Chapter 26
Performance-Based Earthquake Engineering in the U.S.: A Case Study for Tall Buildings

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Abstract Two influential developments in performance-based earthquake engineering in the U.S. are (1) development of the Tall Buildings Initiative *Guidelines for Performance-based Seismic Design of Tall Buildings* and (2) development of the ATC 58 *Guidelines for Seismic Performance Assessment of Buildings*. The content and methods of the two guidelines are summarized. A case-study project uses the Tall Buildings Guidelines to develop tall building conceptual designs for a site in Los Angeles, California, and then uses the ATC 58 Guidelines to explore the performance implications in terms of initial cost and future repair costs considering anticipated future earthquakes. The conceptual designs are done both using a building code prescriptive method and the performance-based method. Earthquake ground motions considered representative of different hazard levels for the site are imposed on an analytical model accounting for nonlinear response characteristics, leading to statistics on engineering demand parameters and associated repair costs. The study identifies apparent shortcomings in the code prescriptive methods as well as benefits associated with the performance-based methods.

Keywords RC buildings • Numerical models • Nonlinear analysis • Performance-based earthquake engineering • Repair cost • Building codes

26.1 Introduction

Performance-based seismic design methods in the U.S. originated as a practical and effective means to mitigate the seismic risks posed by existing buildings and were later extended to permit development of new buildings designed outside

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the prescriptive limits of the building code (Moehle et al. 2011a). This practice has become particularly prevalent in the design of very tall buildings in the Western U.S. Initially, engineers adopted ad hoc procedures for performance-based seismic design of tall buildings. Later, documents including SEAONC (2007), LATBSDC (2008), and TBI (2010) formalized these procedures. The earliest of these guidelines (SEAONC 2007) essentially adopted the building code procedures, including strength and drift checks for the Design Earthquake (ASCE 7 2005), but permitted some building code exceptions if adequate performance was demonstrated by nonlinear dynamic analysis. Experience and research (e.g., ATC-72 2010) led to evolution of the procedures with time. In the most recent of these guidelines (TBI 2010), the requirement to check strength and drift for the Design Earthquake is eliminated. Instead, building acceptability is judged based on demonstrated performance for Serviceability Level and Maximum Considered Level seismic demands.

In the aforementioned design guidelines, performance is measured by engineering demand parameters (EDPs) such as building drift, building stability, and local component demands. For a tall building stakeholder, however, performance may be better represented in terms of initial cost and the cost to repair damage from postulated future earthquakes. Advances in defining useful performance metrics, data, models, and analytical tools (e.g., Taghavi and Miranda 2003; Moehle and Deierlein 2004; Yang et al. 2009; ATC-58 2012; Porter et al. 2010;) have enabled practical assessment of expected future repair costs, now embodied in the ATC 58 *Guidelines for Seismic Performance Assessment of Buildings* (ATC-58 2012). Application of such methods to tall buildings designed by alternative methods can provide insight into the performance potential of tall buildings in general, as well as the effectiveness of the alternative design approaches.

The present study examines the design and expected performance of three tall building configurations designed by alternative methods. The study is part of a larger study (Moehle et al. 2011b) that considered alternative design strategies. This paper presents an overview of the design and assessment approaches, results of the case study designs, results of nonlinear dynamic analyses, and financial implications of the designs. The study illustrates a broadly applicable approach for comparing alternative designs in terms of engineering performance and financial measures.

26.2   The TBI Guidelines

The TBI (Tall Buildings Initiative) Guidelines present an overview of the recommended design and review process for tall buildings in regions of high seismicity, including detailed procedures to design for serviceability (Serviceability Level) and safety (Maximum Considered Earthquake Level). Serviceability Level seismic demands are obtained from modal response spectrum analysis of a three-dimensional model using a 2.5-percent-damped uniform hazard response spectrum having 43-year return period, with inter-story drift ratios limited to 0.005 and

maximum component forces limited to 1.5 times conventional design strengths. In effect, the Serviceability Level check establishes the minimum required building strength, which replaces the strength requirement of the prescriptive building code. The Maximum Considered Earthquake Level check uses nonlinear dynamic analysis of a three-dimensional analytical model subjected to two horizontal components of seven earthquake ground motions scaled to the uniform hazard spectrum representing the Maximum Considered Earthquake Level (ASCE 7 2005). For each pair of horizontal ground motions, the maximum inter-story drift is obtained in each story. For each story, the mean and maximum of the seven drift values are limited to 0.03 and 0.045 (transient), and 0.01 and 0.015 (residual). The Maximum Considered Earthquake Level check intends to demonstrate structural stability during a rare event. Therefore, yielding members are required to respond within limits that can be modeled reliably, and overall strength degradation of the structural system is limited. Provisions also limit the force demands in components with limited ductility (in effect, capacity-protected components). Details of the criteria are found in (TBI 2010).

26.3   The ATC 58 Guidelines

The ATC 58 Guidelines for Seismic Performance Assessment of Buildings (ATC-58 2012) describes a general methodology and recommended procedures to assess the probable earthquake performance of individual buildings based on their unique site, structural, nonstructural and occupancy characteristics. Performance measures include potential casualties, repair and replacement cost and schedule, and potential loss of use due to unsafe conditions. The methodology and procedures are applicable to performance-based design of new buildings, and performance assessment and seismic upgrade of existing buildings. The methodology involves many steps, including assembly of a building performance model that defines the building including occupancy; definition of earthquake hazards; analysis of building response; development of a collapse fragility; and various performance calculations. The buildings included in the present study are deemed rugged against collapse for reasonable ground motions. Therefore, in this study we skip the collapse fragility procedure of the methodology. Furthermore, we consider only losses associated with repair cost.

ATC-58 uses a Monte Carlo procedure to explore variability in building performance outcomes given earthquake shaking intensity (Fig. 26.1). First, the building is defined in terms of geometry, occupancy, and performance groups, that is, groupings of similar elements in each story whose performance is likely to affect the overall building performance outcome. An analytical structural model of the building is subjected to a single ground motion to identify maximum values of engineering demand parameters such as inter-story displacement or absolute acceleration. This process is repeated several times to establish expected values and variability of the engineering demand parameters as a function of ground shaking intensity. A
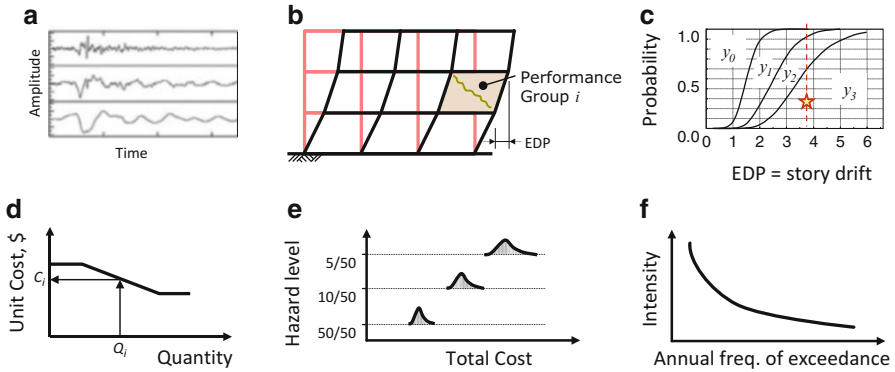
a Amplitude / Time

b Performance Group i / EDP

c Probability / 1.0 / 0.5 / 0.0 / y_0 y_1 y_2 y_3 / 0 1 2 3 4 5 6 / EDP = story drift

d Unit Cost, \$ / C_i / Q_i Quantity

e Hazard level / 5/50 / 10/50 / 50/50 / Total Cost

f Intensity / Annual freq. of exceedance

Fig. 26.1 Capital loss calculation. (**a**) Subject building to ground motions. (**b**) Record engineering demand parameters on damageable performance groups. (**c**) Use random number generator to enter fragility relations and determine damage state. (**d**) Identify repair quantities and costs. (**e**) Repeat many times at each of several hazard levels. (**f**) Integrate with the seismic hazard curve to generate loss measures of interest

statistical technique then is used to generate large numbers of "realizations," each realization representing a plausible response outcome, with the statistics of all the realizations matching that of the smaller set of earthquake response analyses. For each realization, the damage states and repair actions of performance groups are selected based on pre-defined fragility relations. Total building repair cost is then determined based on the total of the building repair quantities and repair actions. By repeating the process a large number of times, the statistics of repair costs are established. The repair costs for each shaking intensity then can be integrated with the seismic hazard curve to establish annual frequencies of exceeding specific repair costs. See ATC-58 for additional details.

26.4 Site Seismic Hazard and Representative Ground Motions

An aim of the study was to identify performance characteristics of prototypical tall buildings exposed to the seismic hazard typical of coastal California. Design and subsequent performance analyses required identification of a hypothetical building site. The selected site is in downtown Los Angeles, California (Longitude $= -118.25$ and Latitude $= 34.05$). The NEHRP soil site class is C (VS30 $= 360$ m/s). The site is close to several known faults, including the Puente Hills and San Andreas faults, respectively at closest distances of 1.5 and 56 km from the building site. Figure 26.2 shows a deaggregation of the seismic hazard at 2 and 70 % in 50 years hazard levels (2,475 and 43 years return period, respectively) for vibration periods of 3 and 5 s. The deaggregation results show that for long-period
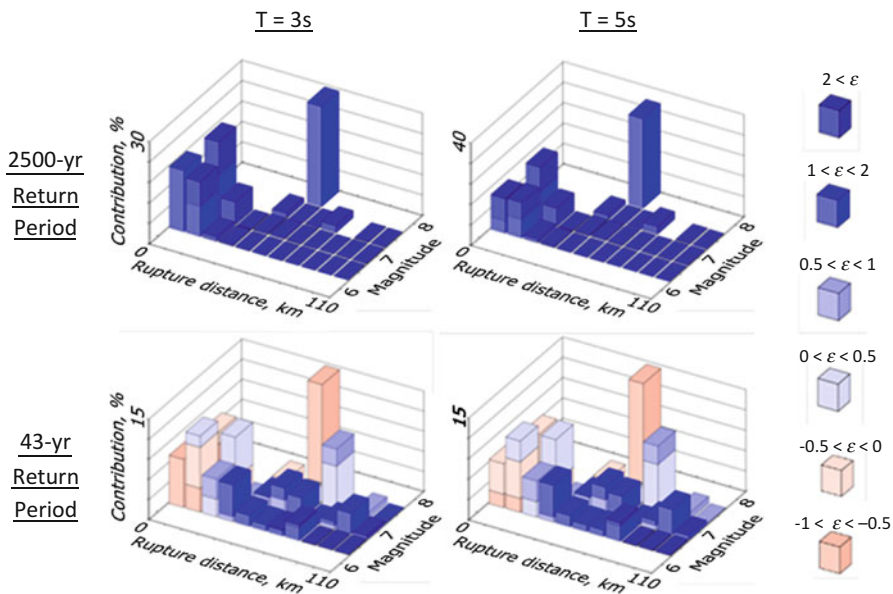
Fig. 26.2 PSHA deaggregation for the building site for dominant periods of 3 and 5 s; and return periods of 2475 and 43 years (Source: OpenSHA)

structures, T > 3 s, low probability seismic hazard is dominated by two types of earthquakes: (1) Relatively-large-magnitude earthquakes at short distance, and (2) extremely-large-magnitude earthquakes at long distance. Higher probability seismic hazard is dominated by a mixture of seismic events. In the figure, ε is the number of standard deviations by which an observed logarithmic spectral acceleration differs from the mean logarithmic spectral acceleration of a ground-motion attenuation equation.

These hazard characteristics are used as a basis for design and for selection and modification of ground motions for the study. Code-based designs consider the Design Basis Earthquake (DBE), defined as two-thirds of the Maximum Considered Earthquake (MCE). Performance-based designs consider seismic hazard at the 43-year return period and at the Maximum Considered Earthquake level according to ASCE 7–05. After the buildings were designed, performance was to be assessed at five hazard levels designated SLE25, SLE43, DBE, MCE, and OVE, corresponding to earthquake shaking hazard with return periods of 25, 43, 475, 2,475, and 4,975 years, respectively. The 4975-year level is an extremely rare event usually not considered in design and analysis of buildings in California.

Various options for selecting and scaling earthquake ground motions for response history analysis exist (e.g., ATC-82 2011). Performance-based designs required ground motions scaled to the Maximum Considered Earthquake Level. For this purpose, recorded earthquake ground motions were spectrum-matched by modifying them in the time domain such that the resulting 5 %-damped linear response

spectrum closely matched the Maximum Considered Earthquake response spectrum defined by ASCE 7–05 over a period range 0.2–1.5 T, where T is the calculated first-mode vibration period. Spectrum matching is permitted by the TBI Guidelines, and was selected because it was judged the most efficient approach for obtaining expected values for design.

For performance assessment of the designed buildings, recorded earthquake ground motions were amplitude-scaled to provide a best fit with site-specific uniform hazard spectra for the various hazard levels. Amplitude scaling was selected to retain more of the record-to-record variability than would be obtained by spectrum matching. Uniform hazard spectra were selected as the targets rather than conditional mean spectra (Baker and Cornell 2006) because of three considerations: (1) Different engineering demand parameters (e.g., drift, acceleration, shear) are affected differently by multiple response modes, such that multiple conditional mean spectra would have been required; (2) The study involved multiple buildings designed by multiple methods, such that there was no single vibration period to which the conditional mean spectrum could be assigned; and (3) The site was strongly affected by multiple types of seismic events. Results obtained using the uniform hazard spectra are likely to exceed those obtained using conditional mean spectra, and likely are conservative. The appropriate use of conditional mean spectra for tall buildings in complex seismic environments continues as an important subject for research at the time of this writing.

For building performance assessments, 15 pairs of horizontal ground motion records were selected from a sub-set of the PEER NGA database (PEER 2005) for each hazard level. The subset database included ground motion records that excluded aftershocks, had a maximum site-to-source distance of 100 km, and were recorded from soil profile with average shear-wave velocity in top 30-m of soil between 180 and 1,200 m/s. The selected records had a long-period filter cutoff significantly longer than the expected fundamental period of the structure. The selected ground motion records were then amplitude-scaled to the target spectrum at each hazard level. The records were limited to have a maximum scaling factor of 5. Although tighter selection and scaling criteria were desirable, this proved not attainable given the available records and the period range for the buildings under consideration.

Available recorded ground motions were insufficient to populate the data set required for the extremely rare OVE hazard level. Thus, eight pairs of amplitude-scaled recorded ground motions were supplemented by seven pairs of synthetic ground motions to complete the OVE set. The latter were obtained from a database of 648 sites from a single simulation of the Puente Hills fault for the Los Angeles region (Graves and Somerville 2006). These ground motions are hybrid broadband signals ($f = 0.0 - 10.0$ Hz); in the low-frequency range ($f < 1.0$ Hz) a 3-dimensional finite difference model that simulates fault rupture, wave propagation to a site, and site response is used, whereas for the high-frequency range ($f \geq 1.0$ Hz) a stochastic method for ground motion simulation is used. The seven records selected are the best match to the OVE uniform hazard spectrum.
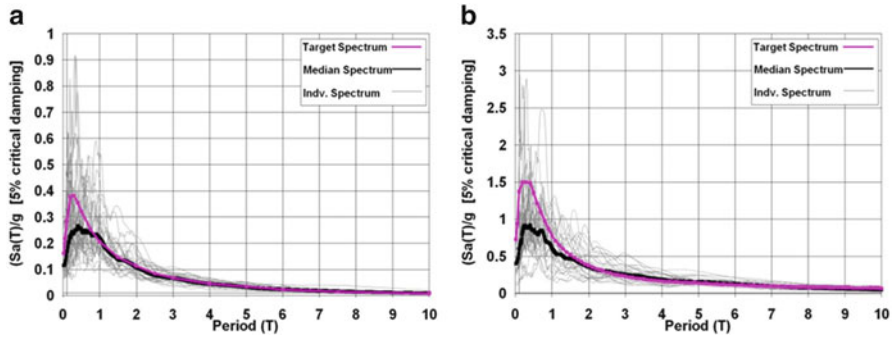
Fig. 26.3 Target and scaled spectra at (**a**) 43-year return period and (**b**) MCE

Figure 26.3 shows the median of the scaled spectra and target spectra for the SLE43 and MCE hazard levels. The median matches the target spectrum reasonably well in the medium to long period range of interest for all hazard levels.

26.5 Case Study Building Designs

Three different high-rise building configurations were selected for study (Fig. 26.4). The 42-story concrete core-wall residential building had four additional stories below grade and used a centrally located concrete core wall with coupling beams for seismic resistance, with unbonded post-tensioned slabs supported by concrete columns and the core wall as the gravity and diaphragm system. The building had typical floor height of 9.67 ft and floor area of approximately 9,000 square feet for each floor above grade. The 42-story concrete dual core wall/frame system had nominally identical configuration except seismic-force-resisting frames replaced gravity slab-column frames along two bays in each principal direction. The 40-story tall steel buckling-restrained braced frame office building had four basement levels and used buckling-restrained braced frames for seismic resistance, with steel gravity framing elsewhere. The building had typical floor height of 13.5 ft and floor area of approximately 18,000 square feet for each floor above grade.

Two alternative designs were carried out. These are designated as Design A (code-based) and Design B (performance-based). Design A uses the prescriptive provisions of the International Building Code (IBC 2006), except the height limit was disregarded. Design B uses the performance-based design approach of the TBI Guidelines (TBI 2010) for seismic design of the structural system. For both Design A and Design B, gravity design, wind design, and nonstructural component seismic design comply with the provisions of IBC (2006). Wind loads are according to ASCE 7 (2005).

The designs were done by structural engineering firms experienced in the seismic design of high-rise buildings. For both Design A and Design B, the structural
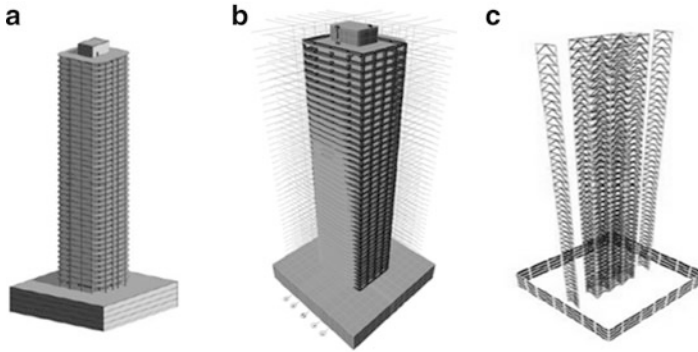
Fig. 26.4 Case study buildings

engineering firm relied on experience to develop conceptual and preliminary designs for the gravity and lateral force-resisting systems. For Design A, the firms' practices, based on extensive experience, resulted in conservative designs for the seismic force-resisting systems compared with minimum requirements of the building code. For Design B, the preliminary designs required iterations to arrive at acceptable configurations and proportions. The analysis for the Maximum Considered Earthquake Level required development of site-specific response spectra and selection of earthquake ground motions by an engineering seismologist; according to design criteria developed by the project team, these records were spectrum matched to the site-specific spectrum over periods ranging from 0.2 to 1.5 T, where T is the estimated building period (5 s). Nonlinear models of the structural system were implemented in PERFORM-3D (CSI 2009). These models included the structural walls and a representation of the gravity framing in accordance with the modeling recommendations of ATC-72 (2010). The models were subjected to each of the seven pairs of scaled earthquake ground motions to verify acceptable response, without need for additional design iterations to meet the performance criteria. A typical performance-based design of a tall building requires additional design and review effort compared with the prescriptive building code design. It is not unusual to estimate 500 h of additional engineering effort and three to four months of additional design/review time relative to a prescriptive code-based design (Fry and Hooper 2011).

Among many differences in the building designs, the salient features were as follows: For the core-wall building, base shear for Design A (4,600 kips) was lower than that for Design B (8,200 kips) because of the serviceability requirements in the latter case. Because design was governed by wall shear, required wall thickness was greater for Design B (32 in.) than Design A (24 in.). Wall vertical reinforcement also was greater for Design B., Conversely, the allowance in the TBI Guidelines for demand-capacity ratio of 1.5 for ductile actions resulted in weaker coupling beams in Design B than Design A. The overall effect was a stiffer building with stronger wall piers and weaker coupling beams for Design B than Design A. The

Table 26.1 Initial construction costs (in million $US)

Building	Structural cost		Total cost (structural, nonstructural, contents)	
	Design A	Design B	Design A	Design B
Core wall	126	128	140	143
Dual core wall/frame	134	159	149	174
Buckling-restrained braced frame	276	268	341	333

dual core wall/frame system required similar design modifications. In addition, a capacity design approach used in Design B resulted in much larger columns than in Design A. For the buckling-restrained braced frame system, an entirely different structural system involving steel braced frame outriggers was required for Design B that was not required for Design A. However, Design A required very large column sizes because of code prescriptive requirements regarding summing brace forces over the building height to determine column axial forces. For complete details, see the appendices to Moehle et al. (2011b).

The initial construction costs of the designs were estimated by an experienced professional cost estimator in California (Moehle et al. 2011b). The initial construction costs (including design and management fees) are listed in Table 26.1. The values given are for above-grade construction only. In loss studies, reported later in this paper, only the above-grade portions were deemed susceptible to damage.

26.6   Analytical Models for Design and Performance Assessment

Analytical models were developed using computer software PERFORM-3D (CSI 2009). The seismic force-resisting systems extended down to the foundation level, which was modeled as rigid. Axial and bending interaction of the core walls was modeled using inelastic fiber elements accounting for longitudinal reinforcement and confined concrete (cover concrete ignored). In-plane shear behavior of the wall was modeled using elastic shear stiffness and an inelastic shear spring with strength equal to $1.5V_n$ (where factor 1.5 is intended to achieve expected strength when applied to nominal strength V_n calculated using ACI 318 (2011)). Coupling beams were modeled using two elastic beam-column elements connected at midspan by a nonlinear shear hinge. Seismic frames were modelled with the usual assumptions considering flexure (nonlinear), shear, and axial flexibilities. Buckling restrained braced frames were modelled considering axial, bending, and shearing flexibilities of beams and columns, and axial flexibilities (nonlinear) of braces. Basement perimeter walls were modeled using elastic shear wall elements with a stiffness reduction factor of 0.8 to account for concrete cracking. A rigid diaphragm was achieved for all levels above ground by "slaving" the horizontal translation degrees

of freedom. Slabs in basement levels were modeled using elastic shell elements with a stiffness reduction factor of 0.25 to account for concrete cracking. P-delta effects were taken into account in the model by applying tributary loads to the lateral force-resisting system with remaining loads applied to a dummy column with no lateral stiffness. Floor mass was assigned as lumped mass at each floor. Gravity framing was found to have negligible influence on stiffness and strength, and was excluded from the analytical models.

26.7   Dynamic Response of the Case Study Buildings

Analytical models of each building were subjected to the 15 pairs of earthquake ground motions at each of five hazard levels described previously. Figure 26.5 presents a glimpse of the results, in this case showing nominal compressive strain in one corner location of the core wall in the core-only building. In this case, the code-designed building sustains greater compressive strain than does the
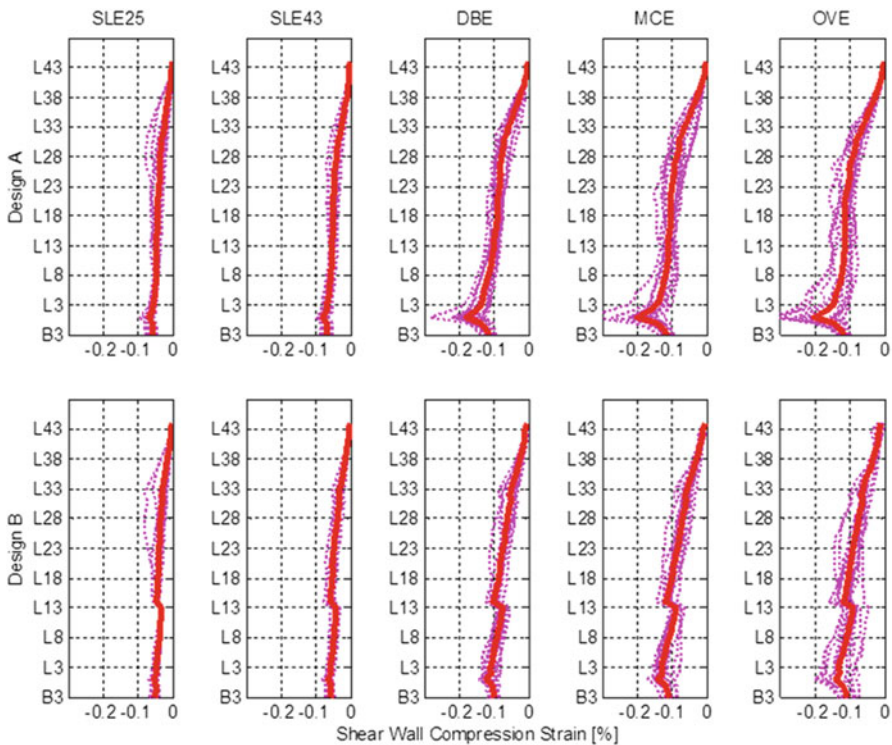

Fig. 26.5 Calculated maximum compressive strain in one corner location of the core wall in the core-only buildings

performance-based design. The engineering demand parameters were not always "worse" for the code-based design than for the performance-based design, making it difficult to judge the overall performance based solely on engineering demand parameters. The repair cost assessment described in the next section integrates the effects of engineering demand parameters on damage and repair costs, facilitating a more comprehensive performance assessment.

26.8   Financial Implications of the Different Design Methods

Using the results from the nonlinear dynamic analyses, the statistical distributions of the structural response were sampled at each of the five hazard levels. The ATC 58 calculation tools were then implemented to generate large numbers of engineering demand parameters (EDPs). The values of the EDPs were then used to assess the damage states of the components. Once the damage states for all components were identified, the repair actions and repair cost for each component were obtained from a look up table (ATC-58 2012). The total repair cost for the entire building was then summed over all the components in the building. Buildings of this size require large amounts of data to define the repair costs. For example, the core wall-only buildings had 1,765 performance groups representing the structural, nonstructural, and contents items deemed susceptible to shaking damage, and each group had to be sampled for each of several thousand realizations to establish repair cost statistics. Fortunately, available software automates the procedures.

It is not possible to show all the repair cost data for all the buildings. Instead, here we focus only on the core wall-only building. Figure 26.6 shows the deaggregation of median total repair costs for each Performance Group type and each hazard level. Performance Groups for core-wall webs and core-wall boundary elements have been combined, and elevators are not shown because the cost is relatively low. For SLE25 shaking intensity level, the repair costs are concentrated in the contents
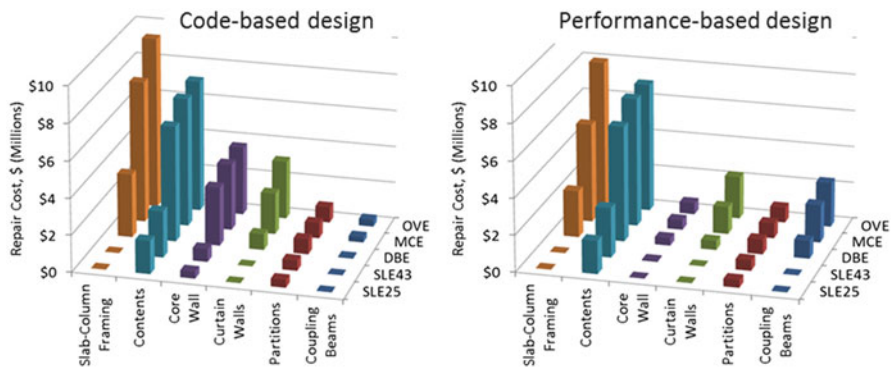

Fig. 26.6 Deaggregation of median repair costs. Core-only building

Table 26.2 Median repair costs for the buildings, normalized to initial construction cost

	43-year return period		2475-year return period	
Building	Code design (%)	Performance design (%)	Code design (%)	Performance design (%)
Core wall	2.9	2.4	15.9	12.4
Dual core wall/frame	2.9	2.0	12.4	10.4
Buckling-restrained braced frame	0.7	0.5	3.1	2.8

and interior partitions, although Design A also sustains some core wall repair costs. As the shaking intensity increases to SLE43, the distribution of repair costs remains similar to SLE25. As shaking intensity reaches DBE, repair costs for slab-column framing and the core wall accelerate for Design A, while repair costs for coupling beams accelerate for Design B. The higher damage for the thinner walls of Design A and higher damage for the weaker coupling beams of Design B is consistent with expectations based on the design results. As the shaking intensity reaches MCE, repair costs for slab-column framing continue to increase for both designs. Curtain wall repair costs also begin to pick up. Contents repair costs have nearly saturated and do not show significant increase with increasing intensity. Trends continue for the OVE.

The relatively high repair costs for slab-column framing is an unexpected result from the analysis. It is possible that fragility relations or repair costs used in the analysis require adjustment. However, the results were reviewed with members of the ATC-58 (2012) project team and confirmed to be reasonable. Assuming the results to be indicative of actual conditions, such results can provide useful information to the structural engineer on a project by indicating where to focus design attention for the purpose of reducing potential damage and repair cost. The results also might suggest revisions in design codes to reduce losses associated with especially vulnerable components, or reduce requirements for more rugged components.

The median repair costs for the buildings at different earthquake hazard levels were calculated for each of the building designs. Results for the 43-year and 2475-year return periods, normalized to initial construction cost, are listed in Table 26.2. Normalized losses for the performance-based designs were, in general, smaller than for the code-based designs, although the results are not drastically different. The buckling-restrained braced frame had the smallest costs. It is noted, however, that residual drift, which might have significant impact to the total repair cost of the buckling-restrained braced frame building, was not included in the calculations.

Table 26.3 summarizes the mean annualized repair cost for the buildings. This number represents the average repair cost per year for all buildings, considering all hazard levels. The results show that the core only building had highest mean annualized repair cost followed by the dual system and then the buckling-restrained braced frame. In general, the mean annualized repair cost decreased as the design shifted from the code-based design to the performance-based design.

Table 26.3 Mean annualized repair costs for the buildings in $US

Building	Code design	Performance design
Core wall	326,000	282,000
Dual core wall/frame	323,000	269,000
Buckling-restrained braced frame	206,000	141,000

Table 26.4 Ratio of total costs

Building	Code design	Performance design
Core wall	1.0	1.0
Dual core wall/frame	1.0	0.96
Buckling-restrained braced frame	1.0	0.97

If we assume the mean annualized repair costs are equivalent to required insurance premiums to be paid annually, we can calculate the net present value of the insurance premiums given a payment period and assumed time value of money. Here we assumed a period of 50 years and interest rate of 0.03. Total Cost can be defined as the initial construction cost plus the net present value of insurance premiums. Table 26.4 compares Total Costs of the Performance-Based Design relative to the Total Costs of the Code Based Design for each building. The results indicate that Total Costs are equal or less for the performance-based designs. Note that these results are insensitive to the assumed time value of money. Note also that the performance-based designs were not oriented toward optimization of Total Cost, but instead were oriented toward more reliable performance by more explicit representation of the building properties in the design process. Furthermore, there was no intent of the performance-based designs to achieve superior performance but, rather, to more reliably achieve Occupancy Category II performance objectives.

26.9   Summary and Conclusions

This paper served to introduce two guidelines recently introduced in U.S. practice, one for performance-based designs of tall buildings and another for the seismic performance evaluation of building designs. The methods are demonstrated through the design and performance assessment of three tall building configurations, each designed according to code-based procedures and according to performance-based procedures. Principal conclusions are:

• The code-based and performance-based designs had notable differences in structural component sizes. The performance-based designs appeared allocate materials more appropriately.

- The code-based and performance based designs had notable differences in seismic performance, but it was difficult to know which performance was superior based solely on the engineering demand parameters and apparent damage. By aggregating the repair costs, it became apparent that the performance-based designs had equal or lesser repair costs. Combining initial costs and annualized repair costs, it was clear that the performance-based designs were slightly more efficient than the code-based designs.

Overall, it is concluded that the performance-based approach is usable and produces acceptable designs. The ATC 58 seismic performance assessment methodology is useful for measuring relative performance of individual buildings and thereby may be useful in deciding among design options. It also can serve as a useful tool to optimize building code requirements.

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