Seismic performance of reinforced concrete core wall buildings with and without moment resisting frames

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SUMMARY:
This study aims to compare the seismic performance of two 42-story reinforced concrete buildings located in Los Angeles, California. Building 1 is a coupled core wall structure and Building 2 is a similar coupled core wall building with perimeter moment resisting frames (i.e. dual system). The designs of the buildings were carried out by two renowned engineering firms in the United States using three design approaches. Approach A follows the traditional code design approach (e.g. IBC 2006), approach B follows the performance-based design procedure published by the LATBSDC, and approach C follows the enhanced performance plus design approach published by the Pacific Earthquake Engineering Center. Nonlinear dynamic responses of these buildings at five levels of earthquake shaking intensities were analyzed using Perform3D. Detailed seismic response comparisons and associated initial and lifetime maintenance costs of these buildings are presented in this paper.

Keywords: concrete core walls, concrete dual systems, seismic performances, tall buildings

1. INTRODUCTION

Reinforced concrete shear wall systems are commonly used as the seismic force resisting system in the West Coast of the United States. For tall buildings, these systems typically utilize a centrally located reinforced concrete core wall to resist most of the seismic forces. This results to relatively small gravity systems at the perimeter of the building. Seismic response of core wall systems has been studied by (Yang et al., 2010). Alternatively, these systems could be designed as dual systems by combining core walls and reinforced concrete moment resisting frames. To compare the seismic performance of reinforced concrete core wall systems with and without the moment resisting frame, a 42-story building located in Los Angeles, California were designed and analyzed. This paper compares the seismic performance of these two systems in terms of initial costs, structural responses, and the expected repair costs at different levels of earthquake shaking intensities.

Building designs were carried out by two renowned engineering firms in the United States using three design approaches. Approach A follows the traditional code design (International Building Code 2006), except the height limit is exceeded for Building 1. Approaches B and C follow the performance-based design procedures published by the Los Angeles Tall Building Structural Design Council (LATBSDC 2008) and the Pacific Earthquake Engineering Research Center Tall Building Initiative (PEER/TBI 2010), respectively.

Three-dimensional, nonlinear finite element models were developed using Perform 3D (CSI, 2006) for both buildings using consistent modeling approaches. Expected material strengths, component strengths and deformation capacities were employed either from ASCE 41-06 Supplement #1 (ASCE, 2007) or values obtained from previous experimental tests. Five hazard levels with return periods of 25, 43, 475, 2475 and 4975 years were selected to represent the earthquakes shaking intensities expected by the buildings. 15 pairs of ground motions were selected for each of the hazard level. Design methodology, modeling criteria, nonlinear response history analysis results, and cost analysis results are presented in the following sections.

2. DESCRIPTION OF THE PROTOTYPE MODELS AND DESIGN METHODOLOGY

Two buildings were designed in this study. Building 1 consisted of a centrally located core with gravity columns at the perimeter. Building 2 consisted of a reinforced concrete core wall surrounded by special reinforced concrete moment frames at the perimeter of the building. Both buildings were composed of L-shaped walls connected with coupling beams. The reinforced concrete core walls were continuous from the foundation to the top of the building. Both buildings had 4 stories podium below grade which was surrounded by 16" thick perimeter shear wall. Diaphragms were added at all floors below grade to transfer the lateral loads to the perimeter basement walls. Figure 1 and Figure 2 show the three-dimensional view and the plan view of the prototype models, respectively.

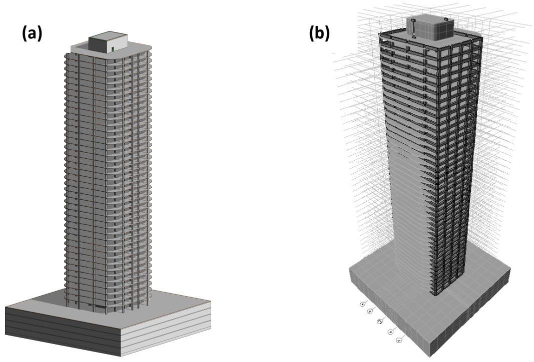

Figure 1. Three-dimensional view of the buildings, (a) Building 1, (b) Building 2

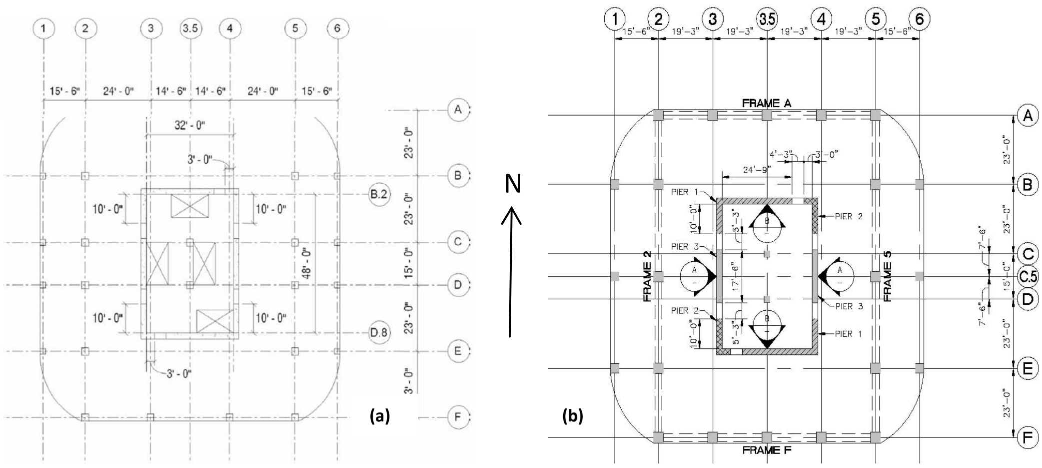

Figure 2. Plan view of the buildings, (a) Building 1, (b) Building 2

2.1. Design of Buildings 1A and 2A (Code-based Design)

Design approach A was based on the design provisions as defined in the International Building Code (IBC) 2006. All provisions of the building code were followed, except the height limit was neglected for Building 1A. The International Building Code has a height limit of 160ft for reinforced concrete core-wall only systems. However there is no specified height limit for dual systems. The member sizes were designed using the modal response spectrum method with a 5% damping site-specific response spectra.

2.2. Design of Buildings 1B and 2B (Performance-based Design)

Buildings 1B and 2B, which had the same layout and floor plan as buildings 1A and 2A, were designed by following the performance-based design procedures published by the Los Angeles Tall

Building Structural Design Council (LATBSDC, 2008). The buildings were designed and checked for serviceability and collapse prevention performance levels. Serviceability level design was checked against the 25-year return period earthquake with 2.5% viscous damping, using linear response spectrum analysis. The overall inter-story drift was not allowed to exceed 0.5% of the story height, and the structural elements were designed to have effectively linear response. The structure was then checked for the maximum considered earthquake (MCE) level using nonlinear dynamic analysis. Specific force and deformation limits are summarized in the report published by Moehle et al. (2011).

2.3. Design of Buildings 1C and 2C (Enhanced Performance-based Design)

Design approach C utilized the performance-based plus design guidelines presented by the Pacific Earthquake Engineering Research Center – Tall Buildings Initiative. (PEER/TBI, 2010). The buildings were designed with higher performance objective including a serviceability analysis check using the 43-year earthquake with 2.5% viscous damping. Ductile elements were allowed to reach 150% of their capacity and the brittle elements to have a demand-to-capacity ratio less than 1.0. Designers of Building 2C observed that Building 2B already satisfied the requirements of the enhanced performance design; thus, a new design was decided to be unnecessary. Therefore, Building 1 had three different design cases; whereas Building 2 had only two variations. This paper compares only the code-based (Design A) and enhanced performance-based designs (Design C) of the two buildings.

2.4. Comparison of the designs

Each building had the same layout for different designs. Shear wall and column element sizes are tabulated in Table 1. Buildings 1C and 2C had stronger shear walls with larger thickness and concrete strength and weaker coupling beams with lower amount of reinforcement. Columns in Building 2C were also thicker, having the same concrete strength. Therefore, code-based design of both buildings had higher stiffness, which results in lower fundamental period. A summary of the periods for different vibration modes is provided in Table 2.

Table 1. Comparison of the member sizes

<table>
<tr><th></th><th>1A</th><th>1C</th><th colspan="2">2A</th><th colspan="2">2C (2B)</th></tr>
<tr><th>Floor</th><th>Wall</th><th>Wall</th><th>Wall</th><th>Column</th><th>Wall</th><th>Column</th></tr>
<tr><td>L42</td><td rowspan="4">21"
(8 ksi)</td><td rowspan="2">21"
(8 ksi)</td><td rowspan="6">18"
(5 ksi)</td><td rowspan="7">36"x36" typ.
(5 ksi)</td><td rowspan="2">16"
(6 ksi)</td><td rowspan="4">36"x36" typ.
(5 ksi)</td></tr>
<tr><td>L31</td></tr>
<tr><td>L30</td><td rowspan="5">24"
(8 ksi)</td><td rowspan="4">18"
(6 ksi)</td></tr>
<tr><td>L26</td></tr>
<tr><td>L25</td><td rowspan="10">24"
(8 ksi)</td><td rowspan="3">42"x42" typ.
(5 ksi)</td></tr>
<tr><td>L21</td></tr>
<tr><td>L20</td><td rowspan="8">24"
(6 ksi)</td><td rowspan="8">24"
(8 ksi)</td></tr>
<tr><td>L14</td><td rowspan="7">32" (E-W)
36" (N-S)
(8 ksi)</td><td rowspan="3">36"x36" int./42"x42" ext.
(6 ksi)</td><td rowspan="3">42"x42"
(6 ksi)</td></tr>
<tr><td>L11</td></tr>
<tr><td>L10</td></tr>
<tr><td>L9</td><td rowspan="2">36"x36" int./46"x46" ext.
(8 ksi)</td><td rowspan="2">46"x46"
(8 ksi)</td></tr>
<tr><td>L0</td></tr>
<tr><td>B1</td><td rowspan="2">36"x36" int./ 46"x46" ext.
(10 ksi)</td><td rowspan="2">46"x46"
(10 ksi)</td></tr>
<tr><td>B4</td></tr>
</table>

Table 2. Comparison of fundamental periods

Period (sec)	Building 1A	Building 1C	Building 2A	Building 2C
Mode 1	5.2	4.6	4.5	4.3
Mode 2	4.0	3.5	4.0	3.9
Mode 3	2.4	2.2	2.5	2.4

3. MODELING

A uniform modeling procedure was established for both prototype models to allow the engineering demand parameters (EDPs) to be compared. 3D nonlinear finite element models were built using Perform3D (CSI, 2006). The seismic mass was lumped at the center of mass at each floor for the floors above the ground level. The seismic mass at the floors on and below the ground levels were assigned as distributed mass. For the floors above the ground, a rigid diaphragm was incorporated by slaving the translational degrees of freedom at each floor level. Flexible diaphragms were modeled at the floors below grade. The foundation of the each building was assumed as rigid. P-Delta effects were taken into account.

3.1. Core wall modeling

Nonlinear vertical fiber elements representing the expected behavior of concrete and steel were used to model the core wall. Expected material strengths of $1.3*f'_c$ (where f'_c is the specified compressive strength) and $1.17*f_y$ (where f_y is the yield strength) were used for concrete and reinforcing steel, respectively. Only the confined concrete was modeled, i.e. the unconfined concrete cover was neglected. The concrete stress-strain relationship was based on the modified Mander model (Mander et. al., 1988). Tension strength of concrete was neglected.
Figure **3**(a) shows the stress-strain relationship of the concrete material. The steel stress-strain relationship was based on the material specifications for A706 steel and shown in
Figure **3**(b). The post-yield stiffness and cyclic degradation of reinforcing steel was modeled using the adjusted vales suggested by Orakcal et al. (2006).

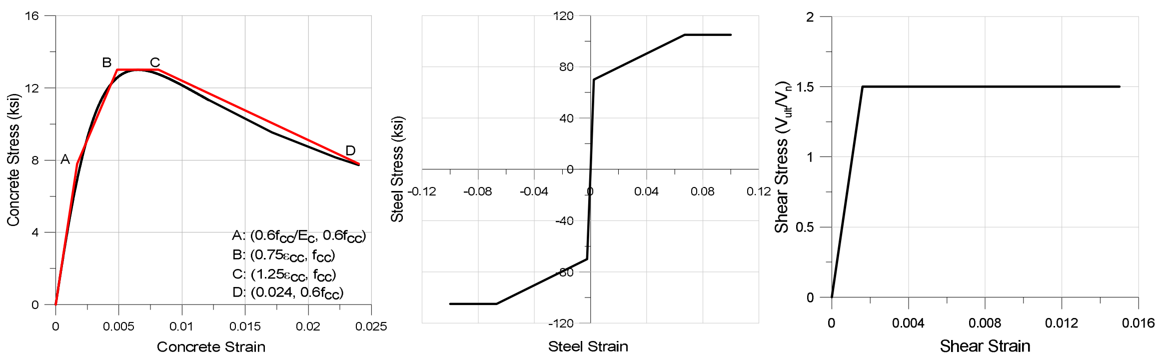

Figure 3. Material stress-strain relationships

The shear behavior was modeled using the inelastic shear property in Perform 3D, where the shear modulus G is calculated using 20% of the expected elastic modulus E_c as recommended by ACI 318 (2008). Inelastic shear material was defined using the elastic-perfectly plastic stress-strain curve provided by Perform 3D. Figure 3(c) shows the stress-strain relationship of shear material, where the ultimate shear strength (V_{ult}) was defined as $1.5V_n$ where V_n is the nominal shear capacity of the shear wall as specified by ACI 318-08.

3.2. Coupling beam modeling

The coupling beams were defined as elastic beam elements with nonlinear displacement shear hinge at the mid-span of the beam. The force-rotation relationship was modeled using a tri-linear backbone curve with the effective flexural stiffness ($EI_{eff} = 0.2EI_g$) and some dimensionless parameters to model to match the experimental test data conducted by Naish et al. (2009).

3.3. Slab and basement wall modeling

Slabs at and below the ground level were modeled as elastic shell elements with stiffness values of $EI_{eff} = 0.25EI_g$ (flexural) and $GA = 0.5GA_g$ (shear). All slabs had a specified concrete strength of $f'_c = 5$

ksi. Shear modulus (G) was calculated using a Poisson's ratio, $\nu = 0.2$. Basement walls were modeled as elastic finite elements with stiffness values of $EI_{eff} = 0.8EI_g$ (flexural) and $GA = 0.8GA_g$ (shear).

3.4. Moment frame beam and column modeling

The moment frame beams in Building 2 were defined as elastic beam elements with nonlinear rotation hinges and rigid end zones at each end. The elastic portion of the beam was modeled with the cross-section properties and the stiffness modification factors such that $EI_{eff} = 0.35EI_g$ (flexural), $GA = 1.0GA_g$ (shear). The non-linear moment-rotation hinges, which were defined based on the tests performed by Popov et al. (1972). Further details can be obtained from the design reports (Moehle et al., 2011).

The moment frame columns were implemented in Building 2, where they were defined as elastic column elements with plastic hinges and rigid end zones at each end. The elastic portion of the column was modeled with the cross-section dimensions and the stiffness modification factors of $EI_{eff} = 0.7EI_g$ (flexural), $GA = 1.0GA_g$ (shear). To define a column plastic hinge, a moment-axial capacity interaction curve was calculated using the expected material properties of column. The backbone curve was elastic-perfectly plastic, neglecting strength loss and cyclic degradation.

3.5. Damping and masses

2.5% Rayleigh damping was used at a period of 1 second and at a period of 5 seconds. The range of damping ratio was selected such that an average of 2.5 % critical damping was covered for the period between $0.2T_1$ to $1.5T_1$ of all models, where T_1 is the first modal period of the buildings.

4. ANALYSIS RESULTS

The prototype models were analyzed with a nonlinear response history analysis for five hazard levels, each consisting of 15 pairs of ground motions (i.e., two horizontal components). The hazard levels include return periods of 25, 43, 475, 2475, and 4975 years which are referred as SLE25, SLE43, DBE, MCE and OVE, respectively. Detailed ground motion selection criteria can be identified from the PEER Report (Moehle et al., 2011). Response history analysis was applied after a gravity load of P=1.0D+0.25L.

4.1. Comparison of the structural responses

Maximum inter-story drifts and absolute floor accelerations were recorded from the response history analysis. Mean value and standard deviation of the peak responses are shown in **Figure 4**. The results show that the code-designed buildings (1A and 2A) have slightly larger drift demands compared to the performance-based designed buildings (1C and 2C). Building 1 (core wall only) shows larger inter-story drift demands that in Building 2. Peak drifts at the serviceability levels approached 0.5%, whereas the drifts at the OVE level reached 2.5% and 4% for Building 2, and Building 1, respectively. The OVE level showed the most dispersion compared to the other hazard levels. In all cases, the peak drift values occurred around the 35^{th} floor and never exceeded the acceptable limit which is 3% for MCE, 2% for DBE and 0.5% for SLE levels. For comparison purposes, acceptable limit of 3% is used for OVE as well, even though a limit is not specified for this level in the code. Peak floor accelerations are presented in **Figure 5**. The results indicate that the fundamental modes were not excited under SLE (very low response associated with the tower). For the OVE and MCE events, accelerations were limited by yielding, with maximum values of approximately 0.5g over a majority of the tower. Building 1C had the smallest acceleration demands at all hazard levels.

Wall shears, and coupling beam rotations were also investigated; mean value and standard deviation of the results are shown in **Figure 6** and **Figure 7**. Shear force distributions were very similar in both buildings. A linear profile was observed for each hazard level, which indicates first mode dominant response in all cases. For all hazard levels, the peak wall shear stresses were much less than the ACI 318-08 limit of $\left(8\sqrt{f_c^{'}}\right)$ except for the OVE level where the shear stress reaches the limit around ground level. Coupling beam rotations (**Figure 7**) were much higher in the performance-based design approaches (1C and 2C) which required stronger walls to be used. In the OVE level, peak rotations reached 5% and 6% for Building 2C and 1C, respectively; whereas serviceability level rotations were much smaller (<1%). Given the small rotations, according to the fragility curves developed by Naish (2010), no repair was likely to be needed.

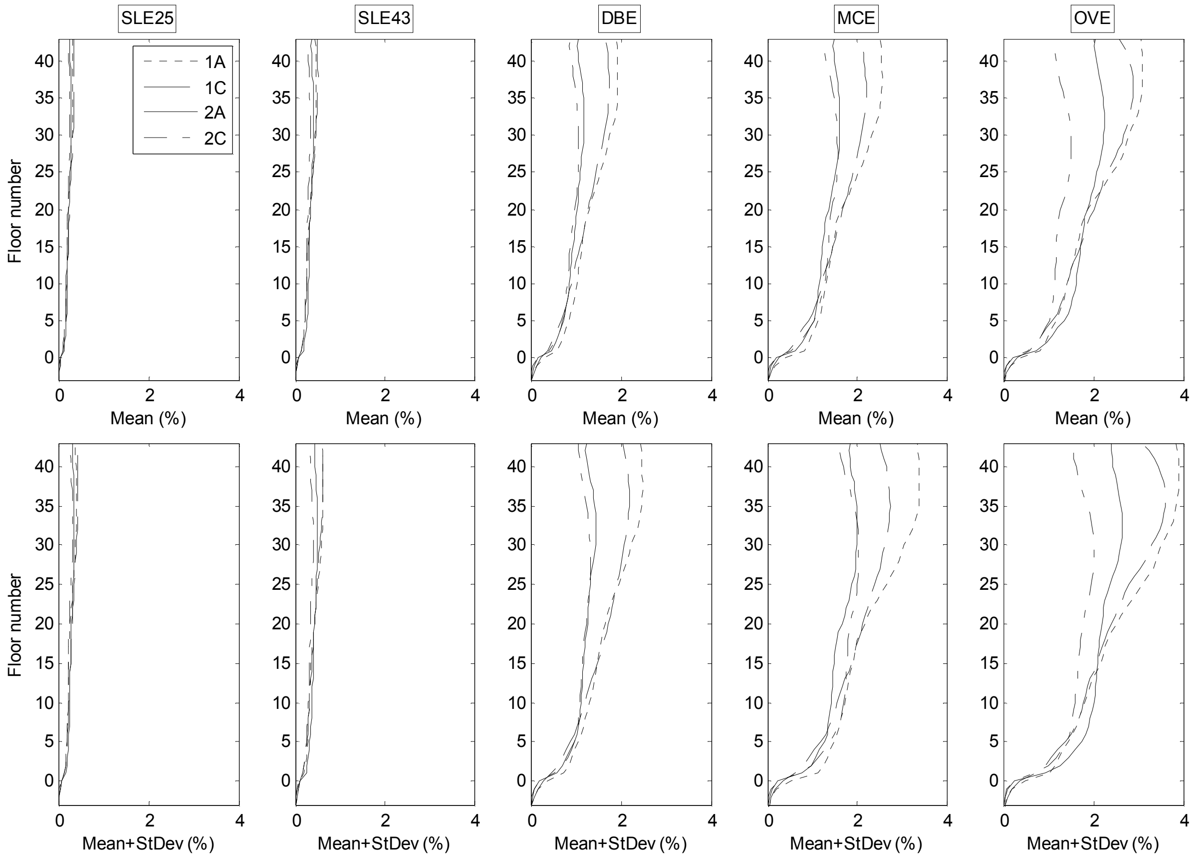

Figure 4. Comparison of interstory drift ratios

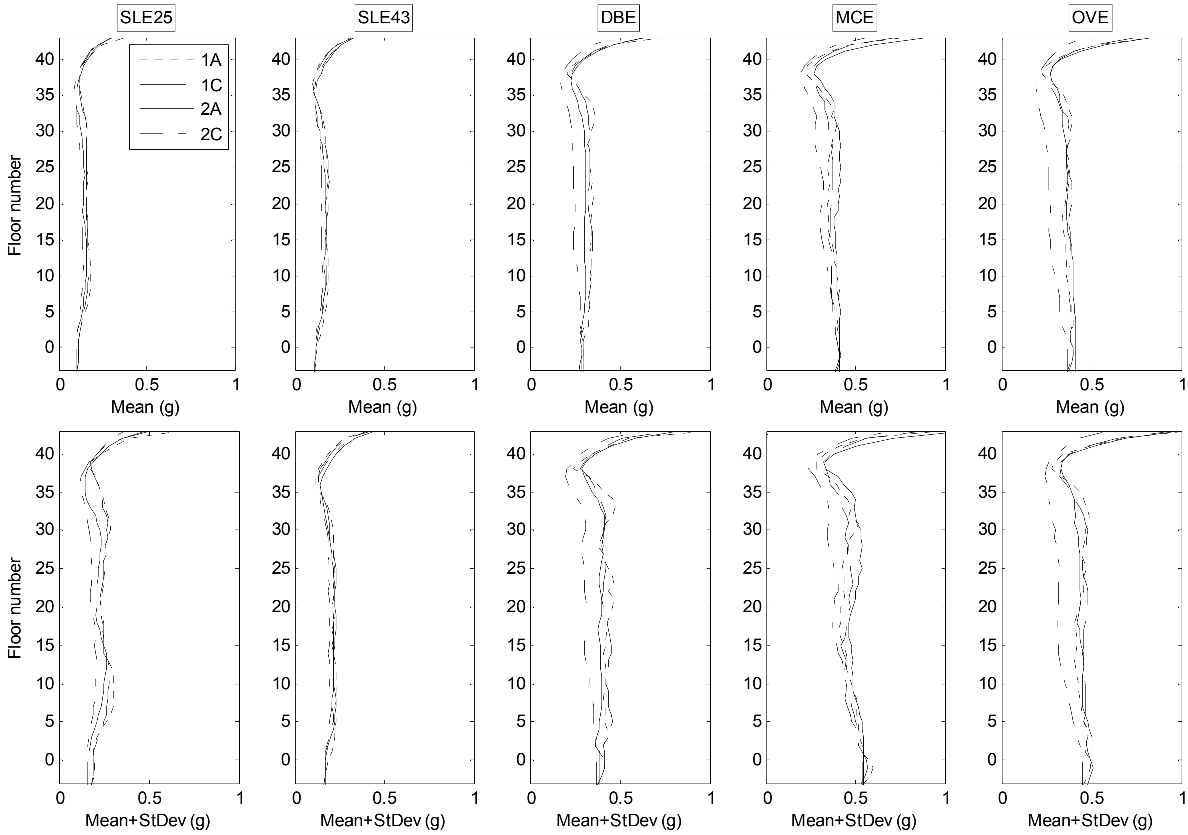

Figure 5. Comparison of floor accelerations

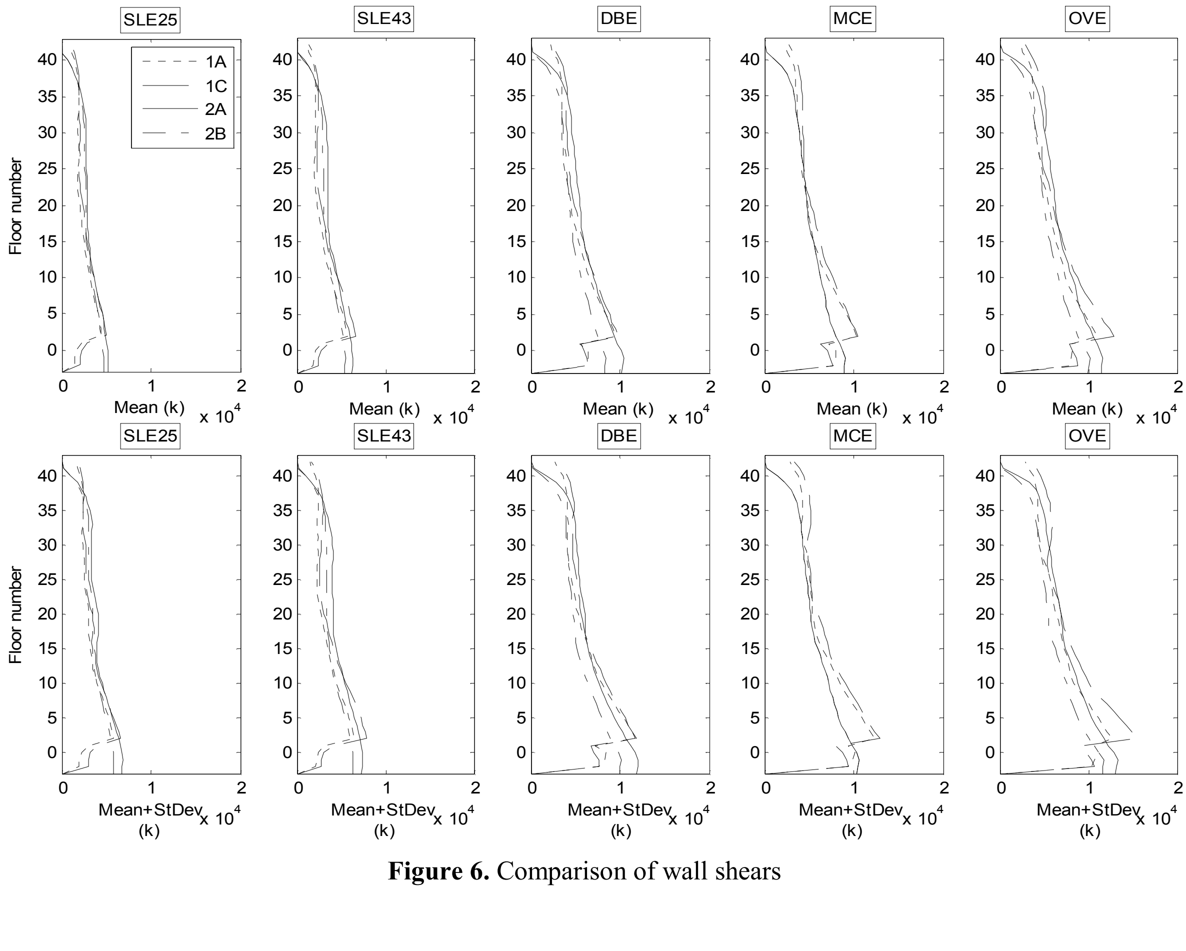

Figure 6. Comparison of wall shears

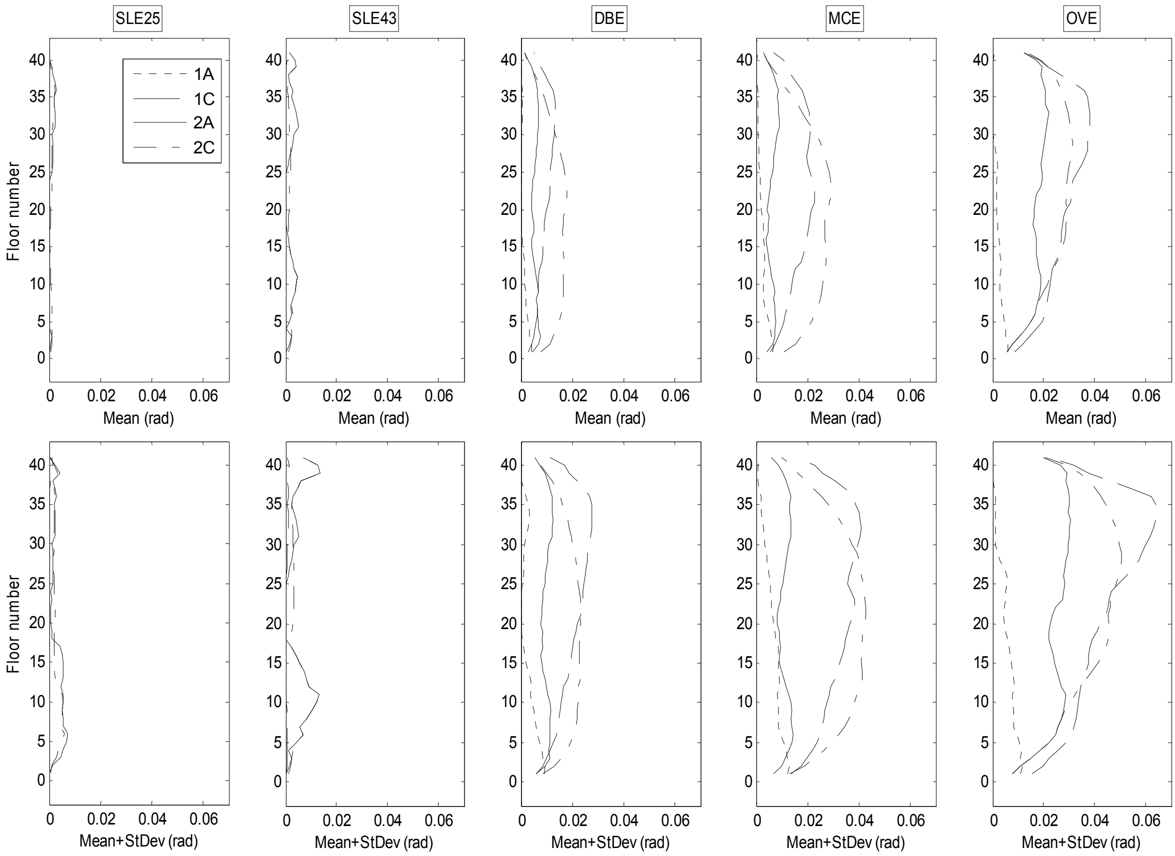

Figure 7. Comparison of coupling beam rotations

4.2. Comparison of initial cost and loss estimation of the buildings

4.2.1. Initial construction cost

The initial costs of the building were estimated by professional cost estimator (Langdon 2010) based on the material usage and additional design fees. Table 3 shows a summary of the initial construction cost per square feet. The results shows the initial construction cost were higher in Building 2 compared to Building 1. As the design approach shift from Design A to C, the initial construction cost was very similar for Building 1, but about 16% higher for Building 2.

Table 3. Comparison of the construction costs

	Core Wall (Building 1)	Dual System (Building 2)
Code-based (Design A)	\$326/sq ft	\$348/sq ft
Performance-based-plus (Design C)	\$332/sq ft	\$411/sq ft

4.2.2. Annual repair cost

Loss estimation analysis revealed that the repair cost for Building 1A was mostly associated with the interior partitions and contents at the SLE25 and SLE42 hazard levels. At the DBE level, some shear wall webs also contributed to the repair cost, in addition to the partitions and contents. At the MCE and OVE hazard levels, additional shear wall web, slab-column connections, and curtain walls were damaged and contributed to the total repair cost. Damage in Building 1C was similar to that in Buildings 1A, except that Building 1C sustained more damage to coupling beams.

Annual repair cost for Building 2A was attributed to the damage to the shear wall webs and interior partitions at all hazard levels including SLE25. At the DBE level, all interior partitions and contents, and more shear wall webs began to contribute to the total repair costs. As shaking intensity increased to the OVE level, additional damage occurred to the moment resisting frames. Similar damage trends were observed for Building 2B all hazard levels considered.

Table 4 summarizes the median annual repair costs for the four different designs considered. For the code design approach, similar annual repair costs were noted for the two buildings; however, repair costs for the dual systems were 25% less than the core wall building using the performance-based design approaches.

Table 4. Comparison of the median annual repair costs

	Core Wall (Building 1)	Dual System (Building 2)
Code-based (Design A)	\$326,000	\$323,000
Performance-based-plus (Design C)	\$282,000	\$269,000

5. SUMMARY AND CONCLUSIONS

Reinforced concrete shear walls are prevalent seismic force resisting systems used in the West Coast of the United States. To analyze the seismic performance of such systems, two prototype buildings located in Downtown Los Angeles were designed. Building 1 represented the reinforced concrete core wall building, whereas Building 2 was a similar reinforced concrete shear wall building with additional perimeter moment resisting frame. The buildings were designed by two renowned structural engineering firms in the United States. Seismic responses of the two buildings were investigated at five seismic hazard levels. For each design, nonlinear dynamic analyses were carried out for 15 ground motions at each of the five seismic hazard levels. The result shows that coupling beam rotation demands were much higher in the performance-based design approaches, which required stronger walls to be used. Based on the tension strain demands, more yielding was expected over the wall height in the code-based designed buildings. Shear wall demands were well below the median shear strength defined as 1.5 times the ACI 318-08 nominal shear strength. Cost analysis results, performed by a professional estimating firm, indicated that the initial construction cost was higher in Building 2 as compared to Building 1. As the design approach shift from Design A to C, the initial construction cost was very similar for Building 1 but about 16% increase for Building 2. The loss simulation analysis revealed that the losses for the performance-based designs were lower than those for the code-based designs. For the code design approach, similar annual repair costs were noted for the two buildings; however, repair costs for the dual systems were 25% less than the core wall building using the performance-based design approaches.

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