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SHEAR DESIGN OF STRUCTURAL WALLS FOR TALL REINFORCED CONCRETE CORE WALL BUILDINGS

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Abstract

The Los Angeles Tall Buildings Structural Design Council (LATBSDC) and Pacific Earthquake Engineering Research Center Tall Buildings Initiative (PEER TBI) performance-based design guidelines for structural wall shear design for tall reinforced concrete core wall buildings are reviewed. Provisions in the two documents are nearly identical, except for the recommended capacity reduction factor ϕ, which is 1.0 for the LATBSDC document and 0.75 for the PEER TBI document. In this study, reliability methods were used to assess the current shear design criterion; various statistical parameters were examined for shear demand and shear controlled test results and ACI318-11 code provisions were reviewed to establish statistical parameters for shear capacity. The results suggest that use of ϕ=1.0, along with the use of appropriate expected material properties, produces an acceptable probability of failure, whereas the use of ϕ=0.75 appears excessively conservative. However, due to a lack of experimental tests on walls that yield in flexure, limitations on curvature ductility or plastic rotation demands are recommended in the plastic hinge regions to avoid potential degradation in shear capacity.

Keywords: reinforced concrete shear walls; shear design; tall core wall buildings; performance-based design

1. Introduction

Along the west coast of the United States, reinforced concrete core wall systems are commonly selected as seismic force resisting systems for tall buildings. During strong ground shaking, core wall systems are intended to dissipate energy by yielding of coupling beams, followed by flexural yielding at the wall base. Although the wall behavior is governed by flexure, the wall design is often governed by shear as the walls experience high shear demands, usually up to the ACI318-11 [1] code limiting shear stress of $8\cdot\sqrt{(f'_c)}$ psi, over a significant height of the core. The high shear demands are due to a lack of redundant walls in tall buildings, as the lengths of the walls are limited to the perimeter of the elevator core.

The design procedures for tall buildings are typically conducted using performance-based design procedures recommended by Los Angeles Tall Buildings Structural Design Council (LATBSDC, [2]) or Pacific Earthquake Engineering Research Center Tall Buildings Initiative (PEER TBI, [3]). Provisions in the two documents recommend shear design per acceptance criterion

$$\mathbf{F_{uc} \leq \kappa_i \phi F_{n,e}} \tag{1}$$

where F_{uc} is 1.5 times the mean shear demand resulting from a suite of ground motions, $F_{n,e}$ is the nominal strength computed from expected material properties, κ_i is the risk reduction factor based on risk categories, and ϕ is the uncertainty in $F_{n,e}$. The 1.5 factor applied to the mean shear demand is referred to as the demand factor, γ. Although shear failure can be fatal due to its sudden and brittle nature, the reliability of this shear design acceptance criterion has not yet been thoroughly researched. Moreover, there is a lack of consensus in the governing codes and tall building guidelines regarding the use of γ, κ_i, and ϕ factors. Given the importance of structural walls in tall reinforced concrete core wall buildings (as the main seismic force resisting system, along with coupling beams), these issues have served as a motivation to pursue this research. The limitations of the current shear design acceptance criterion are summarized as follows.

1. The demand factor, γ =1.5, is an empirical factor established to achieve conservatism in shear design [3].
2. There are discrepancies in ϕ recommendations, where LATBSDC recommends ϕ =1.0 and PEER TBI recommends ϕ =0.75.

This study is a continuation of works by Wallace et al [4]; Wallace et al [4] conducted preliminary studies to assess reliability of structural wall shear design acceptance criteria by extending works by Hamburger [5].

2. Performance-Based Design

Performance-based design of tall, reinforced concrete core wall buildings is commonly achieved in three stages, as recommended by LATBSDC. First, an initial design is created based on experience to proportion members and apply capacity design concepts. In this step, the height limitations set forth by ASCE 7-10 [6], either 160 ft or 240 ft depending on the system, are ignored. Next, the adequacy of the initial design is demonstrated by evaluating the performance of the building at both service and collapse prevention hazard levels using acceptance criteria established in an approved, project-specific "Basis of Design." It is common to conduct the service-level assessment first using linear response spectrum analysis, and then adjust the member proportions based on experience prior to conducting the MCE analysis. The MCE analysis requires developing a three-dimensional nonlinear model subjected to seven or more pairs of earthquake records. Typical adjustments might involve designing for a wall shear stress of 2.0 to 2.5 times the wall shear force obtained in the SLE evaluation, and modifying coupling beam strengths to be more uniform over the building height.

The structural models are recommended to incorporate realistic estimates of strength and stiffness properties for all materials and components; therefore, expected material properties are utilized instead of nominal properties (Table 1) and different reinforced concrete stiffness parameters (Table 2) are recommended for SLE and MCE hazard levels. Although the current recommendation of using $1.3 \cdot f'_c$ for expected concrete compressive strength is generally appropriate for normal strength concrete; a lower value of $1.1 \cdot f'_c$ has been shown to be appropriate for $6\text{ksi} < f'_c \leq 12\text{ksi}$ based on information reported by Nowak et al [7] on over 2000 concrete samples. The lower than expected concrete strengths are typically a result of the softer aggregates used in the western states of the United States, which also has been shown to produce lower than expected values of Modulus of Elasticity [2].

Table 1. Expected material strengths [2]

Material	Expected strength
Yield strength for reinforcing steel	$1.17 \cdot f_y$
Ultimate compressive strength for concrete	$1.3 \cdot f'_c$

Table 2. Reinforced concrete stiffness properties [2]

Element	SLE and wind	MCE
Structural walls	Flexural – $0.75 \cdot I_g$ Shear – $1.0 \cdot A_g$	Flexural – $1.0 \cdot E_c$ Shear – $0.5 \cdot A_g$
Basement walls	Flexural – $1.0 \cdot I_g$ Shear – $1.0 \cdot A_g$	Flexural – $0.8 \cdot I_g$ Shear – $0.5 \cdot A_g$
Coupling beams	Flexural – $0.3 \cdot I_g$ Shear – $1.0 \cdot A_g$	Flexural – $0.2 \cdot I_g$ Shear – $1.0 \cdot A_g$
Diaphragms (in-plane only)	Flexural – $0.5 \cdot I_g$ Shear – $0.8 \cdot A_g$	Flexural – $0.25 \cdot I_g$ Shear – $0.25 \cdot A_g$
Moment frame beams	Flexural – $0.7 \cdot I_g$ Shear – $1.0 \cdot A_g$	Flexural – $0.35 \cdot I_g$ Shear – $1.0 \cdot A_g$
Moment frame columns	Flexural – $0.9 \cdot I_g$ Shear – $1.0 \cdot A_g$	Flexural – $0.7 \cdot I_g$ Shear – $1.0 \cdot A_g$

2.1 Analysis Procedure

When response spectrum analysis is used, the structure is evaluated using the following load combinations:

$$1.0\cdot D + L_{exp} + 1.0\cdot E_x + 0.3\cdot E_y \tag{2}$$

$$1.0\cdot D + L_{exp} + 1.0\cdot E_y + 0.3E_x \tag{3}$$

where D is the service dead load, L_{exp} is the expected service live load taken as 25% of the unreduced live load, and E_x and E_y represent the earthquake loads in X and Y directions. To calculate responses for each horizontal direction, at least 90 percent of the participating mass of the structure should be included, and the Complete Quadratic Combination (CQC) is recommended for modal response calculations. When nonlinear response history analysis is performed, the following load combination for each horizontal ground motion pair is used:

$$1.0\cdot D + L_{exp} + 1.0\cdot E \tag{4}$$

where E represents the dynamic earthquake loads.

2.2 MCE Level Analysis – Global Acceptance Criteria

The global response acceptance criteria under MCE analysis include evaluations of story drift, residual drift, and loss of story strength. For peak transient drift, the following criteria must be met:

$$\bar{\Delta} \leq 0.03 \cdot \kappa_i \tag{5}$$
$$\Delta \leq 0.045 \cdot \kappa_i \tag{6}$$

where $\bar{\Delta}$ is the mean of the absolute values of the peak transient drift ratios from the suite of analyses, Δ is the absolute value of the maximum story drift ratio from any analysis, and κ_i is the risk reduction factor. Risk reduction factors are 1.0 for risk categories I and II, 0.80 for risk category III, and for risk category IV, the value is determined by the seismic peer review panel. For residual drift, the following criteria must be met:

$$\bar{\Delta}_r \leq 0.01 \cdot \kappa_i \tag{7}$$
$$\Delta_r \leq 0.015 \cdot \kappa_i \tag{8}$$

where $\bar{\Delta}_r$ is the mean of the absolute values of residual drift ratios from the suite of analyses and Δ_r is the maximum residual story drift ratio from any analysis. In any nonlinear response history analysis, deformations shall not result in a loss of any story strength that exceeds 20% of the initial strength. Modeling story strength loss for RC core wall buildings using commercial computer programs often leads to non-convergence; therefore, actual modeling of strength loss is rare.

2.3 MCE Level Analysis – Component Acceptance Criteria

All component-level responses and acceptance criteria are classified as either force-controlled or deformation-controlled. Force-controlled actions reflect brittle behavior where reliable inelastic deformation cannot be obtained. The design acceptance criterion from equation (1) applies and per

LATBSDC Section C3.5.4.1.1(a), κ_i is applied for deformation-controlled acceptance criteria but is not considered for force-controlled actions. However, this approach is not necessarily widely accepted, as discussed in the introduction.

Conversely, deformation-controlled actions refer to ductile behavior where reliable inelastic deformation can be obtained with no substantial strength loss. For deformation-controlled actions, the mean responses or member deformations are evaluated against project specific acceptable criteria (Basis of Design) multiplied by κ_i. The project specific acceptance criteria are usually established by referencing appropriate publications (journal papers or technical reports), material specific codes, or Primary Collapse Prevention values published in ASCE41 [8] for nonlinear response procedures.

3. Reliability of Structural Wall Shear Design

3.1. Background

In United States, probability-based limit state design (PBLSD) is adopted in many material-specific codes to establish design acceptance criteria for structural components. PBLSD examines reliability of a structural component by calculating the probability of component failure due to demands exceeding the component capacity, C < D. Although the terminology used in PBLSD is based on load (Q) and resistance (R), load will be referred to as demand, resistance will be referred to as capacity, and limit states will be referred to as acceptance criteria to be consistent with capacity design terminology used in LATBSDC. Per PBLSD, failure is defined as

$$\mathrm{P_f} = \mathrm{P(C < D)} = \int_{-\infty}^{+\infty} \mathrm{F_C(q_i) f_D(q_i) \cdot dq_i} \tag{9}$$

where C is capacity, D is demand, F_C is cumulative probability distribution function of C and f_D is probability density function for D. In practice, rather than using the integral to compute probability of failure, probability of failure is calculated indirectly with a reliability index, β through closed-form solutions [9], [10]. For seismic events conditioned upon MCE level ground shaking, the anticipated reliabilities per ASCE7-10 are 90% for risk categories I and II, 94% for risk category III, and 97% for risk category IV.

In the following sections, full distribution methods to compute probability of failure with β are briefly explained for the two cases where random variables C and D follow normal and lognormal distributions.

3.2 Normal Distribution for Random Variables C and D

When random variables C and D are jointly normal, it is convenient to consider safety margin, which is defined as F = C – D. Since C and D are normal random variables and F is a linear combination the two, F is also a normal random variable. In this case, a closed-form solution can be used to calculate probability of failure, where $P_f[F \leq 0] = \Phi[-\beta]$ and the reliability index is defined as an inverse coefficient of variation of the safety margin:

$$\beta = \frac{\bar{F}}{\sigma_F} = \frac{\bar{C} - \bar{D}}{\sqrt{\sigma_C^2 + \sigma_D^2 - 2 \cdot \rho_{CD} \cdot \sigma_C \cdot \sigma_D}} \tag{10}$$

where, $\bar{F}, \bar{C}, \bar{D}, \sigma_F, \sigma_C, \sigma_D$ are expected values and standard deviations of safety margin, capacity, and demand, respectively, and ρ_{CD} is correlation between capacity and demand. In the case where the random variables C and D are statistically independent ($\rho_{CD} = 0$),

$$\beta = \frac{\bar{C} - \bar{D}}{\sqrt{\sigma_C^2 + \sigma_D^2}} \tag{11}$$

An equivalent representation of the reliability index can be expressed through an introduction of a demand factor and coefficient of variation for capacity and demand

$$\beta = \frac{1 - \left(\frac{1}{\gamma}\right)}{\sqrt{\rho_C^2 + \left(\frac{\rho_D^2}{\gamma^2}\right)}} \tag{12}$$

where demand factor and coefficient of variation for capacity and demand are defined as

$$\gamma = \frac{\bar{C}}{\bar{D}}, \rho_C = \frac{\sigma_C}{\bar{C}} \text{ and } \rho_D = \frac{\sigma_D}{\bar{D}} \tag{13}$$

3.3 Lognormal Distribution for Random Variables C and D

When random variables C and D are jointly lognormal, it is convenient to consider safety factor, which is defined as

$$F = \frac{C}{D} \tag{14}$$

A random variable is defined to be lognormally distributed when its logarithm is normally distributed. Thus, new normal random variables, X=ln(C), Y=ln(D), and Z=ln(F), are introduced and safety factor can be expressed with a normal random variable Z, where

$$\ln(F) = \frac{\ln(C)}{\ln(D)} \tag{15}$$

$$Z = X - Y \tag{16}$$

In this case, a closed-form solution can be used to calculate probability of failure, where $P_f[Z \leq 0] = \Phi[-\beta]$ and the reliability index is expressed as inverse coefficient of variation of Z

$$\beta = \frac{\bar{Z}}{\sigma_Z} = \frac{\bar{X} - \bar{Y}}{\sqrt{\sigma_X^2 + \sigma_Y^2 + 2 \cdot \rho_{XY} \cdot \sigma_X \cdot \sigma_Y}} \tag{17}$$

The reliability index can be rewritten as

$$\beta = \frac{\ln\left(\gamma \cdot \sqrt{\frac{1+\rho_D^2}{1+\rho_C^2}}\right)}{\sqrt{\ln(1+\rho_C^2) + \ln(1+\rho_D^2) - 2 \cdot \rho_{\ln C, \ln D}\sqrt{\ln(1+\rho_C^2)}\sqrt{\ln(1+\rho_D^2)}}} \tag{18}$$

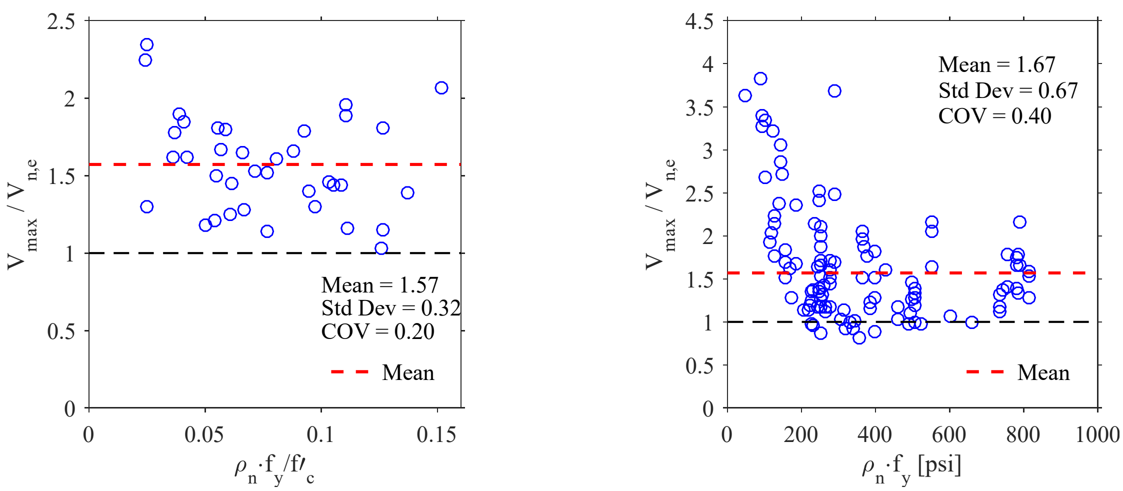
and when random variables C and D are statistically independent,

$$\beta = \frac{\ln\left(\gamma \cdot \sqrt{\frac{1+\rho_D^2}{1+\rho_C^2}}\right)}{\sqrt{\ln(1+\rho_C^2) + \ln(1+\rho_D^2)}} \tag{19}$$

3.4 Shear Demand and Capacity

To evaluate reliability of structural wall shear design, various statistical parameters were established for shear demand and capacity. Shear demand was normalized at 1.0, under the assumption that structural walls were designed per ACI318-11 and LATBSDC guidelines. Various shear demand dispersion values (ρ_D, measured as coefficient of variation) between 0.2 and 0.6 were considered, and both lognormal and normal distributions were evaluated [11]. To establish statistical parameters for shear capacity, compilations of shear-controlled wall tests were referenced from Wallace [12] and Wood [13]. Wallace examined 37 shear-controlled walls with concrete compressive strengths greater than 8ksi and Wood examined 143 shear-controlled walls with concrete compressive strengths between 2ksi and 8ksi. A plot of variations in ratio between maximum shear achieved by test and nominal shear strength calculated with expected properties, $V_{max}/V_{n,e}$ versus reinforcement is shown on Figure 1. From Wallace's data, all 37 specimens had $V_{max}/V_{n,e}$ between 1.0 and 2.5, with a mean $V_{max}/V_{n,e}$ of 1.57 and coefficient of variation of 0.20. The data were observed to follow normal distribution. From Wood's data, the maximum shear strengths of the walls were also mostly high with a mean $V_{max}/V_{n,e}$ of 1.67 and a coefficient of variation of 0.40. The data were observed to follow lognormal distribution. The dispersion measured from data presented by Wallace and Wood derives from uncertainties in nominal shear strength prediction equation, material strengths including concrete compressive strengths and reinforcing steel yield strengths, construction quality, test setup, test measurement, and other possible errors. Due to high quality measures enforced for tall building design and construction through seismic peer review panels [2] and required inspections, no further dispersion was added to the measured values. All combinations of statistical parameters considered for shear demand and capacity are noted on Figure 2, and demand and capacity were assumed to be independent random variables.

It is important to note the context in which these shear controlled wall tests were used. When a tall concrete core wall building undergoes a seismic hazard, the coupling beams are intended to yield first, followed by the flexural yielding of the structural walls. Currently, it is common practice to limit the amount of flexural yielding in walls by restricting axial strains as (a) 0.01 and 2 times yielding strain for tensile strains within and outside of plastic hinge zones, respectively, and (b) 0.0075 and 0.003 for 1.5 times compressive strains within and outside of plastic hinge zones, respectively. When flexural yielding is limited in structural walls, no significant shear degradation is expected and full shear capacity can be assumed; this is the context in which shear controlled wall tests were used to conduct reliability studies in the next section. On the other hand, when flexural yielding exceeds the axial strain limits stated above, the shear capacity may start to degrade. Currently, there are limited test data that can quantify the rate of shear strength degradation associated with increasing nonlinear flexural yielding. Thus, further dynamic tests are needed to examine shear strength degradation with nonlinear flexural yielding including higher mode contributions. Moreover, core wall tests in biaxial loading would be helpful to understand how shear capacities may change due to varying shapes of the compressive zones. When flexural yielding exceeds recommendations of 0.01 for tensile strains within plastic hinge zone and 0.0075 for 1.5 times compressive strains within the plastic hinge zone, a lower ϕ=0.75 should be used to account for the uncertainties in shear capacity.



(a) (b)

Figure 1. Variation in measured shear strength to nominal shear strength ratio versus reinforcement, using data from (a) Wallace, $f'_c \geq$ 8ksi and (b) Wood, $f'_c <$ 8ksi

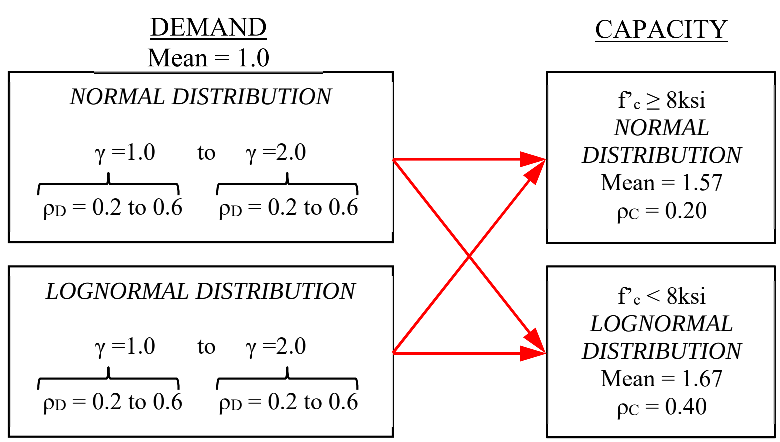
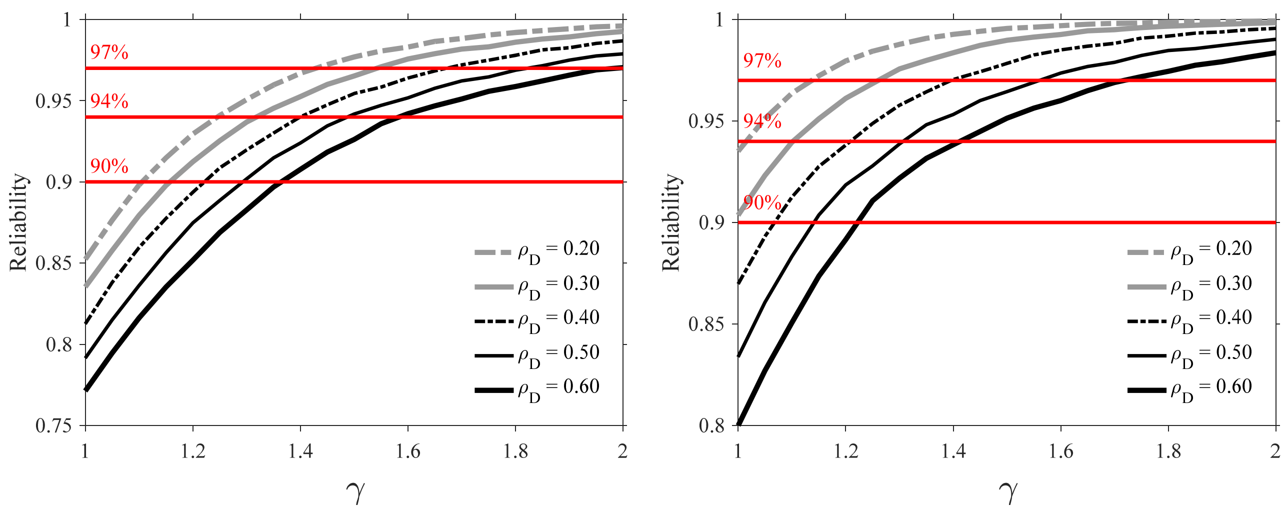


Figure 2. Combinations of shear demand and capacity statistical parameters considered

3.5 Structural wall shear design reliability

Reliability of structural wall shear design was computed according to the methodology presented in aforementioned sections. The results using Wood's data are presented on Figure 3(a) as reliability versus γ at. Both normal and lognormal distributions were evaluated and the more conservative case was plotted for each combination of γ and ρ_D. The same procedures were used to plot reliability results using Wallace's data on Figure 3(b). The current recommended shear design acceptance criterion using γ=1.5, ϕ=1.0, κ_i =1.0 and ρ_D = 0.50, resulted in 94.2% reliability for structural walls with f'_c < 8ksi and 96.5% reliability for structural walls with $f'_c \geq$ 8ksi.



(a) (b)

Figure 3. Shear design reliability for structural walls with (a) f'_c < 8ksi (b) $f'_c \geq$ 8ksi

4. Conclusions

Reliability of structural wall shear design for tall reinforced concrete core wall buildings was examined for various risk categories. Statistical parameters were established for shear demands and capacities and closed-form solutions were used to evaluate reliability of the current shear design acceptance criterion set forth by LATBSDC. Based on results, the following conclusions were drawn:

- The current recommended shear design acceptance criterion using γ=1.5, ϕ=1.0, κ_i =1.0 and ρ_D = 0.50, resulted in 94.2% reliability for structural walls with f'_c < 8ksi and 96.5% reliability for structural walls with $f'_c \geq$ 8ksi.
- The results suggest that use of ϕ=1.0, along with the use of appropriate expected material properties, produces an acceptable probability of failure, whereas the use of ϕ=0.75 appears excessively conservative. However, due to a lack of experimental tests on walls that yield in flexure, limitations on curvature ductility or plastic rotation demands are recommended in the plastic hinge regions to avoid potential degradation in shear capacity.

5. References

[1] ACI318 (2011). "Building Code Requirements for Structural Concrete and Commentary," American Concrete Institute, Farmington Hills, MI.
[2] LATBSDC (2014). "An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region." Los Angeles Tall Buildings Structural Design Council, Los Angeles, CA.
[3] PEER TBI (2010). "Guidelines for Performance-Based Seismic Design of Tall Buildings," PEER-2010/05, Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA.
[4] Wallace, J.W., Segura, C., and Tran, T. (2013). "Shear Design of Structural Walls," 10th International Conference on Urban Earthquake Engineering, Tokyo Institute of Technology, Tokyo, Japan, March 1-2, 2013.
[5] Hamburger, R. (2011). "Reliability Shear Wall Design," Personal Communication. July 22, 2011.
[6] ASCE7 (2010). "Minimum Design Loads for Buildings and Other Structures," American Society of Civil Engineers, Reston, VA.
[7] Nowak, A., Szersen, M. M., Szeliga, E. K., Szwed, A., and Podhorecki, P. J. (2008). "Reliability-Based Calibration for Structural Concrete, Phase 3," Portland Cement Association and Precast/Prestressed Concrete Institute, Skokie, IL.
[8] ASCE41 (2013). "Seismic Evaluation and Retrofit of Existing Buildings," American Society of Civil Engineers, Reston, VA.
[9] Melchers, R. E. (1999). "Structural Reliability Analysis and Prediction," John Wiley and Sons, Inc., New York, NY.
[10] Ditlevsen, O., and Madsen, H. O. (2005). "Structural Reliability Methods," Internet Edition 2.2.5 http://www.mek.dtu.dk/staff/od/books.htm. July, 2005.
[11] Kim, S. (2016). "Reliability of Structural Wall Shear Design for Reinforced Concrete Core Wall Buildings," Doctoral Dissertation, University of California, Los Angeles.
[12] Wallace, J. W. (1998). "Behavior and Design of High-Strength RC Walls," ACI Structural Journal, SP-176, American Concrete Institute, Farmington Hills, MI, pp. 259-279.
[13] Wood, S. L. (1990). "Shear Strength of Low-Rise Reinforced Concrete Walls," ACI Structural Journal, Vol. 87, No. 1, pp. 99-107.