Application of the US Resiliency Council Seismic Rating Procedure to Two Tall Buildings Designed by Alternative Means

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Abstract

The objective of this study is to perform a comparative assessment of the earthquake resilience of two design variants of a 42-story reinforced concrete building using the United States Resiliency Council (USRC) Seismic Rating procedure. The buildings were developed as part of the Pacific Earthquake Engineering Research (PEER) Institute Tall Building Initiative (TBI) project. One variant was designed using prescriptive code provisions and the second using the Los Angeles Tall Building Structural Design Council (LATBSDC, 2008) Guidelines. The Seismic Performance Prediction Program (SP3) is used to perform the building rating analysis based on the FEMA P-58 methodology (FEMA, 2012). Ratings are established for the three categories of performance including safety, repair cost and functional recovery time.

The results showed that both buildings achieve ratings of four, five and two stars for the safety, damage and recovery dimensions respectively. At the design basis earthquake (*DBE)* level, the mean repair cost normalized by the building replacement value is 1.8% for the code-based building and .9% for the *LATBSDC* building. However, at the *MCE* level, the repair costs for the *LATBSDC* building (13%) is about 20% higher than that of the code-based building (11%). This result is explained by the fact that the residual drift demands are significantly higher in *LATBSDC* building and dominates the losses at the *MCE* level. For both buildings, the *REDi* recovery time is dominated by impeding factors which account for more than 80% of the functional recovery time.

Introduction

For the most part, seismic design codes and guidelines are established with the intent of ensuring life safety in the event of large magnitude earthquakes. However, events like the 2010-2011 Canterbury earthquake sequence have highlighted the critical role of building performance in minimizing the impact on community functionality. While the February 22, 2011 event resulted in a low (relatively speaking) number of fatalities, the central business district was severely disrupted. In the hours immediately following the earthquake, local authorities cordoned off 114 square blocks of the downtown area, eventually reducing the zone to 75 blocks ten days later. This was largely due to the risk of aftershock collapse and falling debris from several mid- and high-rise buildings, which were extensively damaged and subsequently slated for demolition. Moreover, local authorities mandated the closure of surrounding streets during the demolition of these buildings (EERI, 2011).

As demonstrated in the Canterbury earthquake sequence, the physical size and concentration of people and services in tall buildings is such that their seismic performance has strong implications to the resilience of the urban environments that they occupy. As such, an explicit quantification of their seismic performance is crucial to understanding their role in ensuring continued functionality of large city centers following a hazard event.

The performance-based earthquake engineering (*PBEE)* framework provides an alternative to the prescriptive design approach of the building code. Following the development of the *PBEE* methodology, several efforts have been directed towards advancing the implementation of performance-based design of tall buildings in structural engineering practice. One such initiative was the Pacific Earthquake Engineering Research (*PEER)* Center Tall Buildings Initiative (*TBI).* As part of the *TBI* project, three different tall building types (concrete core only, concrete core with reinforced concrete moment frame and steel buckling-restrained braced frame system) were designed using three different approaches including building code prescriptive procedures, the *LATBSCD* (2008) and the *TBI* (2010) draft guidelines. The study included probabilistic seismic hazard analysis and ground motion selection for the building site which is located in Los Angeles, California, structural modeling and response simulation and loss assessment studies that estimated repair costs for future earthquakes.

Another initiative that is focused on meeting the challenge of developing earthquake resilient communities is the United States Resiliency Council (*USRC*) Building Rating System for Earthquake Hazards. By providing a means of quantifying risk, the *USRC* rating system is intended to increase the economic value of buildings designed to higher seismic standards using performance- or resilience-based methods. The current study is focused on applying the *USRC* Seismic Rating to two of the design variants of the 42-story reinforced concrete building developed as part of the *TBI* project including the prescriptive code (code-based) and *LATBSCD* (performance-based) procedures. The Seismic Performance Prediction Program (*SP3)* is used to conduct the building rating analysis based on the FEMA P-58 methodology (FEMA, 2012). Ratings are established for the three categories of performance including safety, repair cost and functional recovery time.

Overview of *USRC* Seismic Rating Criteria

The *USRC* rating serves as a tool to communicate the results of an engineering-based building evaluation to the relevant stakeholders of that building. Star ratings are provided for three separate dimensions: safety, damage and recovery.

The safety rating is described in terms of the potential for earthquake-related injuries, loss of life and the ability to evacuate the building following a seismic event. Five stars, which is the highest rating, is assigned in cases where the level of damage is unlikely to cause injuries or prevent timely evacuation. The lowest rating, which is one star, is assigned in cases where there is a high likelihood of collapse and loss of life within or around the building. When serious injuries are unlikely, loss of life is unlikely or loss of life possible in isolated locations, four, three and two stars are assigned respectively.

The damage rating is assigned based on the estimated cost of repairing earthquake-related damage. This cost is defined relative to the replacement cost of the building and includes structural, architectural mechanical, electrical and plumbing components. Content damage is not considered in estimating the repair cost. Five (minimal damage), four (moderate damage), three (significant damage) and two (substantial damage) star ratings are given in cases where the repair cost is less than 5%, 10%, 20% and 40% of the replacement cost respectively. One star (severe damage) is assigned in cases where the repair cost exceeds 40% of the replacement cost.

The recovery rating is assessed based on the time it takes the owner to regain use of the building for its primary intended function. It includes the time needed to perform repairs, mitigate safety hazards and impediments to re-entry and use. The time to address disruptive conditions that originate away from the building site, is not considered. Five, four, three two and one-star rating is assigned in cases where the delay in restoring basic functionality is days, days to weeks, weeks to months, months to a year and more than one year, respectively.

The rating systems relies on existing tools, techniques and professional norms for performing the engineering evaluation. Currently, the ASCE 41-13 and the FEMA-P58 performance assessment methodologies can be used to establish building ratings. The rating is developed assuming the building is subjected to ground shaking at its site corresponding to a hazard level of 10% probability of exceedance in 50 years.

Overview of FEMA-P58 Based Seismic Rating Criteria

Overview of FEMA P58 Methodology

The FEMA-P58 guidelines (Volumes 1, 2 and 3) form the basis of the second-generation of *PBEE*. Key features of the methodology include (1) robust techniques for accounting for and communicating uncertainty to stakeholders, (2) the use of quantitative measures of performance that are relevant to new and existing buildings, (3) explicit assessment of physical damage to structural and non-structural components and (4) an assessment of performance based on global parameters. Performance measures considered in FEMA P58 include the probable number of casualties, the expected cost of repairing or replacing a damaged building, the time needed to restore the building to its pre-earthquake condition and the likelihood of unsafe placarding. The assessment of the probable number of casualties is enabled by an explicit and quantitative evaluation of collapse safety using the methodology outlined in the FEMA P695 (FEMA, 2009) guidelines. Performance functions are used to link the ground shaking intensity to exceeding some level loss (causalities, economic etc.). Three alternative types of assessments have been enabled, which vary based on the treatment of seismic hazard. Intensity-Based assessments are used to evaluate the probable performance measure conditioned on the occurrence of a specific shaking intensity. Scenario-based assessments calculate the probable performance of a building subjected to an earthquake scenario defined by a specific magnitude event occurring at a specific location relative to the site. In time-based assessments, mean seismic hazard curves are used to defined ground shaking hazard, which is used to compute the mean annual frequency of a particular consequence (e.g. collapse, losses exceeding a particular level etc.).

Overview of FEMA P58 Criteria for USRC Seismic Rating

The description for each *USRC* rating level within the three dimensions is linked to a specific FEMA P58 criterion. Recall that the *USRC* safety rating criteria is based on the likelihood injuries and blocking of evacuation routes. The associated FEMA P58 rating criteria is based on the computed probability of fatal and non-fatal injuries considering both collapse and non-collapse falling hazards and egress routes being intact for the 475 event. The probability of fatal injuries associated with associated with five, four and three-star ratings is 0.00003, 0.0001 and 0.0004 respectively. The five and four-star ratings have additional criteria of all egress routes being intact and the probability of a non-fatal injury being less than 0.02 respectively. For the two-star rating, the probability of a non-fatal injury must be less than 0.004. A building is assigned a one-star rating when it does not meet the two-star criterion.

For the damage rating, the repair-cost thresholds for the FEMA P58 criteria, which are described as a percentage of the building replacement cost, are the same as the *USRC* rating criteria. The FEMA P58 criteria for the recovery ratings are based on the median recovery time. For the five, four, three and two-star ratings, the corresponding median recovery times after a 475-year event are 5 days, 4 weeks, 6 months and one year respectively. A one-star rating is assigned if the median recovery time is more than one year.

An estimate of the collapse capacity of the building is needed to compute the repair cost and time and the probability of fatal and non-fatal collapse-injuries, all of which are included in the FEMA P58 rating criteria. The *USRC* rating requires the use of the FEMA 154 approach to estimating the building's collapse capacity in lieu of other methods such as incremental dynamic analyses. The FEMA 154 approach begins by using the checklists to compute the resultant "score" $(S\,value)$ for the building. Given the $S\,value$, the probability of collapse occurring and affecting an occupant at a specific location within the building conditioned the risk-targeted maximum considered earthquake (MCE_R) ground motion is computed.

$$P\left[\text{"}Collapse\text{"} \mid MCE_R\right] = 10^{-S} \tag{1}$$

where $P\left[\text{"}Collapse\text{"} \mid MCE_R\right]$ is the probability of total or partial collapse times the ratio of the area of the building affected by collapse. The collapse area ratio, which is provided in Table 1 of Appendix E of the *USRC* implementation manual, is needed to convert the "collapse" probability from equation 1 to the collapse probability used in the FEMA P58 methodology. The dispersion or log-standard deviation of the collapse capacity is obtained from Table 2 of Appendix E of the *USRC* implementation manual.

The FEMA 154 checklist does not provide score values Risk Category III and IV and base isolated structures. The score values for buildings falling in these three categories are provided on page 6 of Appendix E of the *USRC* implementation manual. In cases where a building is partially retrofitted, checklist deficiencies addressed by "comprehensive building retrofit" can be ignored. A full basic score increase can be used if the retrofit meets the performance objectives at or above 75% of the new code. In cases where it can be demonstrated that specific checklist items do not affect the building performance or are explicitly addressed during the building design, these items can be removed from the checklist. Engineering judgement can be used to modify the collapse fragility curves based on building properties not considered in the FEMA 154 checklist.

The MCE_R hazard level is converted to the 10% in 50-year hazard level using the *USRC* rating conversion factor of 1.5. However, it is noted that this conversion factor does not include near-fault and transition zone regions that are deterministically capped. Conversion factors for these conditions are likely to be less than 1.5 and buildings located at building sites with these characteristics will be required to meet a higher FEMA 154 score in our to achieve a particular safety rating.

The fatality-rate thresholds used in the safety rating are based on the fatality-rates computed at the 10% in 50-year hazard level using the $S\,values$ (FEMA 154), collapse area ratio and the default collapse capacity dispersion and fatality rate for each building type. The computed fatality-rates are summarized in Table 4 of Appendix E of the *USRC* implementation manual. The thresholds are based on the average values for all building types. The allowable fatality-rate for safety ratings corresponding to three stars and higher is increased by a factor of two account for falling-hazard fatalities.

The injury-rate threshold for the four-star safety rating is based on a benchmark study by Cook et al. (2015). This threshold was set such that the building used in the study required additional anchorage above code-requirements to meet the four-star threshold of 0.02.

REDi Methodology for Recovery

The *REDi* methodology defines three sequential recovery levels: re-occupancy, functional recovery and full recovery. To facilitate computing repair times, building component-level damage is divided into repair classes. Repair class two includes damage to non-structural components that do not pose a threat to life safety. Repair class three includes heavy structural or non-structural damage that are a threat to life safety.

The *USRC* recovery time ratings are determined using functional repair time and accounts for the lead time prior to the start of construction. Delays due to disruption in off-site electric power are not considered. Disaggregation of the recovery time into repair time and the time associated with impeding factors is required.

Description of Case Study Building and Structural Analyses

Description of Case Study Buildings

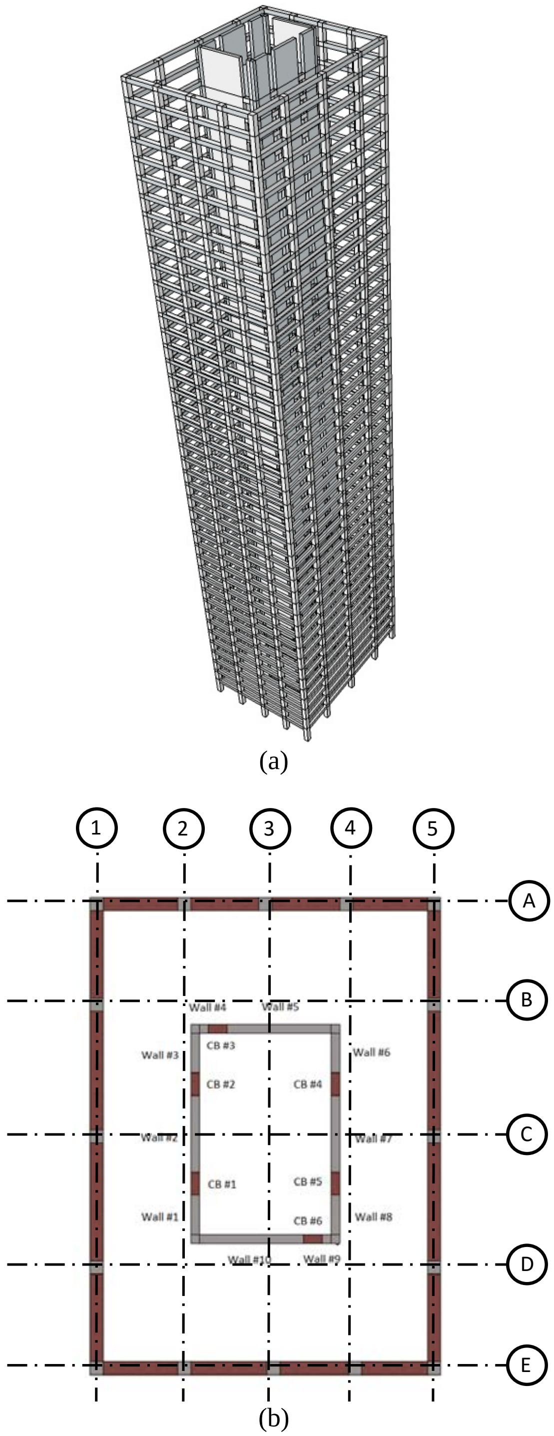
The buildings used in this study were developed as part of the *PEER TBI* project. Two design variants of the core wall and special moment frame dual system building are considered in this study. An isometric view and plan layout of the dual frame system is shown in Figure 1. The building is 42 stories tall above the ground floor, has four basement levels and a penthouse at the roof level. The core walls are L-shaped and are connected with coupling beams.

The first variant (identified as Building2A in *TBI*, 20010) was designed using the IBC 2006 building code provisions, which incorporated the requirements of ASCE 7-05 and ACI 318-08. The seismic demands were obtained using modal response spectrum analysis using site-specific spectra with 5% damping. The period of the first, second and third modes were 4.46 seconds 4.03 seconds and 2.48 seconds respectively. The core walls are 24 inches thick with concrete strength, $f_c^{'} = 6{,}000\ psi$ from the foundation to the twentieth floor and 18 inches thick with concrete strength $f_c^{'} = 5{,}000\ psi$ above the twentieth floor. The coupling beams are 30 inches deep at all floor levels. All special moment frame beams are 30 inches wide and 26 inches deep with concrete strength $f_c^{'} = 5{,}000\ psi$. The special moment frame columns along grid Lines A and E, are 36 inches square in cross section with concrete strength, $f_c^{'}$, varying from $5{,}000\ psi$ to $10{,}000\ psi$ along the height. The reinforcement details for the framing members are provided in Appendix B and *TBI* (2010). The floors are constructed using reinforced concrete to be 10 inches thick at the basement and roof levels, 12 inches thick at the ground level and 8 inches thick in the tower. Post-tensioning is used in the tower floor slabs.

The second variant of the core wall, special moment frame building was designed using the 2008 *LATBSDC*. The structure was evaluated for the serviceability and collapse prevention performance levels. The seismic demands for the serviceability level assessment were obtained from a site specific response spectrum analysis corresponding to a 25-year return period event with 2.5% viscous damping. Up to 20% for the deformation-controlled components were permitted to reach 150% of their capacity at the serviceability level. The minimum base shear requirement in 2008 *LATBSDC* was waived and the ACI 318-08 strength reduction factors were applied in evaluating deformation-controlled actions at the service level. A three-dimensional model of the structure was constructed in Perform 3D and analyzed using seven pairs of spectrally matched ground motions. Nonlinear response history analyses were used to evaluate the collapse prevention performance level at the maximum considered earthquake (*MCE*) hazard level (2475-year return period). The period of the first, second and third modes were 4.28 seconds 3.88 seconds and 2.439 seconds respectively. The following design modifications were applied to code-based design variant (2A) in order to satisfy the requirements of the *LATBSDC* serviceability and collapse prevention checks:

- $8{,}000\ psi$ concrete was used for the 24-inch-thick core walls used from the foundation to the 20th floor.
- $5{,}000\ psi$ concrete was used for the 18-inch-thick core walls used from the twentieth to the thirtieth floor.
- The core walls above the thirtieth floor are 16 inches thick with $6{,}000\ psi$ concrete.
- The coupling beam sizes were unchanged but the concrete strengths were changed to match the connecting core wall and the diagonal reinforcement was reduced.
- The corner columns in the moment frames along grid-lines A and E were increased to 46 inches square from the foundation to the tenth floor and 42 inches square from the tenth to the thirtieth floor.
- The amount of reinforcement in the frame beams and corner columns was reduced.
- The reinforcement in the columns along frame lines 2 and 5 were increased.
- The amount of boundary reinforcement in the core walls is reduced.

These modifications represent the difference in the structural design of the code-based and *LATBSDC* design variants. Further details on the design of the two building variants can be found in *TBI* (2010).



(a)

(b)

Figure 1 (a) Isometric view and (b) plan layout of dual moment frame system

Structural Modeling

The *USRC* Seismic Rating assessment is performed with the *SP3* software tool using the user-defined engineering demand parameters (*EDPs*). Three-dimensional structural models of the lateral force resisting system (gravity system not included) of the tower (basement levels not included) are constructed in OpenSees (Mazzoni et al. 2007). A rigid diaphragm is incorporated at all suspended floor levels by constraining the horizontal translational degrees of freedom. The seismic mass is lumped at the center of mass at each floor. Expected gravity loads (D + 0.25L) are used in the model. A leaning column is used to account for P-Delta effects resulting from the expected loads on the gravity system. The leaning column is axially rigid, has no lateral stiffness and the horizontal translational degrees of freedom of the end nodes are constrained to the floor nodes. The core walls and moment frame columns are fixed at the base.

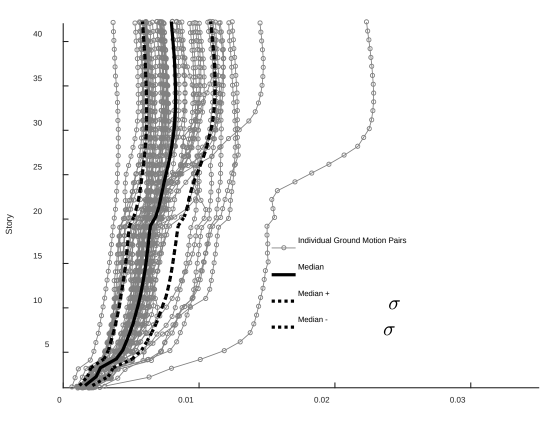
The moment frame elements and coupling beams are defined using elastic beam-column elements with flexural plastic hinges at the ends. The nonlinear behavior of the flexural hinges in the frame beams and columns is based on the Ibarra et al. (2005) peak oriented hysteretic model and the predictive equations developed by Panagiotakos et al. (2001) and Haselton et al. (2008) are used to obtain the backbone parameters. For the coupling beams with diagonal reinforcement, the flexural hinge parameters are based on test results by Naish et al. (2009). A multi-layer shell element (Lu et al., 2015) is used to capture the non-linear behavior of core walls. The cover concrete, confined concrete and vertical and horizontal web reinforcement are modeled using equivalent orthogonal shell layers. The constitutive relation of the concrete material is modeled based on (Loland, 1980) and (Mazars, 1986), and Giuffre-Menegotto-Pinto (Filippou, 1983) model with isotropic strain hardening is used for steel material. The confinement effects, including increase in strength and ductility of the core concrete, are incorporated using the relations suggested by Mander et al. (1988).

Structural Analysis Results and Discussion

Nonlinear response history analyses are performed on the three-dimensional structural model using bi-directional loading. The Conditional mean spectra are computed for the building site (118.25º W, 34.05º N) using the probabilistic seismic hazard analysis deaggregation and the ground motion prediction equations developed by Campbell and Bozorgnia (2014). Three sets of ground motions are selected using the approach suggested by Baker (2010): 25 pairs for the 10% in 50-year and 2% in 50-year hazard levels respectively, which are used for intensity-based analyses; 48 pairs for 1% in 50-year hazard level, which has a mean ε of 1.73 for large magnitude-long distance events and are used for incremental dynamic analysis to collapse.

The *EDPs* needed for the SP3 analyses include the peak transient and residual drifts, peak floor accelerations and chord rotations for the shear wall and coupling beams chord rotations. Three sets of analyses are performed: two intensity-based analyses with the ground motions scaled to match the 1st mode period spectral acceleration associated with the 10% in 50-year and 2% in 50-year hazard levels and incremental dynamic analyses (IDA) to collapse. The results from the IDAs are compared with the collapse results obtained using the FEMA 154 approach, which is for the *USRC* rating.

For the purposes of reporting the results of the nonlinear response history analyses and FEMA P-58 assessment, the transverse direction will be referred to as the X-Direction and the longitudinal, the Z-Direction. Figure 2 shows the maximum story drift profile in the X-Direction of Building 2A for the individual ground motion pairs scaled to the design basis earthquake (DBE) (10% in 50-year) and maximum considered earthquake (MCE) (2% in 50-year) spectral acceleration levels. Two analyses are conducted for each ground motion pair by switching the orthogonal direction of each of the motions. The median maximum story drift profile for the two building cases subjected to the *DBE* and *MCE* level ground motions are shown in Figure 3. For both buildings, the drift demands are generally higher in the X-Direction. For example, at the *MCE* level, the median peak drift in the X-Direction of Building 2A is 1.6% (occurring at the 33rd story) compared to 1.4% (occurring at the 32nd story) in the Z-Direction. The drift demands in Building 2A are generally higher than Building 2B, particularly at the upper stories. At the *MCE* hazard level, the median peak drift in the X-Direction is 1.4% in Building 2B compared to 1.6% in Building 2A. In the Z-Direction, the *MCE* level median peak drift demand is 1.4% and 1.1% in Buildings 2A and 2B respectively. The story drift demands are used to assess damage to several of the deformation-sensitive structural and non-structural components including the gravity and frame beams and columns, interior partitions and exterior cladding.



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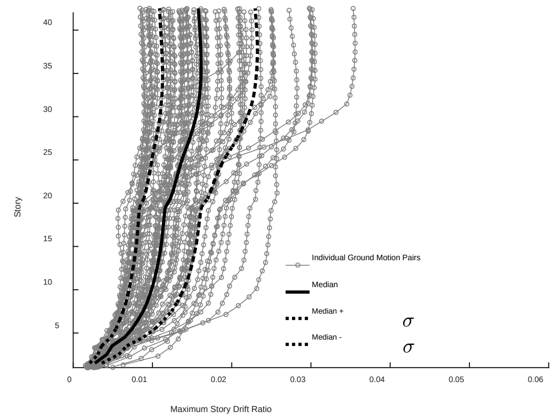
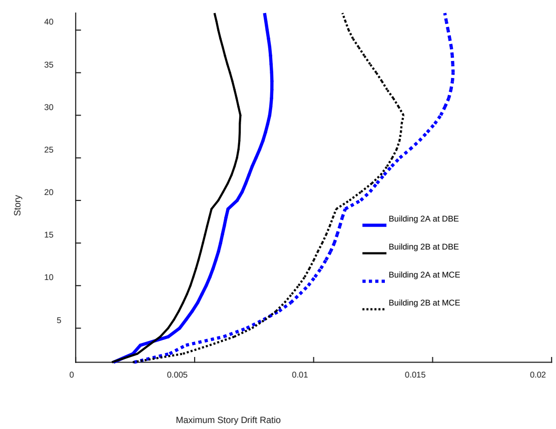


(b)

Figure 2 Maximum story drift profile for Building 2A in the X-Direction for individual ground motion pairs scaled to the (a) DBE and (b) MCE hazard levels



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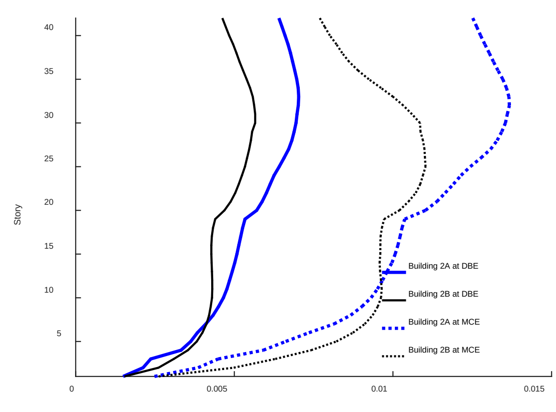
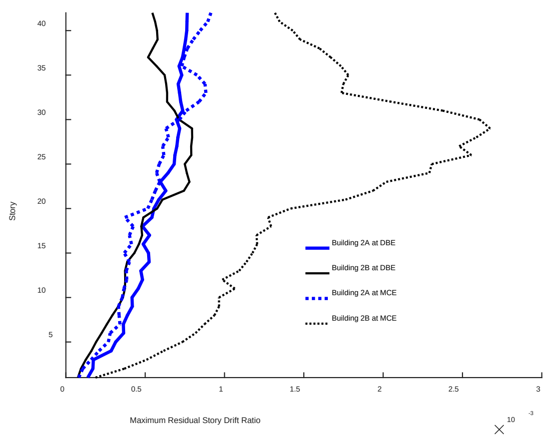


(b)

Figure 3 Median of maximum story drift profile for Buildings 2A and 2B in the (a) X- and (b) Z-Directions for ground motion pairs scaled to the DBE and MCE hazard level

The maximum residual drift demands shown in Figure 4 are relevant to considering the impact of demolition on repair costs and recovery times. The *USRC* Rating procedure requires that the effect of residual drifts be considered in cases where any of the star ratings is greater than 3. Figure 4 shows that residual drift demands are generally higher in Building 2B. At the *MCE* hazard level, the median peak residual drift in the X-Direction is 0.27% in Building 2B and 0.09% in Building 2A. This is likely the result of the reduced core wall reinforcement in Building 2B.



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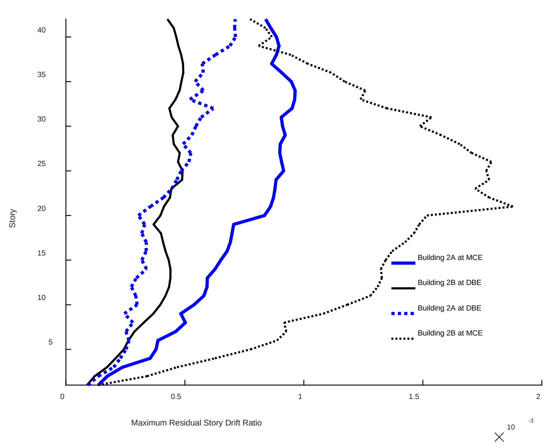
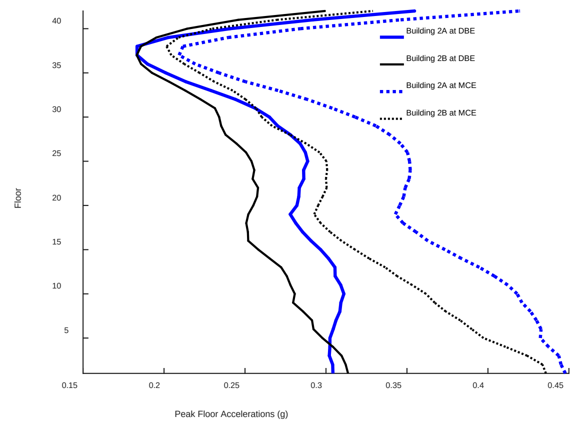


(b)

Figure 4 Median of maximum residual story drift profile for Buildings 2A and 2B in the (a) X- and (b) Z-Directions for ground motion pairs scaled to the DBE and MCE hazard level

Peak floor accelerations are used to simulate damage to acceleration sensitive non-structural components and contents such as ceiling tiles and plumbing lines. Figure 5 shows the median profile of peak floor accelerations for the two building cases. For both buildings, the magnitude and profile of the peak floor accelerations are almost identical in the two orthogonal directions. The demands are generally higher in building 2A.



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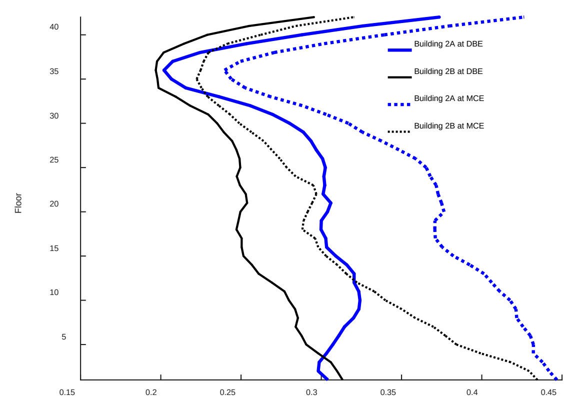
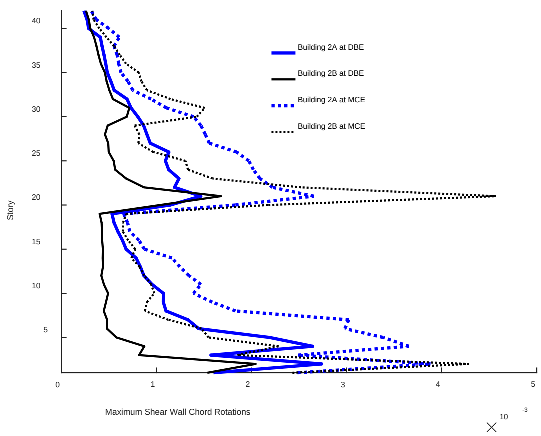


(b)

Figure 5 Median of maximum peak floor acceleration for Buildings 2A and 2B in the (a) X- and (b) Z-Directions for ground motion pairs scaled to the DBE and MCE hazard level

Chord rotations are used to assess damage in the shear wall. The median maximum chord rotation profiles for core walls 5 (X-Direction) and 7 (Z-Direction), which are identified in Figure 1b, are presented in Figure 6. The median chord rotation profiles are generally comparable for the two buildings. For example, in the X-Direction, the median maximum chord rotation at the *MCE* hazard level is 0.0039 for Building 2A and 0.0045 for Building 2B.



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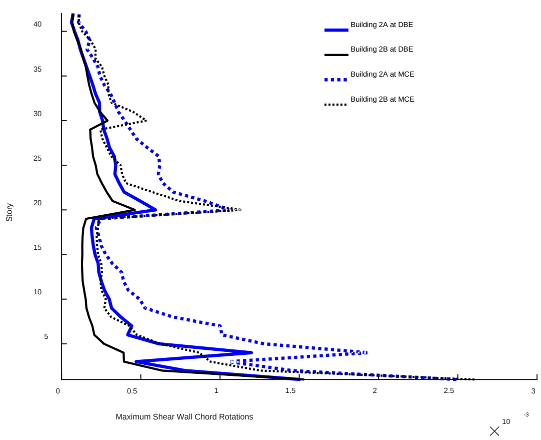
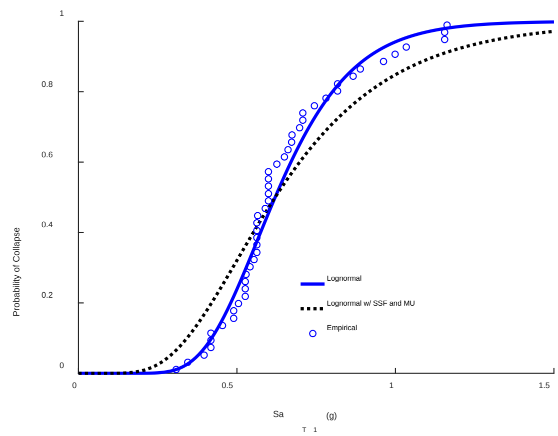


(b)

Figure 6 Median of maximum chord rotation profile in shear walls (a) 5 (X-Direction) and (b) 7 (Z-Direction)

The collapse safety of the two building cases is assessed using incremental dynamic analyses, where each ground motion pair is scaled until the collapse point is reached. The 48 pairs of ground motions are scaled such that their geometric mean match the target intensity. As noted earlier, two analysis cases are used for each record pair by switching the orthogonal direction of the ground motions. Figure 7 shows the collapse fragility curves obtained from Incremental Dynamic Analysis including the effect of spectral shape (*SSF*) and modeling uncertainty (*MU*). The median collapse capacity for Buildings 2A and 2B are 0.6 g and 0.5 g respectively and the record-to-record variation is approximately 0.45 for both buildings. The probability of collapse at the *MCE* spectral acceleration (0.20 g) is 0.0073 and 0.021 for Buildings 2A and 2B respectively.

Figure 8 shows the collapse fragility curves from *IDAs* overlaid with those obtained using the FEMA 154 checklist. It shows that the *IDA* collapse results are conservative compared to the FEMA 154 results. For example, the median collapse capacity of Building 2A obtained from the FEMA 154 checklist approach is almost twice that (1.35g) computed from the *IDAs*. It should be noted that the relative difference in the collapse performance of the two buildings is not captured by the FEMA 154 checklist. Using the *IDA* approach, there is a 20% difference in the median collapse capacity of Buildings 2A and 2B. The difference is only 2% when the FEMA 154 checklist is used.



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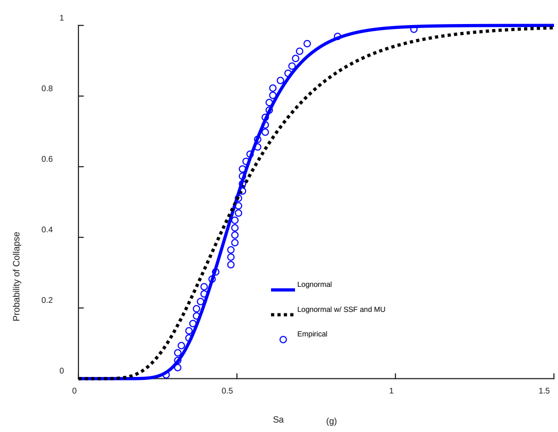


(b)

Figure 7 Collapse fragility curves from incremental dynamic analysis including the effects of spectral shape factor (SSF) and modeling uncertainty (MU) for (a) Building 2A and (b) Building 2B

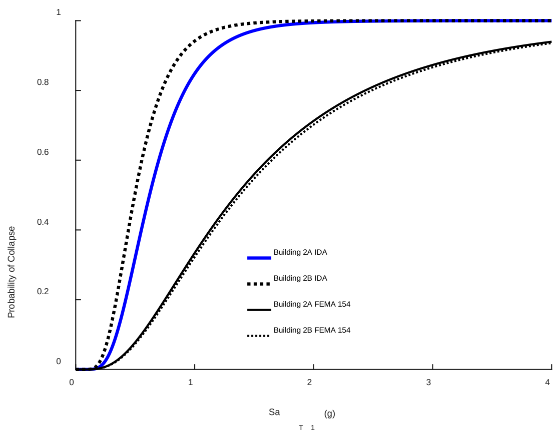


Figure 8 Collapse fragility curves from incremental dynamic analysis (including *SSF* and *MU*) and FEMA 154 checklist for Buildings 2A and 2B

USRC Seismic Rating Assessment

A *USRC* seismic rating assessment is performed for the two building cases using the FEMA P58 approach. Intensity-Based analyses are performed at the *DBE* and *MCE* hazard levels. Only the *DBE* level assessment was used for the *USRC* rating. *REDi* recovery times are computed which includes repair times and impeding factors. A comparative *ASCE-31/ASCE-41* rating was not performed. The construction cost for Buildings 2A and 2B is $149 million and $174 million respectively and commercial office space occupancy type was assumed. The effect of residual drifts on the expected losses and repair time is considered.

A summary of the results of the rating is shown in Table 1. Both buildings received 4, 5 and 2 stars for the safety, damage and repair dimensions respectively. The mean repair cost (normalized by the replacement cost) at the 475-year event is 1.84% for Building 2A and 0.91% for Building 2B. Figure 10 shows the disaggregation of losses at the *DBE* event for the two buildings. For both buildings, the exterior cladding dominated the losses accounting for 45% of the total repair cost. For the safety dimension, the total probability of injuries for Building 2A (0.000544) is about 60% higher than that of Building 2B (0.000319). The difference is consistent with the level of damage for the two buildings as reflected in the repair costs. The median *REDi* functional downtime is 325 days for Building 2A and 307 days for Building 2B. For both buildings, the impeding factors account for more than 80% of the recovery time. This can be observed in Figure 11 which shows that the *REDi* functional recovery time without impeding factors is only 52 days and 22 days for Buildings 2A and 2B respectively. This suggest that the main impediments to a higher star rating for both buildings are the impeding factors.

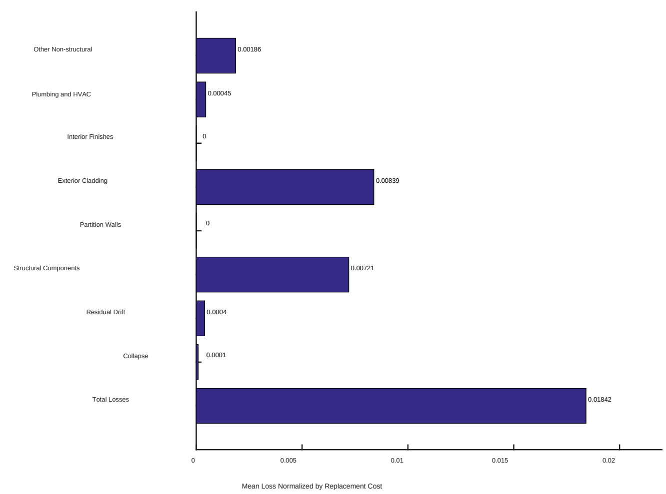
Table 1 Summary of USRC Rating for (a) Building 2A and (b) Building 2B

(a)

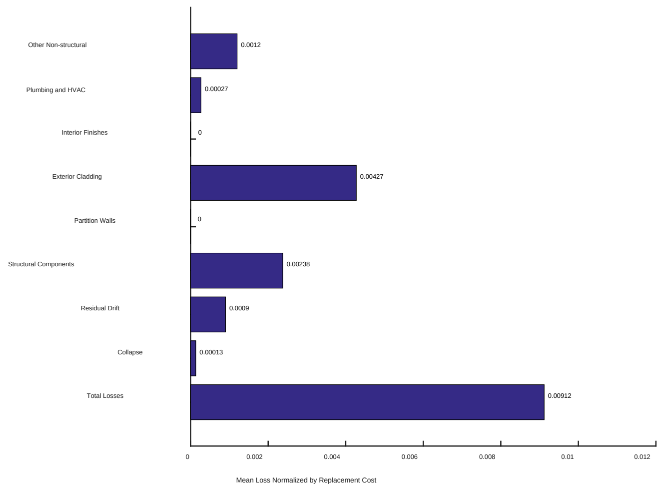
Rating Dimension	Rating	Rating Description
Safety	4 Stars	Total Probability of Injuries: 0.000544 Total Probability of Fatalities: 1.8E-5
Damage	5 Stars	Mean Repair Cost at 475 year event: 1.84 %
Repair	2 Stars	Median REDi Functional Down Time at 475 Year Event (including impedance factors): 325Days

(b)

Rating Dimension	Rating	Rating Description
Safety	4 Stars	Total Probability of Injuries: 0.000319 Total Probability of Fatalities: 2.2E-5
Damage	5 Stars	Mean Repair Cost at 475 year event: 0.91 %
Repair	2 Stars	Median REDi Functional Down Time at 475 Year Event (including impedance factors): 307 Days

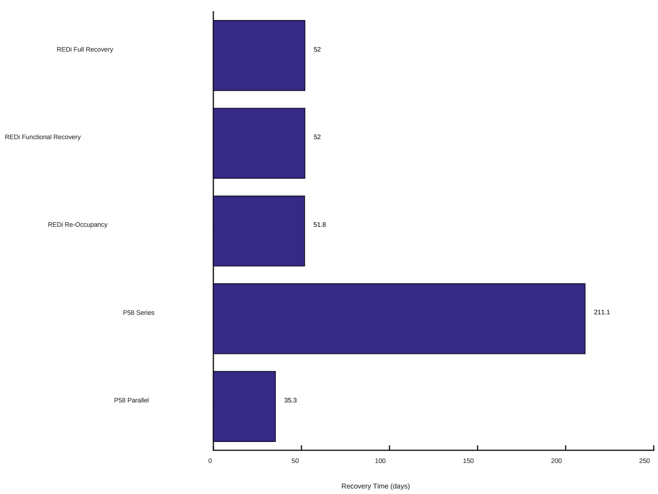


(a)

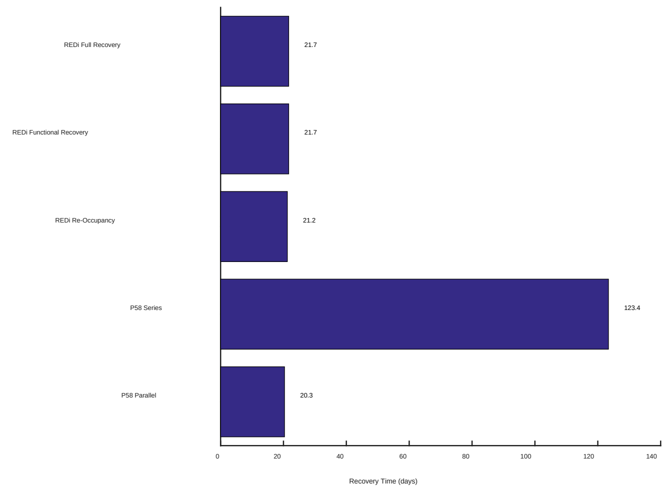


(b)

Figure 9 Disaggregation of losses at the DBE hazard level for (a) Building 2A and (b) Building 2B using FEMA 154 collapse performance and residual drifts not considered (USRC Rating Methodology)



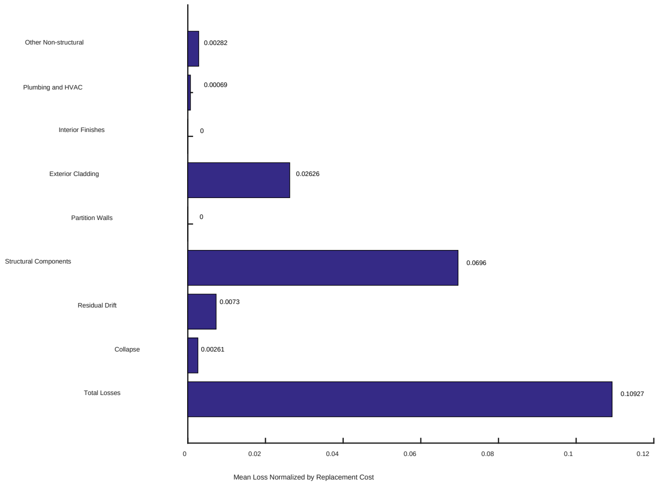
(a)



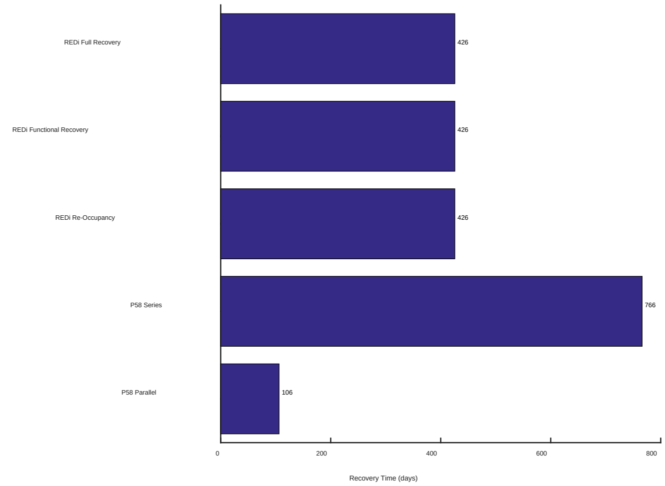
(b)

Figure 10 Comparing recovery times at the DBE hazard level for (a) Building 2A and (b) Building 2B using FEMA 154 collapse performance and residual drifts not considered

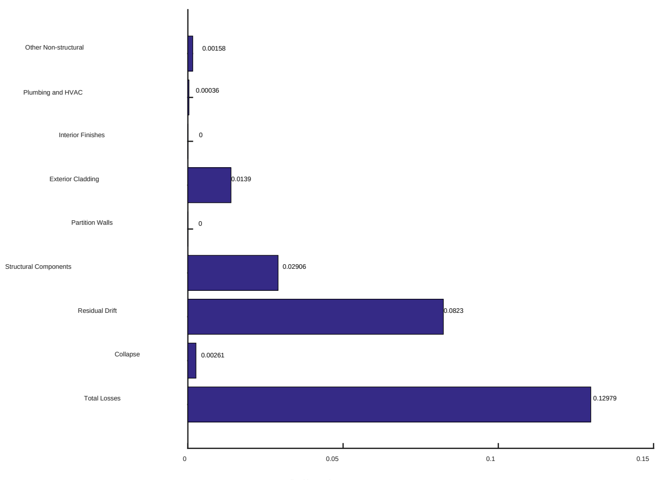
The results of the FEMA P58 assessment at the *MCE* hazard level are summarized in Figures 11 and 12. It is interesting to observe that, at the *MCE* hazard level, the total losses in Building 2B is 20% higher than Building 2A compared to the *DBE* case where it was about half as much. The reason being that, unlike at the *DBE* level, there is a measurable contribution from residual drifts which is significantly higher in Building 2B (as observed in Figure 4). At the *MCE* level, residual drifts account for 68% of the total losses in Building 2B compared to 7% for Building 2A. The residual drifts also affect the *REDi* functional recovery time (shown in Figure 12) at the *MCE* level, which is computed to be 426 days for Building 2A and 300 days for Building 2B without the effect of impeding factors.



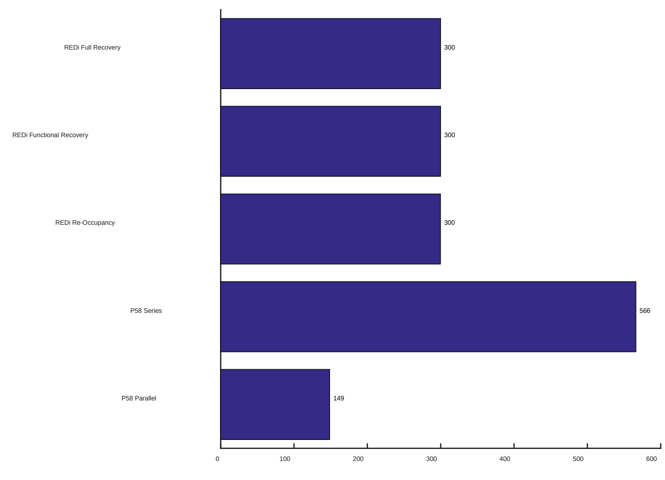
(a)



(a)



(b)



(b)

Figure 11 Disaggregation of losses at the MCE hazard level for (a) Building 2A and (b) Building 2B using FEMA 154 collapse performance and residual drifts not considered

Figure 12 Comparing recovery times at the MCE hazard level for (a) Building 2A and (b) Building 2B using FEMA 154 collapse performance and residual drifts not considered

Conclusion

The *USRC* seismic rating procedure is applied to two variations of a 42-story concrete building with a core wall and special moment frame lateral system. The buildings were developed as part of the *PEER TBI* project. One variation was designed using the prescriptive requirements of the *IBC* 2006 and the other using the *LATBSDC* guidelines. Three-dimensional structural models of the two variants were constructed in *OpenSees* and nonlinear response history analyses were performed using bi-directional inertial loading including *IDAs* to collapse. The intensity-based analyses were performed at the *DBE* and *MCE* hazard levels.

SP3 was used to perform the *USRC* Seismic Rating assessment based on the FEMA P58 methodology. User-defined *EDPs* were incorporated into the assessment using the results from the nonlinear response history analyses. Story drift demands were used to assess the extent of damage to the moment frame elements and other deformation-controlled structural and non-structural components. Chord rotations were used to assess the damage to the core wall and coupling beams. Floor accelerations were used to simulate damage to acceleration-controlled components such as ceilings and mechanical, electrical and plumbing equipment. Residual drifts were used to account for the effect of demolition on the mean repair costs and recovery time. Collapse fragility curves were developed using the *IDA* results, however, the FEMA 154 checklist-based collapse capacity was used in the *USRC* rating assessment.

The two buildings achieve the same *USRC* rating: four, five and two stars for the safety, damage and recovery dimensions respectively. The *USRC* rating is performed at the *DBE* hazard level. The mean repair cost for the code-based and performance-based designs is 1.84% and .91% of the replacement cost respectively. At this intensity level, the repair cost for both buildings is dominated by damage to the exterior cladding, which accounts for about 45% of the losses. For both building cases, impeding factors account for more than 80% of the *REDi* functional recovery time. The results from the nonlinear response history analyses show that the residual drifts are significantly higher in the performance-based design case. This is likely the result of using less boundary element reinforcement in the performance-based design case. In fact, at the *MCE* hazard level, residual drifts dominate the losses for the performance-based design case and the mean repair cost is about 20% higher than the code-based design case.

Acknowledgements

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