazard level). Collapse safety should be verified for MCE shaking levels (possibly 10%/100 yr or 10%/250 yr hazard levels). Nonlinear dynamic analysis is the preferred means of verification for DBE and MCE levels, though alternatives may be sufficient for some structures.

Nonlinear dynamic analysis invariably results in statistics of response quantities, in contrast with code prescriptive provisions that result in specific design requirements. Assuming not less than seven ground motion records is used as the basis of establishing the statistics, common practice for drift and for ductile response quantities is to use the mean for design. For less ductile response modes, such as shear in reinforced concrete, design should consider the dispersion about the mean. (Figure 12 provides an example with sample results not uncommon in design.) Studies show that the dispersion in nonlinear dynamic analysis results is exaggerated by various aspects of ground motion selection and scaling (Der Kiureghian, 2005). Therefore, no specific recommendation can be made at this time regarding appropriate amplification factors to be selected for design of less ductile components. Engineering judgment continues as the guiding principle.

8. STRUCTURAL CONCRETE DETAILING

A significant percentage of high-rise building construction in the western USA will be primarily for residential occupancies. As such, much of it will be constructed of reinforced concrete. Laboratory and field experience demonstrates that getting the details right is critical to good performance, and

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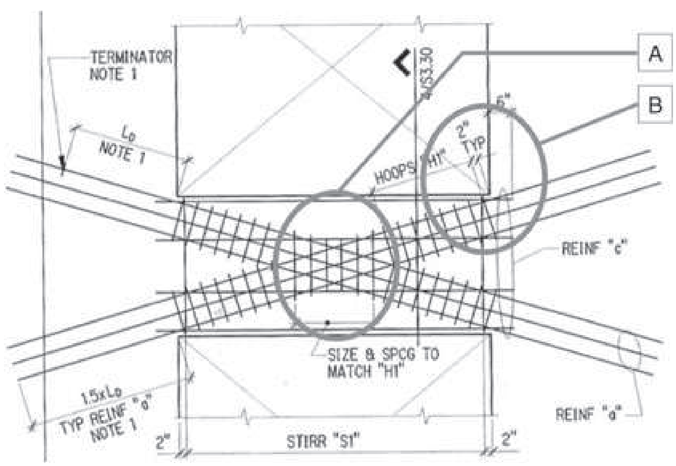
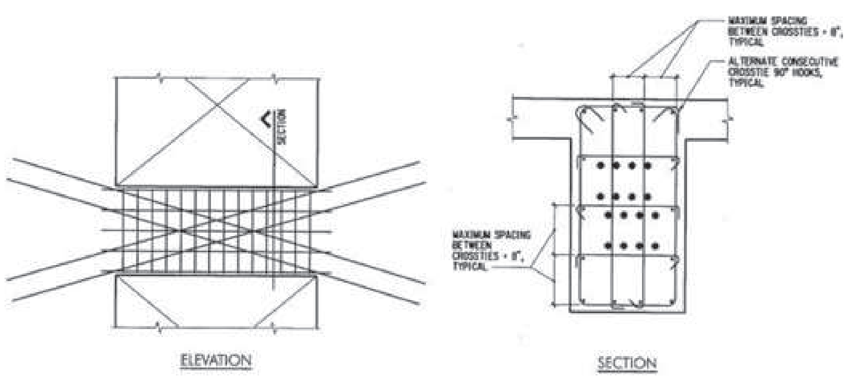
reviews of tall building designs and field implementations suggest some areas where special attention may be needed.

Reinforced concrete core walls have become increasingly popular in high-rise construction in recent years. To function properly under severe earthquake loading, the 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. While simple in concept, these beams can be challenging to construct properly in the field following the building code prescriptions (ACI 318, 2005). As prescribed, the individual diagonals are to be confined by transverse reinforcement, with the remainder of the beam basketed by nominal transverse and longitudinal reinforcement. Two problem areas commonly arise. First, it is difficult to confine the complex geometry where the diagonals intersect (see 'A' in Figure 13a). A second area of difficulty is where the diagonal bars enter the wall (see 'B' in Figure 13a); especially in low-aspect-ratio beams, the steep inclination of the diagonals can leave the diagonal bars unconfined in this most critical zone. Selective placement of crossties often is the only remedy to these confinement challenges.



(a)



(b)

Figure 13. Coupling beam detailing: (a) common detailing problem areas; (b) full section confinement

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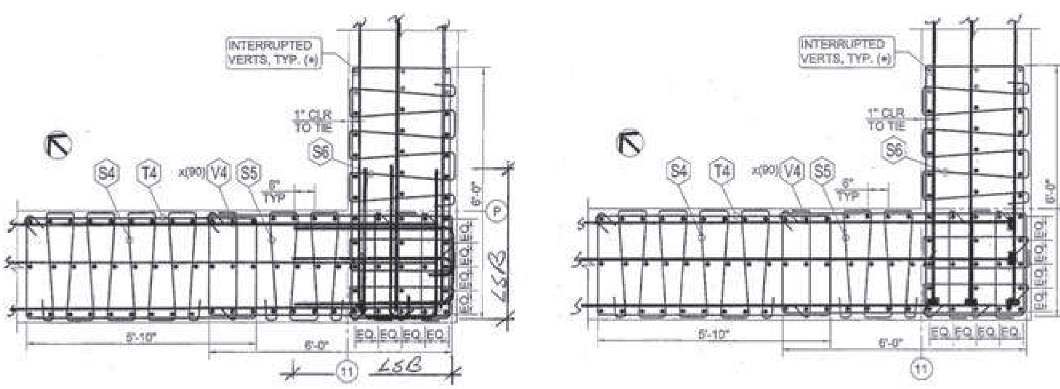
Figure 13(b) shows an alternative that meets the intent of the code, in which the entire beam cross-section is confined by transverse reinforcement. This detail is being considered for explicit incorporation in ACI 318 at the time of this writing. This detail may prove more economical than that specified in Figure 13(a) because of savings in labor costs. Interestingly, the original tests on which the diagonally reinforced concrete beam is based did not use heavy confinement of the individual diagonals (Paulay and Binney, 1974; Barney *et al.*, 1978).

The core wall itself can be heavily stressed under significant inelastic deformations near the base (and elsewhere). 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 $1{\cdot}25f_y$, where f_y is the nominal yield strength of the longitudinal reinforcement. Furthermore, longitudinal reinforcement shall be extended a distance $0{\cdot}8l_w$ 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 indicates compression strains well below the ACI limit of 0·003; computer software for nonlinear earthquake response analysis may produce strain as one of the output quantities, but such values can be strongly dependent on modeling assumptions and should be validated (by the engineer of record) against strains measured in laboratory tests. 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 14 shows example details for boundary element confinement and anchorage of shear reinforcement. Figure 14(a) illustrates a detail in which the horizontal reinforcement for shear is lapped with an equal area of U-bars. Figure 14(b) shows a detail using headed bars. The alternative of hooking the horizontal reinforcement (not shown) is another acceptable alternative, though it may not be feasible if large-diameter horizontal bars are used.

Adjacent to the potential plastic hinge zone (or zones) it may be prudent to have a transition from the full level of confinement, to an intermediate confinement, to low confinement. The transition to essentially no confinement, as seems permitted by some interpretations of the building code, seems to this author to be imprudent because of uncertainty in curvature distributions up the height of the



(a) (b)

Figure 14. Wall boundary details: (a) boundary element plan A; (b) boundary element plan (b)

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building. It is this author's view that longitudinal reinforcement should be restrained throughout the building height by lateral reinforcement at least satisfying the requirements for tied columns in Chapter 7 of ACI 318 (2005).

Special moment frames also are widely used in high-rise building construction, either as the primary lateral-force-resisting system or as part of a dual system. Ductile response of these frames relies on detailing of columns, beams, and beam–column joints, as well as proportions that promote a strong-column, strong-joint, yielding-beam mechanism dominated by flexural yielding without shear or anchorage/splice failure. It has long been known that columns with high axial loads have reduced flexural ductility capacity. Designs that maintain axial loads at or below balanced axial load (approximately $0{\cdot}35A_g f_c'$) are encouraged. Designs with higher axial loads can perform well if flexural yielding is minimized or transverse reinforcement detailing is improved relative to code minimum requirements (Bayrak and Sheikh, 1997).

Conventionally reinforced or post-tensioned floor slabs supported by gravity frames are commonly used with either core walls, special moment frames, or both. Slab–column connections should be designed considering effect of vertical slab shears on the lateral drift capacity of the connection. ACI 318 (2005) includes requirements considering this interaction; see the seismic requirements for members not designed as part of the lateral-force-resisting system. Stud rails and other alternative systems are seeing increasing use to reduce the likelihood of punching around the connection under imposed vertical loads and lateral deformations. Provisions for structural integrity also should not be overlooked. For conventionally reinforced slabs, ACI 318 (2005) requires at least two of the main bottom bars in each direction be continuous over the column; ACI 352 (1989) recommends additional bottom reinforcement based on equilibrium considerations. For unbonded post-tensioned slabs, at least two of the strands in each direction must pass through the column cage.

Reinforced concrete gravity columns near the base of the building may sustain relatively high axial loads and may yield especially at locations where they frame into basement walls or foundation elements. Minimum code requirements (e.g., ACI 318, 2005) should be adhered to, of course, but consideration should also be given to pursuing a higher performance goal at this location by improving confinement details (e.g., not permitting 90° hooks on cross-ties) and increasing quantities and lengths of confinement to improve axial-load-carrying capacity following spalling of cover concrete.

9. PERFORMANCE-BASED DESIGN OF BUILDINGS IN PRACTICE

A wide variety of performance-based earthquake engineering approaches are currently being implemented in various jurisdictions, including Los Angeles, San Francisco, Seattle, and other cities. These approaches are challenging both engineers and code officials. To facilitate acceptance by the local building authorities, it may be advisable to design the building, to the extent possible, according to the prescriptive provisions of the code, deviating only where necessary to accommodate the high-performance materials or system for which approval is being sought. Structural analysis or testing then can be used to demonstrate that the performance will be acceptable, and not less than that anticipated for conventional materials and systems. It can be helpful to develop a written set of criteria that delineate what code provisions are being excepted and what verification procedures will be used to demonstrate performance.

In performance-based designs today it is common practice to provide strength per the code requirement with component and system acceptance criteria from the prescriptive code provisions, though exceptions can be granted. Commonly applied criteria include code drift limits, ductile component strengths based on expected material properties, and non-ductile component strengths based on design material properties and including strength reduction factors. A trend (e.g., LATB, 2006) is to check

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serviceable performance for earthquakes with relatively short return periods (e.g., 43 years) and stable response (collapse prevention) for rare events (e.g., return periods of 2500 years). For the latter check, nonlinear dynamic analysis is used as a means of verifying performance, with component deformation capacities commonly based on quantities specified in FEMA 356 (2000) or based on tests of representative subassemblies. These should be reviewed on a case-by-case basis.

Few building departments have the expertise required to understand and approve the code exceptions and alternative means being proposed. 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 result interpretations, acceptance criteria, and quality assurance in design and construction. Peer review by independent qualified experts is recommended to help assure the building official that the proposed materials and system are acceptable.

10. CONCLUSIONS

Nonlinear dynamic analysis is increasingly being used as a tool to verify seismic performance of significant structures. Available software tools, 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. Care must be exercised to specify, and implement, reinforcement details that will perform as intended. Peer review remains an essential part of performance-based design of high-rise buildings.

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