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Quantification of the Impact of Detailing on the Performance and Cost of RC Shear Wall Buildings in Regions with High Uncertainty in Seismicity Hazards

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ABSTRACT

This paper aims to quantify the impact of lateral force resisting system detailing on the performance and cost of RC shear wall buildings in regions with high uncertainty in seismicity. Six archetype buildings are designed and detailed as per the IBC'12. The results showed lower collapse margin ratios for the ordinary detailing, which indicates poorer performance as suggested by engineering intuition. Furthermore, it is demonstrated that opting to utilize special RC shear walls substantially improves the structural response. The marginal initial cost increase associated with special detailing is considerably outweighed by the resulting savings in the repair and downtime costs.

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Seismicity Uncertainty; RC Shear Walls; FEMA P695; Seismic Performance; Fragility Analysis; Earthquake Damages

1. Introduction and Background

One of the most important steps in the structural design phase of any building is the proper estimation of the expected loads. The estimated loads should represent horizontal and vertical loads that are likely to affect the structure during its life time. Among other loads, lateral dynamic loads due to wind and seismic forces generally exhibit the highest degree of uncertainty. The seismic forces are estimated from the ground motion parameters derived for each location, based on its geological and seismological conditions. In some regions, there is a high level of uncertainty when it comes to estimating the seismic hazard. For instance, in United Arab Emirates (UAE), there is a significant uncertainty regarding the actual seismicity level which should be designed for. This is due to the insufficient information about the seismological conditions and the lack of available recordings of historical events. Those data are necessary for accurately setting the level of seismicity [AlHamaydeh *et al.*, 2013]. Over the past two decades, there have been a substantial number of research studies concerned with UAE and Dubai seismicity such as Al Haddad *et al.* [1994], Grünthal *et al.* [1999], Abdalla and Al-Homoud [2004], Sigbjornsson and Elnashai [2006], Pascucci *et al.* [2008], Aldama-Bustos *et al.* [2009], Shama [2011], and Khan *et al.* [2013]. These seismic hazard studies presented significantly varying estimates of Dubai seismicity, as illustrated in Table 1. Unfortunately, the reported estimates vary from very low seismicity level as provided by Aldama-Bustos *et al.* in 2009

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Table 1. Variations in estimated PGA for Dubai.

Study by	Year	PGA (10% exceedance probability in 50 years) (g)
Al Haddad <i>et al.</i>	1994	<0.05
Grunthal <i>et al.</i>	1999	0.32
Abdalla and Al Homoud	2004	0.14
Sigbjornsson and Elnashai	2006	0.16
Pascucci <i>et al.</i>	2008	0.06
Aldama-Bustos <i>et al.</i>	2009	<0.05
Shama	2011	0.17
Khan <i>et al.</i>	2013	0.047

to very high seismic hazard reflected in Shama's probabilistic seismic hazard study in 2011. Therefore, based on these estimates, the implications of the seismicity level on the design could vary from minor or no seismic design requirements to mandatory special design and detailing. Furthermore, Dubai municipality enforces a minimum seismic design requirements of zone 2A based on the 1997 Uniform Building Code (UBC'97). The minimum requirements were raised in 2013 to zone 2B for buildings higher than nine stories and kept as zone 2A for buildings between four to nine stories. However, no specific requirements were mentioned for buildings less than four stories. This revision came as a proactive response from the municipality. Following the tremor that was felt and recorded in Dubai in 2013, the municipality sent a memo to all engineering offices amending their minimum seismic design requirements. Therefore, there is a clear uncertainty regarding the adequate seismic design level that should be followed in Dubai. Given Dubai's rapid economic growth, the lack of specific seismicity level complicates the vulnerability to earthquake effects. In addition, other regions which are characterized with low or moderate seismic hazard, such as France, Belgium, central United States, and parts of eastern United States, are considered very vulnerable to unexpected large earthquakes [Han and Choi, 2008]. Historically, several large earthquakes took place in regions which were known to have low and moderate seismicity, for example, the Mississippi Valley earthquake in 1811 and Charleston earthquake in 1886. The occurrence of such unexpected large earthquakes in regions categorized as low seismicity areas would result in significant damage and collapse to many structures [Han and Choi, 2008]. This is because the structures in such areas are typically designed without any special design and detailing considerations for seismic performance. They are usually designed to resist only vertical forces from gravity loads.

The objective of this study is to evaluate the seismic performance of RC shear wall buildings designed in regions with high uncertainty in seismicity level. This paper attempts to answer the question: what are the quantified performance and cost implications for RC shear wall buildings (designed for low seismicity) if they get subjected to major seismic events? The growing concern about the performance of buildings designed for low seismicity during unexpected large seismic events is the primary motivation to assess the potential consequences. In addition, the present study investigates and quantifies the impact of lateral force resisting system detailing on the structural performance and associated construction and repair costs. According to IBC'12, it is permitted to use either ordinary or special RC shear walls as the main lateral force resisting system in regions with low seismic hazard. Despite the apparent advantages of simple detailing when opting to use ordinary RC shear walls, utilizing special RC walls would result in lower demands for

the seismic design forces (due to dividing by a higher R -factor) as well as more favorable structural performance. The use of special detailing in RC shear walls can be expected to enhance the drift response and energy dissipation capacity of the building. For instance, it has been recently shown by Li *et al.* [2015] through experimental testing of RC shear walls under simulated seismic loading that an increase in the boundary element confinement reinforcement substantially improves the overall seismic response. The beneficial attributes were represented by an increase in deformation and energy dissipation capacities. Furthermore, it has been also demonstrated that the ductility capacity of ordinary RC shear walls without seismic detailing might be insufficient in regions with expected moderate seismicity [Kuang and Ho, 2007]. This conclusion was supported by testing of ordinary RC shear walls under reversed cyclic loading. Therefore, given the seismic vulnerability of ordinary RC shear wall buildings designed for low seismicity and the existing uncertainty regarding the seismicity level in some regions, such as in Dubai, it is essential to evaluate and quantify the detailing impact on the performance and life cycle cost. To that effect, six archetype RC shear wall buildings are selected to represent buildings designed for low seismicity. The buildings are made of two groups and each group consists of 6-, 9-, and 12-story archetypes. The three archetypes in each group spanned the typical range of number of stories and fundamental periods of the relevant building inventory stock. One group of 6-, 9-, and 12-story archetypes is furnished with special seismic detailing, while their counterparts had ordinary detailing. All buildings are designed and detailed according to IBC'12 [ICC, 2012] relevant standards for an arbitrary, but representative low seismic hazard level ($S_s = 0.18\text{ g}$ and $S_1 = 0.06\text{ g}$). The ground motion input parameters (i.e., S_s and S_1) are primarily selected to result in a seismic design category B (i.e., low seismicity) according to IBC'12. The seismic performance and collapse margin ratio (CMR) of the reference buildings are evaluated according to the FEMA P695 [2009] methodology. FEMA P695 seismic assessment and collapse evaluation approach is based on nonlinear static and dynamic analyses. Furthermore, the seismic vulnerability of the reference buildings is evaluated using fragility curves at three performance levels adopted from ASCE41-13 [2013]. Finally, the impact of the seismic detailing level (i.e., special vs. ordinary) on the construction and repair costs is quantified considering both structural and nonstructural components. This quantification is necessary to link the study outcomes with meaningful conclusions to different stakeholders.

2. Reference Archetype Buildings

2.1. Archetype Buildings Description

The buildings considered in this study have a generic plan layout, shown in Fig. 1, which represents typical office buildings. They are RC buildings with shear walls as the lateral force resisting system. The structural heights are 24, 36, and 48 m, which correspond to 6-, 9-, and 12-story buildings with typical floor height of 4 m, respectively. This range of heights covers the expected fundamental period range of the relevant building inventory stock. For three of the reference buildings, the RC shear walls are special (i.e., satisfying American Society of Civil Engineers [2010] special design and detailing requirements), and for the other three buildings, the walls are ordinary walls, as classified by ASCE7-10. According to ASCE7-10, the special walls are assigned response modification factor (R)

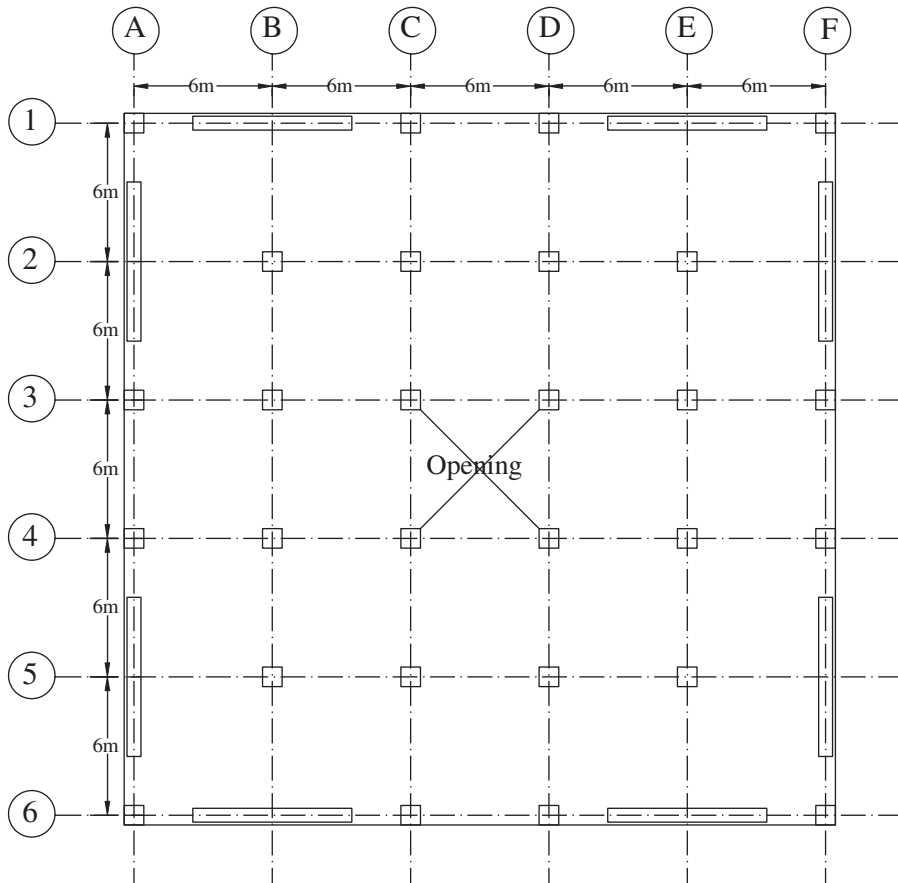


Figure 1. Typical floors plan view.

equal to 6 and deflection amplification factor (C_d) of 5. On the other hand, the R and C_d for the ordinary walls are equal to 5 and 4.5, respectively. The reference buildings' plan consists of five 6-m bays in north–south and east–west directions. This results in a total plan dimension of 30 m $\times$ 30 m. A regular floor plan layout is selected to represent a range of buildings rather than a specific case with particular plan irregularities or torsional sensitivity. The lateral force resisting system (i.e., RC shear walls) is located at the perimeter of the buildings as recommended by Moehle *et al.* [2012] in National Earthquake Hazards Reduction Program technical brief no. 6. Such relatively simple configurations of RC shear wall buildings are often used by researchers when a class of buildings is studied rather than a specific building with particular plan and torsional irregularities [e.g., Gogus and Wallace, 2015]. Locating the shear walls along the perimeter maximizes its torsional resistance. Furthermore, it enhances the ductile response by lowering the axial forces acting on the walls. The reference buildings' gravity system is made up of RC square columns and cast *in-situ* 200-mm thick flat slabs without beams. The characteristics of reference buildings are shown in Table 2. It is observed that the buildings' natural periods, which were estimated using 3D finite element models considering cracked concrete properties in ETABS, are on the flexible side. This is a

Table 2. Buildings' details.

Ref. building	No. of floors	Shear wall type	Natural period (s)	Vertical seismic force distribution coefficient "k"
6S-S	6	Special	2.92	1.20
6S-O	6	Ordinary	2.92	1.20
9S-S	9	Special	3.51	1.36
9S-O	9	Ordinary	3.51	1.36
12S-S	12	Special	3.93	1.50
12S-O	12	Ordinary	3.93	1.50

consequence of the relatively lower lateral stiffness resulting from the highly optimized design and the relatively heavy seismic weight. The buildings' designs presented were aiming at minimizing the inertial forces. In addition, the chosen structural system of flat slabs with columns relies solely on the shear walls for the lateral stiffness as there is no framing action. Thus, the optimized design combined with the low seismic forces resulted in relatively low shear wall area compared to the total floor area. However, the estimated natural periods are still within acceptable approximations expressions [e.g., Crowley and Pinho, 2004; Crowley and Pinho, 2010] for moment resisting frames. For instance, a 48-m tall RC frame building is estimated to have a yield period of $0.1H$ (H : building height in meters); for the 12-story RC shear wall building, the analytical period is determined to be 3.93 s (empirical formula produces a 4.8-s estimate). The difference is naturally attributed to the European building stock used for the development of the empirical formula and the fact that it is intended for framed structures, among other reasons. Furthermore, for the 12-story building, the natural period was estimated using two very well-established techniques, Rayleigh's method and the generalized coordinate approach. The calculation showed that the natural period is ranging from 3.41 s to 3.97 s, depending on which method is used. This kind of design optimization technique is not uncommon and many similar approaches can be found in the literature [e.g. AlHamaydeh et al., 2013, 2017a and 2017b]. Nevertheless, the relatively flexible building designs presented herein are expected to have favorable seismic response stemming from the wise design approach which minimizes the inertial forces. It is worthy of noting that these analytical periods were not used for the seismic force calculations, and they are capped with the maximum code values. The simplified empirical expressions given in design codes generally produce highly conservative estimates of natural periods. Such relationships are developed for force-based seismic design procedures and shall result in a conservative estimate of the base shear [Crowley and Pinho, 2004]. It should be noted that the buildings' ID refers to the number of stories (i.e., 6, 9, or 12) and type of shear walls (i.e., Special or Ordinary).

2.2. Material Properties and Design Loads

The concrete compressive strength (f'_c) is assumed to be 28 MPa (4000 psi) for gravity system (i.e., columns and slabs). However, for the shear walls, a slightly higher compressive strength (f'_c) of 38 MPa (5500 psi) is assumed. The reinforcement is assumed to have a yield strength (f_y) of 420 MPa (60 ksi). The minimum design loads are following ASCE7-10 minimum values for office buildings. Therefore, live load is 2.4 kPa for typical floors and 1 kPa for the roof floor. Floors superimposed dead loads are 3.6 kPa, excluding self-

weight of structural elements, with 0.72 kPa along the perimeter to account for the curtain wall. To achieve the objective of quantifying the impact of detailing in buildings located in low seismic hazard regions, the selected reference buildings are designed for the lateral forces due to seismic loads only. Although wind lateral forces might govern the design of some buildings when seismic loads are very low, seismic loads are of particular interest in this study. The choice of seismic loads over wind loads allows investigating the impact of detailing on structural performance, construction, and repair costs. According to the design code (i.e., IBC'12), there is no specific detailing requirements for the lateral force resisting system when designed for wind loads only. In addition, as the studied buildings are located in an arbitrary region representing the low seismicity, the calculation of wind loads would be very subjective depending on the selected wind speed/pressure. Thus, the implications of wind loads on the structural performance and cost would be a location-dependent parameter, which is beyond the scope of the current study. For calculating seismic loads, IBC'12 response spectrum is used with the ground motion input parameters of $S_s = 0.18$ g and $S_1 = 0.06$ g (selected to result in a seismic design category B) to simulate an arbitrary location with low and uncertain seismicity level. A site class D is assumed, as recommended by IBC'12 when there are no specific information about the soil class in the building location. IBC'12 response spectrum is a uniform hazard spectrum constructed for a very rare earthquake event with 2475-year return period. Figure 2 depicts the elastic design response spectrum with the six reference buildings, represented by the approximate time period (T_a) as calculated in accordance to ASCE7-10.

2.3. Design Summary

The design and detailing of the reference buildings are performed according to IBC'12 relevant standards. IBC'12 is a commonly used building code in many regions outside the

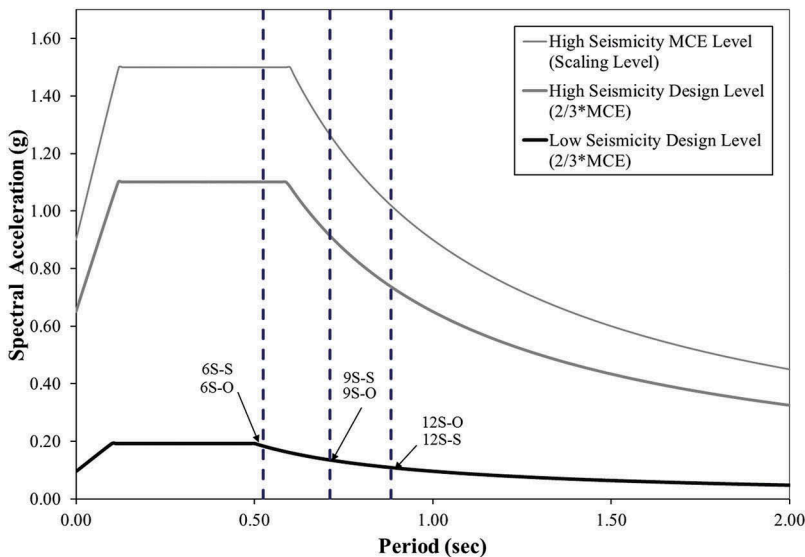


Figure 2. Elastic (without R-factor) MCE and design response spectra for reference buildings.

United States such as the UAE. For the six reference buildings, the gravity system is designed first to start with reasonable approximation of the seismic weight needed to calculate the design base shear. Gravity columns are designed to resist all vertical forces resulting from gravity loads. Columns are not designed to be part of the lateral force resisting system; yet, they are designed to safely undergo the imposed deformations by seismic loads (deformation compatibility). This is achieved by ensuring that columns have sufficient resistance toward bending moments and shear forces induced at maximum allowable inter-story drift by IBC'12 (i.e., 2%). Linear elastic analysis is performed for the reference buildings using 3D models on ETABS [CSI, 2015]. Flat slabs and columns are design using in-house spread sheets. Columns' cross-sections and reinforcement are grouped and changed every three floors to result in an optimized design. Summary of gravity columns' dimensions and reinforcement details is shown in Fig. 3. Flat slabs are reinforced with T16 reinforcement bars spaced at 125 mm in top and bottom layers in both directions. At top of columns, additional 2-m long T20 reinforcement bars are provided with 125 mm spacing in both directions.

The lateral force resisting system, which consists of eight RC shear walls, is designed based on the static equivalent lateral force method given in ASCE7-10. This method is applicable for all reference buildings since they are regular buildings with maximum height less than 48 m and seismic design category B. The resulting lateral forces are distributed horizontally among the shear walls based on their relative rigidities. First, the lateral system is designed to limit the peak inter-story drifts within design code limit (i.e., 2%) by changing the walls' in-plan length. Then, the shear walls are designed to resist the imposed overturning moments and shear forces by providing sufficient vertical and horizontal reinforcement. Shear walls are designed with the precaution that higher mode effects might be predominant for the tall buildings (i.e., the 9 and 12-story buildings). It was highlighted by Bachmann and Linde [1995], Tremblay *et al.* [2001], Panneton *et al.* [2006], and El-Sokkary *et al.* [2013] that higher mode effects might result in higher demands on upper floors that have reduced cross-sections. Therefore, the upper six floors in the 9- and 12-story buildings are given the same cross-section and reinforcement details. The 6-story buildings are designed with a single cross-section, but the boundary

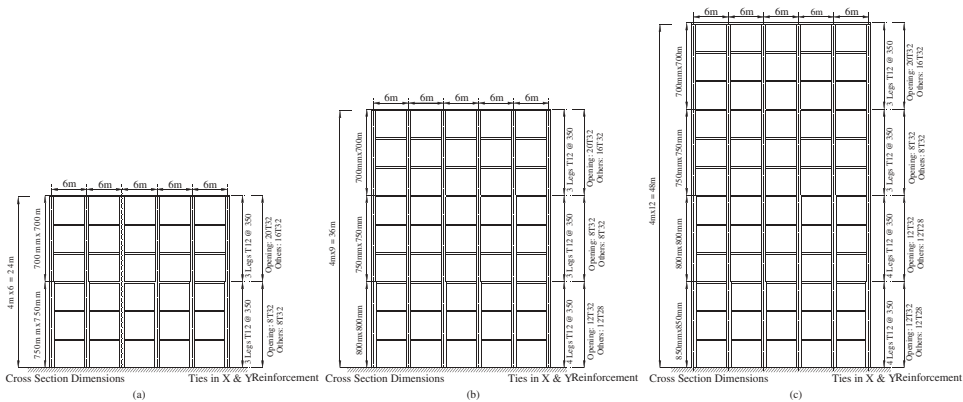


Figure 3. Gravity columns dimensions and reinforcement details: (a) 6S-S and 6S-O; (b) 9S-S and 9S-O; (c) 12S-S and 12S-O.

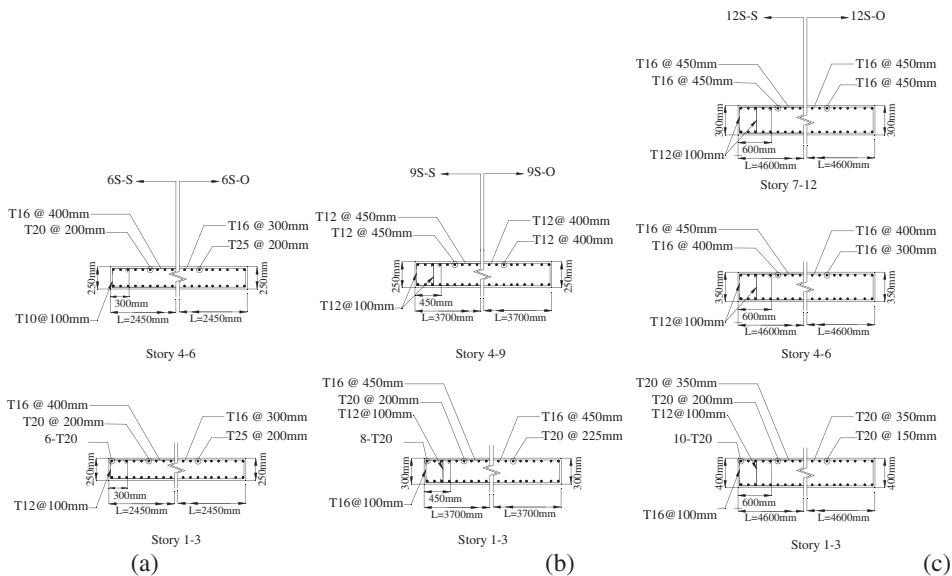


Figure 4. RC shear walls dimensions and reinforcement details: (a) 6S-S and 6S-O; (b) 9S-S and 9S-O; (c) 12S-S and 12S-O.

elements are terminated after the third floor. For the reference buildings with the special RC shear walls, the need for confined boundary elements is based on the displacement approach given in ACI318-11 [2011]. Based on this approach, boundary elements are required when the concrete strain at ultimate loads exceeds the maximum allowable compressive strain of concrete. The strength design of the shear walls is performed using the Integrated Engineering Software (IES) [2015]. Summary of shear walls' dimensions and reinforcement details is illustrated in Fig. 4 for the six archetype buildings.

3. Nonlinear Modeling and Analysis

In this study, the nonlinear modeling of reference buildings is performed on IDARC-2D. IDARC-2D is a competent simulation tool for concrete structures [Reinhorn *et al.*, 2009] and has been extensively used by researchers to quantify inelastic response of structures, e.g., AlHamaydeh *et al.* [2011], Gracia *et al.* [2010], and Roh and Reinhorn [2010]. In IDARC-2D, the structural elements are simulated based on macro-modeling approach using fiber-based wall elements with concentrated plasticity (spread plasticity and yield penetration features are not utilized for computational efficiency). This modeling approach is validated against experimental measurements by Reinhorn *et al.* [2009]. The hysteretic response of structural components is simulated based on the three-parameter hysteretic model developed by Park *et al.* [1987] and implemented within IDARC-2D. This multi-linear hysteretic model incorporates stiffness degradation and strength deterioration based on ductility and energy dissipation rules. Recommended model-specific coefficients for strength deterioration and stiffness degradation have been used in modeling of the special and ordinary RC shear walls [Reinhorn *et al.*, 2009]. The utilized coefficients in this study are shown in Table 3. The values of the hysteretic modeling

Table 3. Parameters of the multi-linear hysteretic model by Park *et al.* [1987], as recommended in IDARC-2D technical report, Reinhorn *et al.* [2009].

Parameter	Special RC shear walls	Ordinary RC shear walls
Stiffness degrading parameter (α)	130	7
Ductility-based strength degrading parameter (β_1)	0.035	0.45
Energy-controlled strength degrading parameter (β_2)	0.03	0.375
Slip or crack—closing parameter (γ)	0.5	0.15

parameters of the special RC shear walls were taken as the average recommended values for *no degradation* and *mild degradation* appropriate for special detailing performance. On the other hand, for the ordinary RC shear walls, the modeling parameters were taken as the average recommended values for *severe degradation* and *moderate degradation*. The two modeling decisions resulted in similar hysteric responses found in the literature for ordinary and special detailed RC shear walls [Kuang and Ho, 2007; Oh *et al.*, 2002]. P-delta effects are accounted for through explicit modeling of the gravity system in the nonlinear model. This is opposed to simplified leaning column approach. It allows for damage assessments of both gravity and lateral systems. The inelastic seismic response of the reference buildings is quantified by nonlinear static and dynamic analyses performed using IDARC-2D. Since the considered buildings have a regular symmetric plan, mass participation of torsional modes is negligible. Therefore, a two-dimensional analysis is deemed sufficient. The seismic response quantification is based on FEMA P695 approach.

3.1. Nonlinear Pseudo Static Pushover analysis

Displacement controlled pushover analysis is performed to assess the reference buildings strength and deformation capacities. In addition, pushover analysis is necessary to validate the nonlinear models and quantify the period-based ductility of the buildings. An inverted semi-triangular displacement profile is used in the analysis to represent the first mode of vibration as well as the higher mode effects. The higher mode effects are captured through the appropriate vertical distribution of seismic loads (through the story height/elevation power: “ k ”), as follows:

$$F_x = C_{vx} V \quad (1)$$

$$C_{vx} = \frac{W_x h_x^k}{\sum_{i=1}^n W_i h_i^k} \quad (2)$$

The seismic force vertical distribution profile was calculated in accordance with the IBC’12 for design purposes as expressed above in Eqs. (1) and (2). The same vertical distribution profile was used for the displacement profile in the pushover analyses.

The intensity of the applied pushing function is gradually increased until there is 20% degradation in the structure’s base shear capacity. This allows calculating the ultimate roof displacement corresponding to 20% strength degradation, as defined by FEMA P695 [2009]. Then, the period-based ductility is calculated as the ratio between ultimate displacement and effective yield displacement. Definitions of ultimate and effective yield displacements, according to FEMA P695, are pictorially represented in Fig. 5.

The pushover capacity curves of the reference buildings are presented in Fig. 6. It is observed that, although buildings 6S-S, 9S-S, and 12S-S are designed with special RC shear

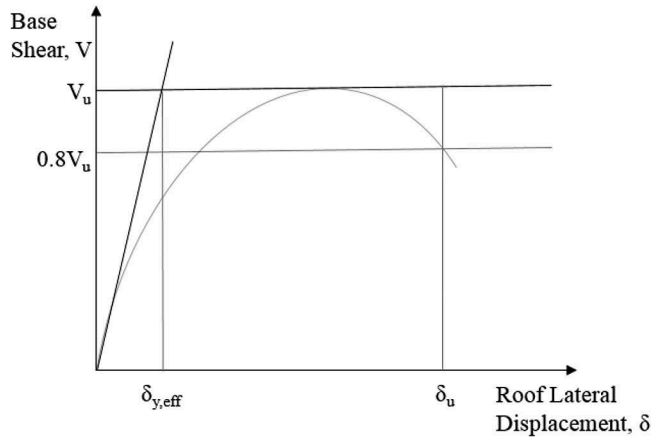


Figure 5. Definition of ultimate and effective yield displacements (adapted from FEMA P695 [2009]).

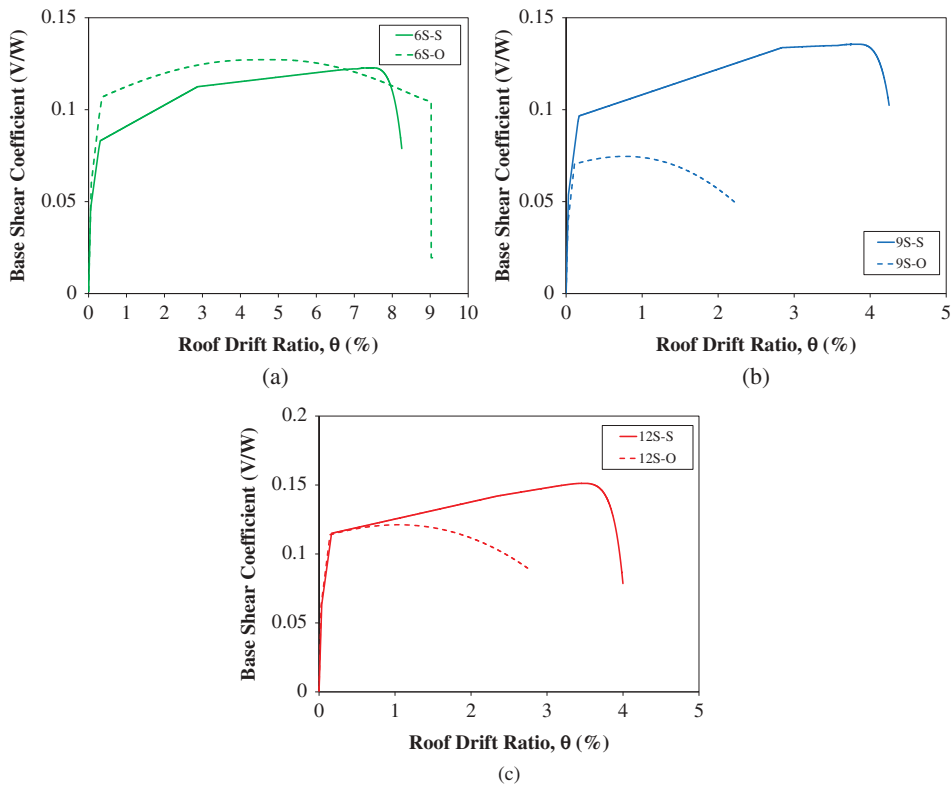


Figure 6. Pushover capacity curves (displacement control): (a) 6S-S and 6S-O; (b) 9S-S and 9S-O; (c) 12S-S and 12S-O.

walls, the backbone curves indicate obvious post-peak degradation in stiffness and sudden drop in strength. This could be attributed to the very low seismicity level adopted in the wall's design. Similarly, the capacity curves for the three designs with ordinary RC shear walls (i.e., 6S-O, 9S-O, and 12S-O) show severe degradation in stiffness and deterioration in

strength. The buildings reach their peak base shear strength and suddenly drop afterward. This is very clear in the 12-story (12S-O) and 9-story (9S-O) buildings. In the 6-story building (6S-O), the drop in strength is less severe at the post-peak. Furthermore, the base shear coefficient is 0.12 for 6S-S, 0.14 for 9S-S, and 0.15 for 12S-S. On the other hand, base shear coefficients are 0.13, 0.075, and 0.12 for buildings 6S-O, 9S-O, and 12S-O, respectively. In 9- and 12-story buildings, the choice of special shear walls resulted in a slight enhancement in performance. This is reflected by the increased base shear capacity and ductile response of the special walls. However, it is interesting to note that the 6-story building with ordinary RC shear walls (6S-O) has marginally better performance than the building designed with special RC shear walls (6S-S). As shown in Fig. 6a, 6S-O has slightly higher yield strength and base shear capacity compared to 6S-S. This observation can be explained by the fact that according to IBC'12, ordinary RC shear walls are designed for higher forces (smaller response modification factor, $R = 5$) compared to special RC shear walls ($R = 6$). Furthermore, for the squat shear walls, the design is mostly governed by strength requirements rather than drift requirements. Therefore, the ordinary RC shear walls cross-section required more shear and flexural reinforcement compared to the special RC shear walls. This resulted in a slightly higher capacity for the ordinary walls. For the taller buildings (9- and 12-stories), the shear walls design is mostly governed by drift requirements. Hence, the design resulted in the same cross-section for both special and ordinary walls, and the provided reinforcement was based on the minimum code requirements. With the same flexural reinforcement and cross-section dimensions, special shear walls will clearly show an enhanced performance (strength, ductility, and toughness) due to its special detailing and the presence of the confined boundary elements. The six reference buildings' estimated period-based ductility according to FEMA P695 is more than 8. Furthermore, final damage states at 20% strength degradation show plastic hinging at the critical section at the shear wall base. This mechanism is matching the design code assumption for the static design approach.

3.2. Nonlinear Incremental Dynamic Analysis

Nonlinear incremental dynamic analysis (IDA) is performed to examine the dynamic response of the reference buildings. IDA allows investigating the response of structures from the linear elastic range through the nonlinear behavior until excessive lateral displacement and collapse [Vamvatsikos and Cornell, 2002; Vamvatsikos and Cornell, 2004]. In addition, dynamic analysis allows evaluating the designs for the possible presence of higher mode effects. IDA is performed using a suite of 22 ground motion records given in the far-field set of FEMA P695. The records are obtained from PEER NGA database (PEER, n.d.). These records are a standard set that is accepted globally in the earthquake engineering field to be representative of strong ground motion associated with far fields. They have been carefully selected to cover wide range of Peak Ground Acceleration (PGA)/Peak Ground Velocity (PGV) ratios from 0.38 to 1.51 g s/m, which makes the ensemble critical for low-, mid-, and high-rise buildings. Furthermore, the structure's response depends highly on the characteristics of each particular ground motion record. Therefore, for an accurate estimation of the seismic performance, a sufficient number of ground motion records are used. Additionally, the number of selected records (22 records with 44 combinations of horizontal and vertical components suitable

for 2D analysis) inherently possesses the record-to-record variability essential for ensuring statistically appropriate results. These natural records are obtained from earthquake events having large magnitudes ranging from 6.5 to 7.6 and originating from distant sources more than 10 km away from fault rupture. Figure 7 shows the response spectra of the records at an intensity scale factor of 1. The choice of far-field records simulates a possible seismic hazard scenario originating from distant sources that might affect regions with low seismicity. In IDA, all the three components of the records, two horizontal and one vertical, are considered. The inclusion of the vertical component in the IDA increases the accuracy of estimating expected damages caused by earthquakes. The ground motion vertical component is generally attributed to the arrival of the vertical compression waves (P-waves). Naturally, compression waves have shorter wave length, higher frequency, and lower energy compared to shear waves (S-waves). Most of P-waves energy is stored in high frequency ranges and thus it is proven to be damaging to buildings with vertical periods close to those ranges [Elnashai and Di Sarno, 2008]. Additionally, many researchers, such as Collier and Elnashai [2001] and Papazoglou and Elnashai [1996], emphasized on the damaging effect of ground motions vertical component and recommended including it in the design of earthquake resistant structures. The records are scaled using FEMA P695 [2009] methodology such that at the structure's natural fundamental period, the median spectral acceleration of the far-field record set matches the maximum considered earthquake (MCE) response spectrum. In this study, the MCE is selected to represent an unexpected large earthquake event. Thus, the ground motion input parameters, S_s and S_1 , were selected as 1.65 and 0.65 g, respectively, resulting in a seismic design category "D" effects. The design and MCE level (i.e., 1.5R times the design level) response spectra of the selected high seismicity are shown in Fig. 2. In IDA, the records are multiplied and scaled up with increments of 0.1 from 0 to 2.5 g. First mode, 5% damped spectral acceleration, $S_a(T_1, 5\%)$, is chosen as the intensity measure (IM) because of its competency in reducing results scatter. Hence, it reduces the number of required runs and provide a complete characterization of the response [Vamvatsikos and Cornell, 2001; Vamvatsikos and Cornell, 2004]. In addition, the use of the first mode spectral acceleration (with 5%

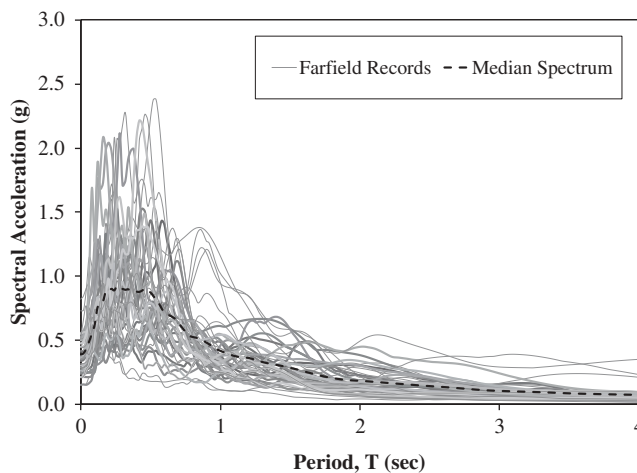


Figure 7. Far-field ground motion records response spectra.

damping ratio) as a measure of the expected ground motion shaking intensity reduces the uncertainty in results compared to the use of PGAs [Jeong *et al.*, 2012]. Furthermore, in this study, maximum inter-story drift ratio is selected as the damage measure (DM) to capture global dynamic instabilities [Vamvatsikos and Cornell, 2001; Vamvatsikos and Cornell, 2004]. Peak inter-story drift ratios provide meaningful quantifications of the building's damage states [Jeong *et al.*, 2012]. IDA results are post-processed and used in preparing IDA curves which are utilized in defining the expected seismic performance of structures.

Figure 8 shows IDA curves for buildings 6S-O, 9S-O, and 12S-O designed with ordinary RC shear walls. In all the three buildings, collapse is occurring below spectral accelerations of 2 g. The 6-story building (6S-O) has few flat lines above MCE level, but more lines below it indicating higher probability of collapse at spectral accelerations lower than MCE level. On the other hand, the other two buildings (9S-O and 12S-O) have most of the flat lines below MCE level. This highlights that most structural collapse incidents are at levels below MCE. Furthermore, as shown in Fig. 8, the three designs experience a bit of weaving and softening in most records. In most cases, for the buildings designed with ordinary RC shear walls, collapse is seen at low drift ratios. Therefore, based on the IDA curves, the expected performance of buildings with ordinary RC shear walls is clearly very poor. The median collapse intensity ($\hat{S}_{CT}$), spectral acceleration at which 50% of the records causes collapse, is 0.43 g for 6S-O, 0.25 g for 9S-O, and 0.24 g for 12S-O. Based on

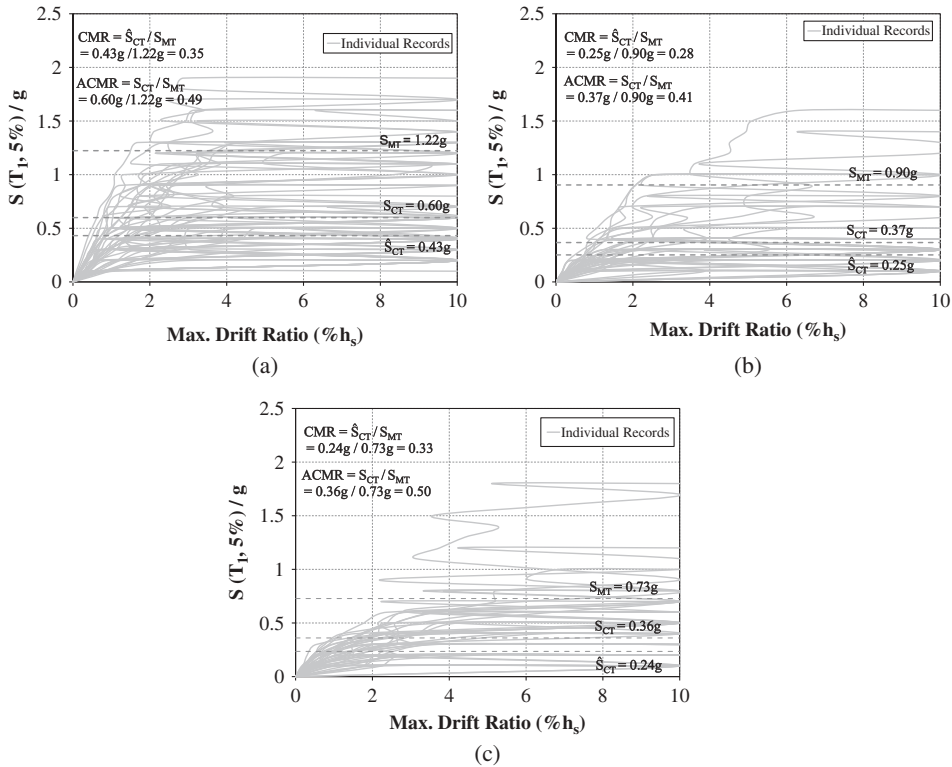


Figure 8. Incremental dynamic analysis curves: (a) 6S-O; (b) 9S-O; and (c) 12S-O.

the calculated median collapse intensity ($\hat{S}_{CT}$), the CMR is calculated for each building as the ratio between its $\hat{S}_{CT}$ and the MCE level spectral acceleration (S_{MT}) (i.e., $\hat{S}_{CT}/S_{MT}$). The calculated CMRs are 0.35, 0.28, and 0.33 for buildings 6S-O, 9S-O, and 12S-O, respectively. Given the embedded uncertainty in input ground motion records, design standards, material and component testing data, and nonlinear modeling, the CMRs have to be adjusted. The Adjusted Collapse Margin Ratio (ACMR) accounts for the various uncertainty sources as well as the spectral content of the input ground motion records. The uncertainty modeling along with the calculated ACMR for the reference buildings is presented in the next section.

Similarly, IDA curves are given for buildings with special RC shear walls (i.e., 6S-S, 9S-S, and 12S-S) in Fig. 9. It is observed that the 6-story building (6S-S) exhibits significant hardening compared to 9- and 12-story buildings (9S-S and 12S-S) which are experiencing a clear softening response to most ground motion records. The IDA curves for the three designs show that most of the flat lines which represent structural collapse are below the MCE level (S_{MT}). This clearly indicates substandard performance and collapse at spectral accelerations lower than S_{MT} . Additionally, in general, buildings 6S-S, 9S-S, and 12S-S are approaching excessively large lateral displacements at low drift ratios except for some records with building 6S-S. The median collapse intensity ($\hat{S}_{CT}$) is 0.55 g for 6S-S, 0.41 g for 9S-S, and 0.37 g for 12S-S.

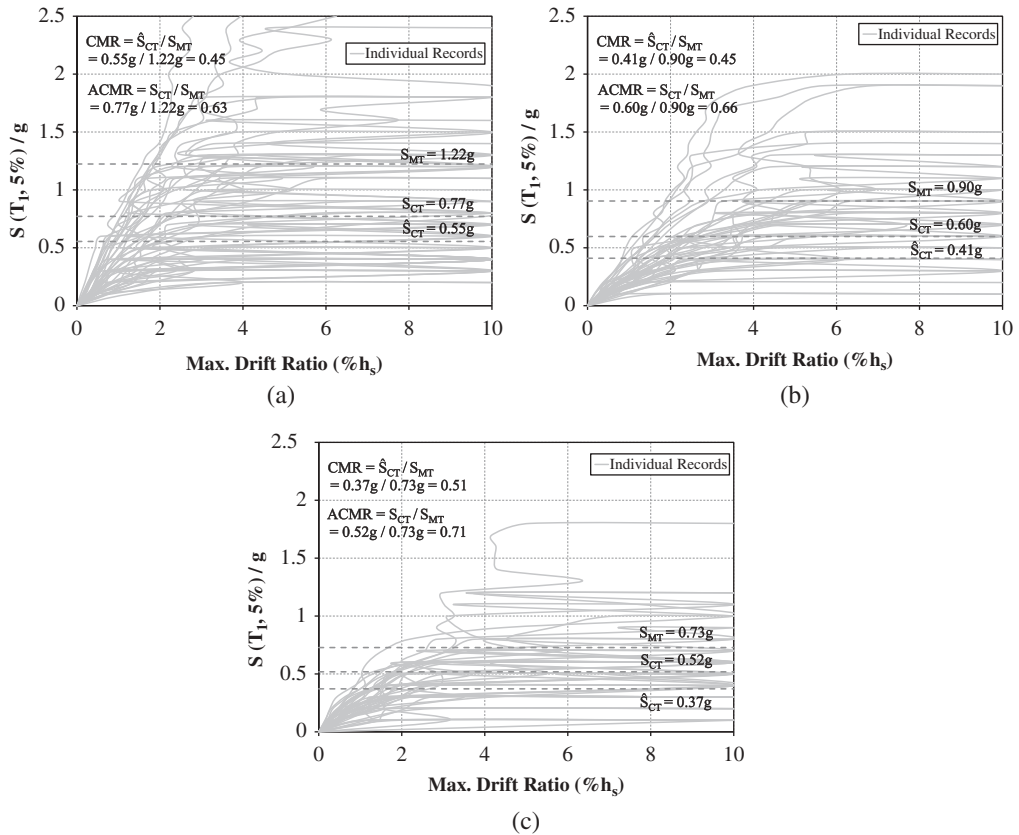


Figure 9. Incremental dynamic analysis curves: (a) 6S-S; (b) 9S-S; and (c) 12S-S.

for 12S-S. Furthermore, CMRs are 0.45, 0.45, and 0.51 for buildings 6S-S, 9S-S, and 12S-S, respectively. Therefore, designing the buildings with special instead of ordinary RC shear walls resulted in 28%, 64%, and 54% increase in median collapse intensity ($\hat{S}_{CT}$) for 6-, 9-, and 12-story buildings, respectively. As expected, the choice of special shear walls over their ordinary counterparts resulted in a significant improvement in the seismic performance and capacity. The ACMRs considering the embedded uncertainty sources are presented in next section.

It is worth mentioning that the collapse mechanism seen in dynamic analysis results significantly differ from the final damage state due to static pushover analysis. As shown in Fig. 10a, the dynamic analysis resulted in plastic hinging at several floors which initiated at higher floors where the wall thickness and reinforcement were reduced. On the other

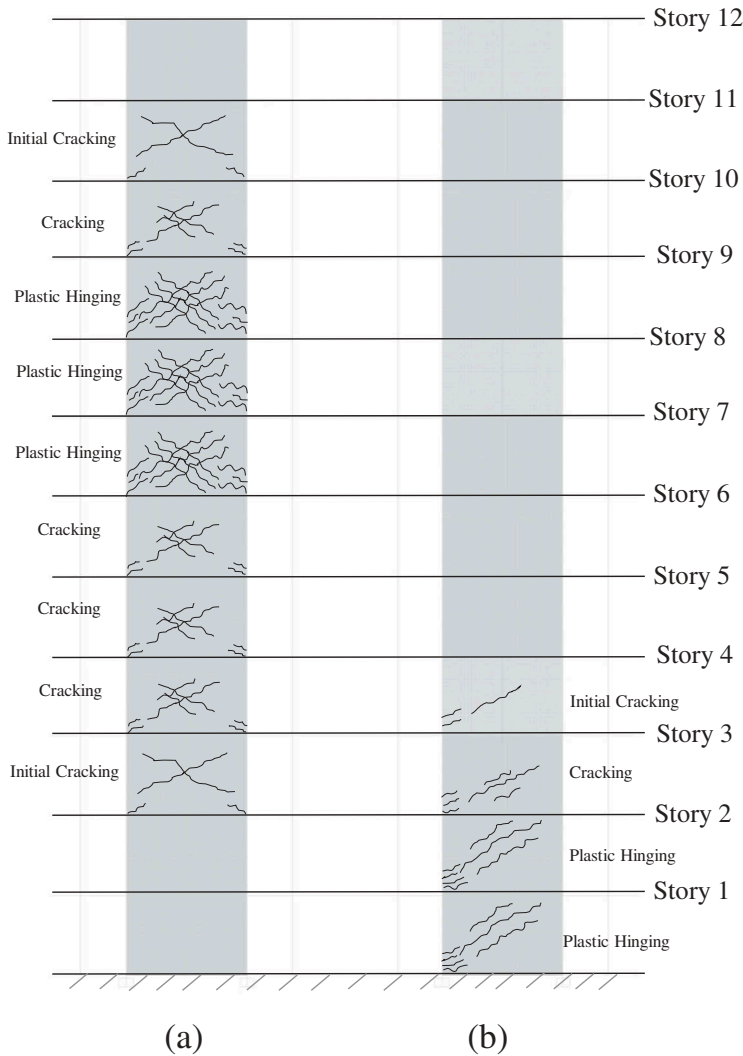


Figure 10. Building 12S-S shear walls final damage state: (a) dynamic analysis results and (b) static pushover analysis results.

hand, the final damage state resulting from the static analysis shows plastic hinging at the first two stories as presented in Fig. 10b. This is mainly attributed to the presence of higher mode effects and ability of dynamic analysis in capturing such effects. The modal analysis results of building 12S-S are presented in Table 4. It can be seen that 7th and 12th vibration modes have relatively high modal participation factors and relative modal weights. This highlights the contribution of higher modes in the structural response which can be captured through dynamic analysis.

3.3. Fragility Analysis

Fragility curves provide the probability of exceeding a specific damage state or performance level over a range of chosen IMs (i.e., a seismic response parameter such as PGA, PGV, or spectral accelerations). They are also essential tools needed for effectively estimating and mitigating potential earthquake losses [Mwafy, 2012]. Seismic fragilities are typically defined based on global ground motion or local response parameters. The global performance limit states are usually classified based on exceeding a specific inter-story drift ratio [Jeong *et al.*, 2012]. In this study, the IM is chosen to be the 5% damped first mode spectral acceleration $S_a(T_1, 5\%)$. Furthermore, the study adopts the performance levels specified by ASCE41 [2013], which are immediate occupancy (IO), life safety (LS), and collapse prevention (CP). The fragility functions are derived using IDA results corresponding to the three damage states. Analytical fragility curves are obtained through the cumulative distribution function shown in Eq. (3) assuming that the seismic response can be essentially represented by a log-normal probability distribution. Such an assumption is widely accepted and considered reliable by many researchers [e.g., Ibarra and Krawinkler, 2005; Jeong *et al.*, 2012; Baker, 2015; and Siyam *et al.*, 2016].

$$P(C|IM = x) = \varphi \left(\frac{\ln(\frac{x}{\theta})}{\beta} \right) \quad (3)$$

$P(C|IM = x)$ is the conditional probability that the structure would reach or exceed a particular limit state at an IM equal to x . The fragility function is described using median of IMs (θ) and IMs' standard deviation or dispersion (β). In this study, the fragility parameters are approximated using the maximum likelihood statistical approach as proposed in Baker [2015]. According to maximum likelihood method, fragility parameters

Table 4. Building 12S-S modal analysis results.

Mode	Frequency (Hz)	Period (s)	Modal participation factor	Relative modal weight (%)
1	0.25	3.93	6.45	56.71
2	1.65	0.60	3.35	15.27
3	4.68	0.21	-1.82	4.53
4	8.85	0.11	1.16	1.85
5	13.82	0.07	0.63	0.53
6	19.02	0.05	0.20	0.06
7	22.24	0.04	-2.63	9.46
8	25.02	0.04	-1.28	2.24
9	30.13	0.03	-0.47	0.31
10	34.89	0.03	-0.22	0.07
11	38.47	0.03	-0.14	0.03
12	1093.66	0.00	2.56	8.96

are calibrated to produce a fragility distribution that would most likely reproduce the observed data. This method is selected for estimating the fragility functions since it provides better representation of the phenomenon and works well with truncated IDA and multiple stripe analysis [Baker, 2015]. The IDA results are used to calculate the probability of exceeding the limit states (i.e., IO, LS, and CP) at each IM in the used ground motions' suite. Then, the fragility parameters are estimated using binominal distribution and assuming the independence of limit states exceedance probabilities from the different IMs. The median (θ) and dispersion (β) are evaluated by maximizing the likelihood expression given in Eq. (4).

$$likelihood = \left[\prod_{i=1}^m \varphi\left(\frac{\ln(IM_i/\theta)}{\beta}\right) \right] \left[1 - \varphi\left(\frac{\ln(IM_i/\theta)}{\beta}\right) \right]^{n-m} \quad (4)$$

where n is the total number of used ground motions, m is the number of ground motions exceeding the limit state at the particular IM, and Π is the product of i values from 1 to m . The parameters (i.e., θ and β) are varied in Eq. (4) to get the maximum likelihood value. It is both mathematically and numerically more efficient to maximize the logarithm of the likelihood function as given in the following equation:

$$\{\theta, \beta\} = \arg \max_{\theta, \beta} \sum_{j=1}^m \left[\ln \varphi\left(\frac{\ln(IM_i/\theta)}{\beta}\right) \right] + (n-m) \ln \left[1 - \varphi\left(\frac{\ln(IM_i/\theta)}{\beta}\right) \right] \quad (5)$$

This maximization process was performed using a spreadsheet to estimate the fragility parameters at the three performance levels (IO, LS, and CP).

3.3.1. Uncertainty Modeling

The fitted fragility curves are adjusted to account for the uncertainties in input ground motion records (β_{RTR}), design criteria (β_{DR}), test data (β_{TD}), nonlinear modeling (β_{MDL}), and records spectral shape. The adjustment is based on FEMA P695 recommendations by calculating the total system uncertainty (β_{TOT}) using the following equation:

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2} \quad (6)$$

Since the far-field ground motion set is selected in performing the IDA, the uncertainty factor β_{RTR} is given a value of 0.4. As the reference buildings are designed based on IBC'12 standards, the design criteria are rated with good quality and the uncertainty parameter (β_{DR}) is equal to 0.2. This is obtained from Tables 1 to 3 in FEMA P695 based on high confidence in basis of design requirements and medium completeness and robustness. The reference buildings' lateral force resisting system consists of RC shear walls which have well-established and well-documented testing data in the literature. Therefore, quality of test data is rated as good and the uncertainty parameter β_{TD} is given a value of 0.2 from Tables 2 and 3 in FEMA P695. This quality rating corresponds to high confidence in test results and medium robustness and completeness. The quality rating of index archetype models is considered good and the parameter β_{MDL} is equal to 0.2. This is based on medium representation of collapse characteristics and high accuracy and robustness of models. The archetypes nonlinear models are developed on IDARC-2D which simulates and captures a wide range of structural collapse modes, but it is limited to two-

dimensional analysis. Using Eq. (6), the calculated total system uncertainty (β_{TOT}) for the reference buildings is equal to 0.525. This value along with the spectral shape factor, obtained from FEMA P695 Tables 7-1a and 7-1b, is used to adjust the fragility curves and account for the uncertainties.

Figure 11 depicts observed, log-normal fitted and adjusted collapse fragility curves for buildings with special RC shear walls (i.e., 6S-S, 9S-S, and 12S-S). It is very clear that the collapse fragility curves are very steep for the three buildings. This reflects a sharp increase in probability of collapse with any small increase in spectral accelerations for the three buildings. The adjusted median collapse intensity (S_{CT}) is 0.77, 0.60, and 0.52 g for buildings 6S-S, 9S-S, and 12S-S, respectively. Therefore, ACMR is 0.63 for 6S-S, 0.66 for 9S-S, and 0.71 for 12S-S. This reflects no safety margin against collapse for buildings designed with special RC shear walls. Collapse fragility curves for buildings 6S-O, 9S-O, and 12S-O with ordinary RC shear walls are given in Fig. 12. These curves are very sharp, showing high collapse probabilities at low spectral accelerations for the three buildings. The adjusted median collapse intensity (S_{CT}) is 0.60 g for 6S-O, 0.37 g for 9S-O, and 0.36 g for 12S-O. Furthermore, the ACMRs are 0.49, 0.41, and 0.50 for buildings 6S-O, 9S-O, and 12S-O, respectively. Similarly, this reflects no collapse margin of safety for designs with ordinary RC shear walls. However, it is observed that the use of the special walls increased the ACMR by 29% for 6-story buildings, 61% for 9-story buildings, and 42% for 12-story buildings. It is noteworthy to mention that the 6-story buildings have slight flatness in its

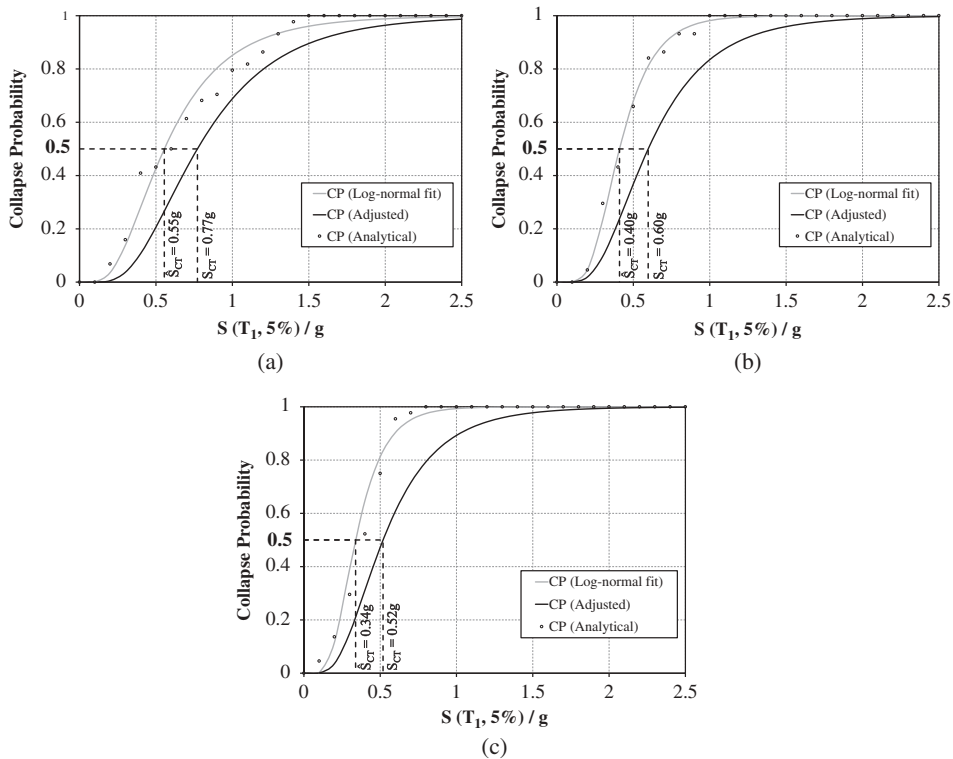


Figure 11. Collapse fragility curves: (a) 6S-S; (b) 9S-S; and (c) 12S-S.

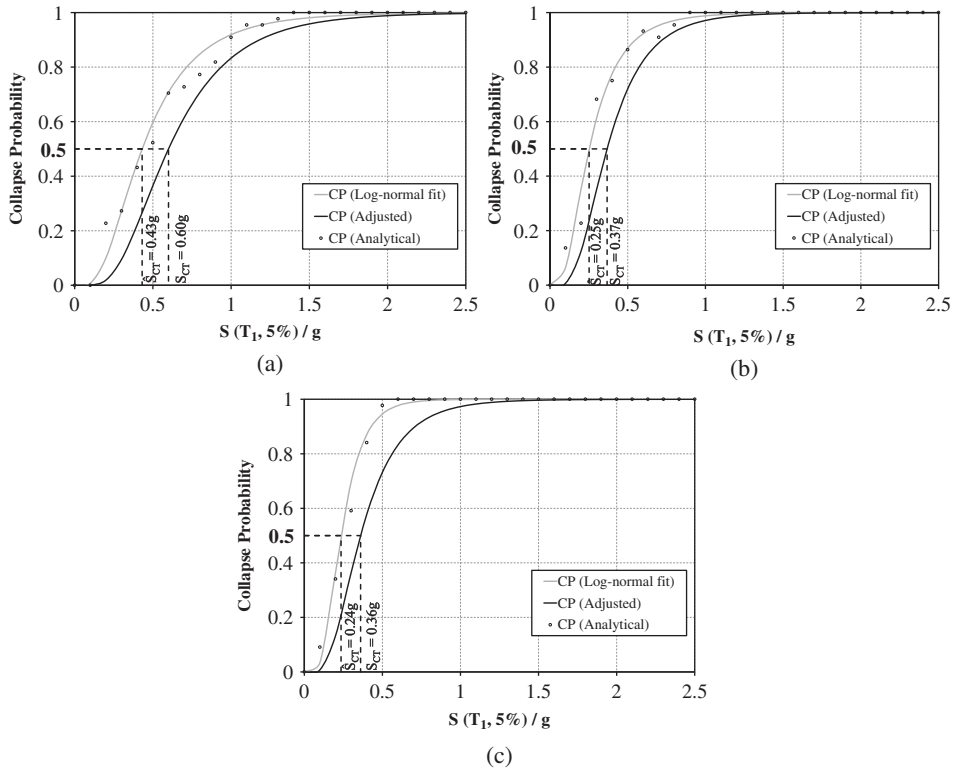


Figure 12. Collapse fragility curves: (a) 6S-O; (b) 9S-O; and (c) 12S-O.

collapse fragility curves relative to 9- and 12-story buildings. It is worth mentioning that careful inspection of the analytical fragility estimates, derived from the IDA (the unconnected circular markers), occasionally displays some seemingly anomalies. For instance, in some regions, analytical probability of exceeding collapse decreases with increasing the IM. This phenomenon is known as structural resurrection. It indicates an intermediate collapse region in the IDA, beyond which the structure appears to resurrect and exhibit decreasing response to the continually increasing magnitude of the IM. The structure will eventually exhibit flat line response at its ultimate dynamic capacity (complete collapse). For further details about this phenomenon, the reader is referred to the original work of Vamvatsikos and Cornell [2002] on IDA.

The three performance levels (i.e., IO, LS, and CP) fragility curves for the six reference buildings are shown in Fig. 13. As expected, the buildings designed with special RC shear walls as their seismic force resisting system show a relatively enhanced structural performance. This is reflected by the flatter fragility curves at the three performance levels (i.e., IO, LS, and CP). It represents lower probabilities of exceeding IO, LS, and CP at similar spectral accelerations. This conclusion is valid for the considered 6-, 9-, and 12-story buildings. At the MCE level (S_{MT}), the choice of special over ordinary RC shear walls does not result in significant reduction in probability of exceeding IO performance level. However, the impact is more evident at LS and CP performance levels. For instance, 6S-S has 80% probability of exceeding CP, while 6S-O has 91% probability of exceeding CP. Similarly, for 9-story buildings, 9S-S and 9S-O have 78% and 95% probability of exceeding

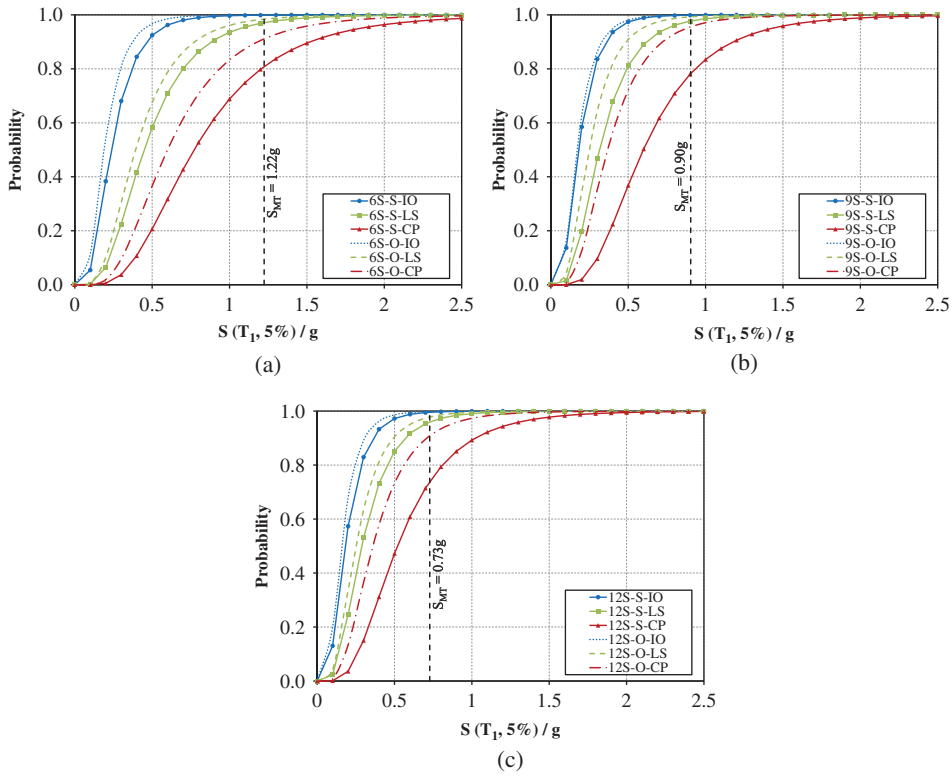


Figure 13. IO, LS, and CP fragility curves: (a) 6-story buildings; (b) 9-story buildings; and (c) 12-story buildings.

CP, respectively. The same observation is applicable to 12-story buildings. 12S-S and 12S-O have 73% and 91% CP exceedance probabilities, respectively. In general, it is clear that the six reference buildings designed for the low seismicity estimate exhibit poor seismic performance which does not satisfy FEMA P695 performance objectives. The poor seismic performance is seen in the high collapse probabilities at MCE level. Similar observations were reported by Mwafy [2012] confirming the seismic vulnerability of buildings, ranging from 10 to 60 stories, located in areas with potential high seismic hazard in UAE to strong earthquake events originating from distant sources. Moreover, it is worth mentioning that the choice of special RC shear walls resulted in a relative enhancement in performance and reduction in collapse probabilities as well as seismic vulnerability.

4. Detailing Impact on Performance and Cost

To quantify the effects of seismic detailing, it is crucial to assess the implications on construction cost. Initial investment is definitely affected by the selected seismic detailing level. However, it is not the only factor that should affect the choice of the seismic detailing level and the seismic force resisting system type. Repair and downtime costs associated with earthquake damages are important factors that shall be considered and may have a significant impact on the decision. For that purpose, a comprehensive cost–benefit analysis is performed considering construction and repair costs of the reference buildings for appropriate

Table 5. Summary of construction cost.

Ref. building	Construction cost		
	Structural cost	Nonstructural cost	Total construction cost
6S-S	\$313,804	\$364,320	\$678,124
6S-O	\$317,698	\$364,320	\$682,018
9S-S	\$511,760	\$546,480	\$1058,240
9S-O	\$505,710	\$546,480	\$1052,190
12S-S	\$661,384	\$728,640	\$1390,024
12S-O	\$655,101	\$728,640	\$1383,741

quantification of the impacts. The initial construction cost of the archetype buildings is shown in Table 5. It is noteworthy that the shown cost is only considering the buildings' structural and nonstructural costs. This includes the RC structure plus its façade; no infill walls are used for the office occupation. Instead, light movable partitions are assumed. The nonstructural cost is limited herein to drift-sensitive elements only, and nonstructural damage to acceleration-sensitive elements, such as equipment, etc., is not within the scope of this investigation. The initial construction cost does not account for substructure cost, contractor's profit, and other indirect costs. This is intended in order to focus on the objective of the study, which is the quantification of the direct impact of seismicity and lateral force resisting system detailing on construction and repair costs. From Table 5, it is observed that there is a marginal impact on total construction cost corresponding to buildings with special or ordinary RC shear walls. For the 6-story buildings (6S-S and 6S-O), the total construction cost is actually slightly lesser (0.6%) for the special RC shear walls building. However, in case of 9- or 12-story buildings, there is a negligible increase (0.5% and 0.6%) in total construction cost associated with the choice of the special shear walls. The construction cost is slightly lower for the 6-story building with special shear walls (6S-S) since the design of the shorter buildings is controlled mostly by strength requirements. Therefore, the ordinary walls being designed for higher forces (smaller response modification factor, R) will require more reinforcement compared to the special walls. As a results, this causes a minor increase in initial construction cost. Overall, it is evident from Table 5 that choosing RC shear walls with special seismic detailing did not result in substantial increase in initial investment (i.e., construction cost).

Repair cost associated with potential damages due to earthquakes is calculated for the six reference buildings. The calculation is based on the economic losses methodology of HAZUS [FEMA, 2010]. This method approximates repair cost based on initial cost of structural and nonstructural components, probability of exceeding performance level at MCE, and the corresponding damage percentage. In this study, the exceedance probabilities used are corresponding to the MCE level of the assumed high seismicity (i.e., $S_s = 1.65$ g and $S_1 = 0.65$ g) to simulate a large unexpected seismic event. This is selected to achieve the objectives of this research by comparing the performance and total cost of buildings designed for low seismicity under a potential high seismic hazard. The structural and nonstructural damage percentages are adopted from SEAOC bluebook [Structural Engineers Association of California, 1999]. SEAOC bluebook specifies five structural and five nonstructural performance levels. In this study, the three most utilized performance levels (IO, LS, and CP) were used in assessing the seismic vulnerability. The selected performance levels, their equivalence in the current study, and their associated damage percentages are outlined in Table 6. The nonstructural damage ratios were assigned, based on engineering judgment and experience, the lower bound of the SEAOC bluebook

Table 6. SEAOC bluebook [1999] structural and nonstructural damage percentages.

Structural/Nonstructural performance level	Qualitative description	Equivalent performance level	Structural damage ratio (%)	Nonstructural damage ratio (%)
SP1/NP1	Operational	–	0	0–10
SP2/NP2*	Occupiable	IO	30	5–40
SP3/NP3*	Life safe	LS	60	20–50
SP4/NP4*	Near collapse	CP	80	40–80
SP5/NP2	Collapsed	–	100	>70

*The performance levels used in the present study.

ranges. This decision is justifiable given the fact that the archetype buildings have RC shear walls as the lateral force resisting system which are typically more rigid compared to other structural systems. According to HAZUS [Federal Emergency Management Agency, 2010] economic loss estimation methodology, the structural/nonstructural repair cost is quantified by multiplying the initial construction cost by the damage percentage of the performance level or damage state as well as the probability of exceeding that particular performance level. Then, the total repair cost is calculated by summing the repair costs resulting at the individual performance levels (i.e., IO, LS, and CP). The probability of exceeding any performance level is obtained from the fragility curves at the MCE level. Initial cost of structural components is evaluated by quantifying materials' quantities and assigning average market unit prices. The cost of nonstructural components is estimated as a fixed percentage of the total construction cost.

The impact of lateral force resisting system detailing (i.e., special vs. ordinary) on the seismic performance is summarized in Table 7. At CP performance level, which is the expected performance level during large magnitude earthquakes, there is a substantial reduction (12–20%) in probabilities of exceedance associated with the utilization of special RC shear walls. This conclusion is consistent in 6-, 9-, and 12-story buildings. However, the special detailing does not show any clear enhancement in seismic performance at IO and LS, while the enhanced performance is evident at CP. This is expected as the special detailing does not necessarily affect the stiffness or strength significantly but rather affects the ductility and overall toughness of the wall. This prevents some of the ground motion records from causing collapse and thus it reduces probability of collapse. The effect of seismic force resisting system detailing on total repair cost is illustrated in Table 8. It is observed that, choosing the special walls resulted in 4%, 9%, and 10% reduction in total repair cost for 6-, 9-, and 12-story buildings, respectively. Therefore, there are evident

Table 7. Effect of seismic force resisting system detailing on performance.

	Ordinary shear wall		Special shear wall		
	Performance level	Probability of exceedance at MCE	Performance level	Probability of exceedance at MCE	Effect of detailing (%)
6-Story	IO	1.00	IO	1.00	0
	LS	0.98	LS	0.97	–1
	CP	0.91	CP	0.80	–12
9-Story	IO	1.00	IO	1.00	0
	LS	0.99	LS	0.98	–1
	CP	0.95	CP	0.78	–18
12-Story	IO	1.00	IO	0.99	–1
	LS	0.98	LS	0.96	–2
	CP	0.91	CP	0.73	–20

Table 8. Effect of seismic force resisting system detailing on repair cost.

	Ordinary shear wall	Special shear wall	
	Total repair cost		Effect of detailing (%)
6-Story	\$718,665	\$687,054	−4
9-Story	\$1179,634	\$1078,717	−9
12-Story	\$1503,114	\$1352,366	−10

potential long-term savings associated with designing the buildings using a well-detailed lateral force resisting system (i.e., special RC shear walls). As a result, the marginal increase in initial construction cost of 9-story and 12-story buildings is outweighed by the reduction in estimated repair costs associated with earthquake damages and the enhanced seismic performance. Further substantial economic benefits and savings are to be expected from shorter downtimes and more rapid resumption of normal revenue-generating operations in the well-detailed buildings.

5. Conclusions and Recommendations

This research investigates and quantifies the structural performance of buildings designed for the low seismic hazard if unexpected large earthquake events occur. It also highlights the consequences associated with the level of detailing choice in RC shear walls (i.e., special or ordinary). The scope covers 6-, 9-, and 12-story RC buildings with shear walls as the seismic force resisting system. The buildings are designed and detailed based on the widely used building code, which is IBC'12. Vertical design loads are in accordance with ASCE7-10 recommended values for office buildings. Lateral loads are calculated based on seismic forces obtained from a selected set of ground motion parameters that resulted in a seismic design category B (i.e., $S_s = 0.18$ g and $S_1 = 0.06$ g). This represents a region with low and uncertain seismic hazard. The seismic performance of reference buildings is investigated according to FEMA P695 [2009], which is based on nonlinear static and dynamic analyses. In addition, fragility analysis is performed at the three performance levels of ASCE41-13 to assess the seismic vulnerability of the different designs. Finally, impact of seismic design level and lateral force resisting system detailing on construction and repair costs is quantified.

Based on IDA results, designing the 6-story building (6S-S) with special RC shear walls decreases CP exceedance probability at MCE by 12%. Similarly, the choice of special RC shear walls in 9-story (9S-S) and 12-story (12S-S) buildings results in 18% and 20% reduction in probabilities of exceeding CP limit state at MCE. In addition, the use of special instead of ordinary RC shear walls resulted in 29%, 61%, and 42% increase in ACMR for 6-, 9-, and 12-story buildings, respectively. This upgrade in seismic performance at MCE is combined with marginal increase in initial investment for the 9- and 12-story buildings. The initial construction cost is slightly reduced for the 6-story buildings with special RC shear walls. However, choosing the special RC shear walls over their ordinary counterparts results in clear reduction in total repair cost of the reference buildings. Therefore, when the buildings are designed for a low seismicity estimate, the choice of special RC shear walls over ordinary RC shear walls would clearly enhance the buildings' seismic performance during major earthquake events. This enhancement is combined with marginal impact on initial construction cost and in most cases with reduction in overall cost (considering repair and downtime

costs). However, it should be noted that designing the special RC shear walls for low seismicity does not fully utilize its expected high seismic performance. This is seen in the low ACMR for the reference buildings with special RC shear walls. The low ACMR reflects the absence of margin of safety against collapse at MCE which is expected if hypothetical large earthquakes occur. From IDA, most of collapse incidents are below MCE for the six archetype buildings. In addition, fragility curves for the different designs had a steep slope reflecting high probabilities of exceeding the three performance levels (i.e., IO, LS, and CP).

Based on the outcomes of this research, it is found to be more appropriate to adopt special detailing over the ordinary detailing in RC shear wall buildings located in regions with highly uncertain seismicity. This is recommended even when the use of ordinary RC shear walls is permitted by the design code. This recommendation is supported by both performance and cost estimates. The special RC shear walls showed significantly improved structural response compared to their ordinary counterpart with negligible impact on initial investment cost. In fact, in some cases, the overall (initial and total repair) cost was slightly lesser for the designs utilizing special RC shear walls. This reflects the substantial potential savings when repair and downtime costs are additionally considered.

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