

TECHNICAL PAPER

Seismic response and life-cycle cost of reinforced concrete special structural wall buildings in Dubai, UAE

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The primary objective of this paper is to quantify the effect of Dubai's diverse seismicity estimates on the seismic response of reinforced concrete shear wall buildings. In this regard, three 12-story office buildings are designed and detailed in accordance with common practices and building codes utilized in Dubai. The buildings are designed for three possible seismic hazard estimates of Dubai (i.e., high, moderate, and low) with specially detailed shear walls. The seismic response is evaluated based on FEMA P695 methodology and fragility analysis. The methodology is based on nonlinear pseudo-static pushover and incremental dynamic analyses (IDA). Pushover analysis is performed and its results are compared against design levels. IDA is performed using the set of 22 far-field ground motion records recommended by FEMA P695. The results of this research confirm that designing for the conservative seismicity estimate significantly enhances the seismic response of the buildings. The building designed for the highest seismicity showed a favorable structural response, while the moderate seismicity design had a reasonable response. For the lowest seismicity design, results showed a poor seismic performance. Furthermore, the resulting enhancements in the seismic response reduce potential earthquake damages and collapse probabilities. Therefore, the marginal increase in initial investment is outweighed by the reduction in the buildings' life-cycle (repair and downtime) cost.

KEYWORDS

Dubai seismicity, fragility analysis, seismic performance, seismic vulnerability, structural walls

1 | INTRODUCTION

Over the past few decades, there has been significant economic development in the United Arab Emirates (UAE) in general and specifically in Dubai. Substantial investments are taking place in the real-estate sector in Dubai, especially after announcing Dubai to be the hosting city of EXPO 2020.¹ As a result of the announcement, several skyscrapers and iconic structures are being planned to be built to serve as touristic attractions and to accommodate the growing

population. During the structural design phase, it is well established that majority of the challenges stem from the uncertainty in the assumed design loadings that should be resisted by the structure during its service life. Among other loads, dynamic loadings such as seismic forces have the highest degree of uncertainty. This is because, in most cases, the seismic hazard estimation is probabilistic in nature, except for zones that are close to fault lines. The UAE and specifically Dubai suffer from considerable uncertainty in its seismicity level and the seismic design procedures that should be adopted.

Dubai seismic hazard has been the subject of many published research studies since the early 1990s. Additionally,

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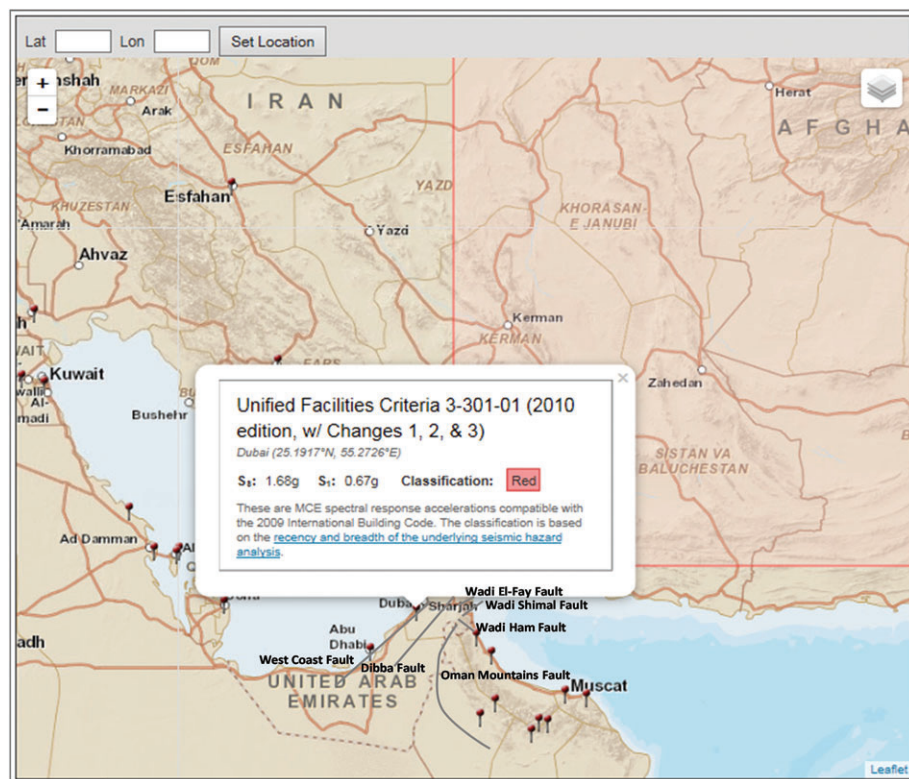


FIGURE 1 Mapped spectral accelerations for Dubai (USGS, retrieved May 31, 2015, from <http://geohazards.usgs.gov/designmaps/ww/>) and local faults

the number of earthquakes occurring in or close to UAE has been increasing since 1924–2006.² Seismicity of the UAE is generally attributed to earthquakes originating from seismic sources that are local and distant. Shama³ considered the seismic source zones affecting Dubai seismicity are Zagros fold-thrust region, Markan subduction zone, and Zendan-Minab-Palami fault system. In addition, the author suggested an influence from several local crustal faults in the UAE, such as Dibba, Wadi El Fay, Wadi Ham, Wadi-Shimal, Omanmountains, and West Coast fault are shown in Figure 1.

Although major seismic activities are rare in the UAE, several studies have been conducted in an attempt to provide a clear picture regarding the seismic hazards in Dubai.^{4–11} The available seismic hazard studies show significant diversity in estimating the seismicity hazard of Dubai. Furthermore, there are at least 10 seismic hazard assessment studies for UAE and Dubai, for instance References 3 and 6–16. However, unfortunately, there is not much consensus

in these researches about the seismicity level that should be designed for in the UAE. Table 1 shows a summary of the estimated peak ground acceleration (PGA) for Dubai from several seismic hazard assessment studies, corresponding to 10% probability of exceedance in 50 years (i.e., 475-year return period). The PGA estimates show significant diversity ranging from very low seismicity that does not require any seismic design to very high seismicity levels that mandate particular attention to the level of seismic design and detailing. The diverse seismic hazard estimates can be attributed to the lack of in-depth seismological data and historical recordings of ground motions in this region that should be used in providing comprehensive seismic design guidelines.⁷ Furthermore, the local authorities of Dubai have minimum seismic design requirements based on the UBC'97.¹⁷ These requirements were raised in 2013 following the tremors that were felt and recorded in the UAE to zone 2B for buildings higher than nine stories and 2A for buildings with four to nine stories. The new requirements were communicated to engineering consulting offices in Dubai through a memorandum sent by Dubai Municipality. One of the main consequences of the seismic hazard level on the design is the choice of the lateral force resisting system type, its detailing requirements, and the implications on construction cost.⁷

2 | PROBLEM DESCRIPTION

Due to the lack of consensus among the seismic hazard studies for Dubai, it is a quite challenging task for structural designers to pick the appropriate design level had they been

TABLE 1 PGA estimates for Dubai from several studies

Study by	Year	PGA (475-year return period) (g)
Al-Haddad et al ¹²	1994	<0.05
Grunthal et al ¹³	1999	0.32
Sigbjornsson and Elnashai ¹⁴	2006	0.16
Pascucci et al ¹⁵	2008	0.06
Aldama-Bustos et al ¹⁶	2009	<0.05
Shama ³	2011	0.17

PGA, peak ground acceleration.

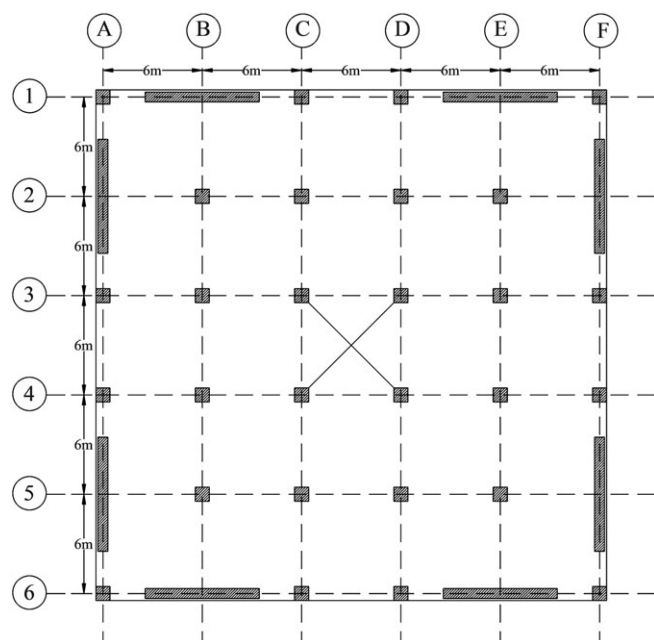


FIGURE 2 Typical floor plan view

given the choice. Additionally, the update in the seismic design requirements by the municipality increases the importance of investigating the impact of the seismic design level on the performance and construction cost of buildings. It is always very crucial to propose designs for structures that are not only economical but also safe in the short and long runs. The objective of this research is to quantify the impact of the diverse estimated seismic levels in Dubai on the performance and life-cycle cost of RC special shear wall buildings. The life-cycle cost assessment includes the initial investment, repair cost, and downtime cost. It is based on an assumed design life duration of 50 years, which is the expected service life of buildings according to IBC'12¹⁸ and ASCE7-10,¹⁹ the minimum design loads standard.

The buildings under consideration are office buildings located in Dubai made up of reinforced concrete (RC). The lateral system consists of special RC shear walls. RC shear walls are popular lateral force resisting elements with considerably high lateral resistance and tendency to fail under shear forces.²⁰ Moreover, the gravity system consists of cast-in-situ flat slab with square columns. The typical floor heights are 4 m, and the spacing between columns is 6 m, Figure 2 shows a typical floor plan view. The concrete compressive strength is 38 MPa, and the reinforcement yield strength is 420 MPa. Furthermore, to enhance the buildings' torsional resistance, the shear walls are placed along the periphery of the buildings. The model buildings are

designed for the three possible seismic hazard estimates in Dubai. This is to achieve the objective of assessing the impact of the seismic hazard level on the structural performance and life-cycle cost of RC structural wall buildings in Dubai, given the available uncertainty in seismicity level. The highest seismicity design level represents the upper bound, and it is obtained from USGS²¹ estimate for Dubai as shown in Figure 1. Moreover, the moderate seismic design level represents ADIBC'11²² seismic hazard estimate for Dubai. Furthermore, the lowest seismicity level corresponds to the lower bound given by Aldama-Bustos et al¹⁶ seismic hazard assessment study. Hence, to realistically assess the impacts of the design level on overall performance, the buildings are all subjected to far-field ground motions that are scaled to match the maximum considered earthquake (MCE) response spectrum of the highest seismicity estimate in the UAE. This seismicity hazard level is as estimated by USGS for Dubai ($S_s = 1.65$ g and $S_1 = 0.65$ g). As such, this would highlight the implications of seismic loading level on design, seismic performance, and life-cycle cost of RC shear wall buildings in Dubai, if the high seismicity estimate materializes. This testing level represents the worst, but a possible, seismic hazard scenario in the UAE from distant sources (e.g., Zagros Thrust or Makran subduction zone) as highlighted by Sigbjornsson and Elnashai.¹⁴ Thus, it allows assessing the consequences of the selected seismic design level on the overall response of RC shear wall buildings.

The three buildings are analyzed and designed to limit their inter-story drift within the building code (i.e., IBC'12¹⁸) specified limits using ETABS²³ and to ensure sufficient lateral resistance. The buildings' seismic design parameters (i.e., ground motion input parameters and time periods) are shown in Table 2. Building B1 is designed for the highest seismicity, B2 for the moderate seismicity, and B3 for the lowest seismicity. Furthermore, the buildings' seismic performance is investigated using fiber-based finite element numerical model with concentrated plasticity incorporating material and geometric nonlinearities. The hysteretic behavior of special RC shear walls is modeled using the multi-linear hysteretic model in IDARC-2D.²⁴ This modeling technique is widely accepted and has been utilized in many publications in the available literature (e.g., References 5 and 27–29). Additionally, the damping is taken as 5% of critical and is assumed to follow Rayleigh's formulation. The two types of nonlinear analyses that are performed to investigate the buildings' seismic performance are pseudo-static pushover analysis and incremental dynamic analysis (IDA).

TABLE 2 Buildings' seismic design parameters

Building ref.	S_s (g)	S_1 (g)	Natural period (T_b) (s)	Upper limit ($C_u \cdot T_a$) (s)
B1	1.65	0.65	1.8	1.24
B2	0.42	0.17	3.13	1.29
B3	0.18	0.06	3.97	1.5

TABLE 3 Gravity column design summary

Floors	Column type	Column size (mm × mm)	Reinforcement	Ties in <i>X</i> and <i>Y</i>
10–12	Opening	700 × 700	20-T32	3 Legs T12 @350 mm
	Others	700 × 700	16-T32	3 Legs T12 @350 mm
7–9	Opening	750 × 750	8-T32	3 Legs T12 @350 mm
	Others	750 × 750	8-T32	3 Legs T12 @350 mm
4–6	Opening	800 × 800	12-T32	4 Legs T12 @350 mm
	Others	800 × 800	12-T28	4 Legs T12 @350 mm
1–3	Opening	850 × 850	12-T32	4 Legs T12 @350 mm
	Others	850 × 850	12-T28	4 Legs T12 @350 mm

3 | BUILDING'S DESIGN DETAILS

The three buildings are designed and detailed in accordance with IBC'12¹⁸ standards for the three different estimates of Dubai seismicity. The buildings layout and design are representative of the state-of-the-practice in Dubai. The gravity system is common between the three buildings and is designed only to resist vertical loads and to satisfy the deformation compatibility requirements. The gravity columns and flat slabs are designed according to ACI318-11³⁰ and ASCE7-10¹⁹ requirements. Columns cross sections and reinforcement are optimized (grouped) every three floors; the design details are summarized in Table 3. Furthermore, the flat plate is 200 mm thick with 16 mm diameter reinforcement bars at 150 mm center-to-center spacing in both directions. Additionally, 12–20 mm diameter bars spaced at 125 mm are required at the top of each column in both directions.

The lateral system of each building consists of eight special RC shear walls (two per building side). Moreover, the walls are placed along the building perimeter to increase their effectiveness in resisting applied lateral loads, especially torsional effects. The RC shear walls are designed and detailed as per ACI318-11³⁰ and ASCE7-10¹⁹ standards using quick concrete wall program.³¹ They have rectangular cross sections with embedded boundary elements having the same wall thickness. The boundary element's vertical extent is based on the displacement procedure requirements. However, they are at least extended through the first three floors. Initially, the thickness and reinforcement of the walls were optimized every three floors, while keeping the wall in-plan length constant throughout the building height; this is to avoid any in-plane vertical irregularities in the lateral load resisting system. Nevertheless, during the initial seismic performance investigations using IDA, it was observed that the higher mode effects governed the structure collapse mechanism and the critical sections were not occurring at the base as suggested by the design code. The critical sections resulting in the dynamic analysis were consistently found to be shifted upwards to the bottom of 10th floor (i.e., the lowest story in the upper quarter of the building). Therefore, to avoid this unfavorable premature collapse mechanism and enhance the seismic performance of the

structure it was decided to use the same wall section for the upper half of the reference buildings. Table 4 shows the design details of the shear walls for the three buildings.

4 | RESULTS AND DISCUSSION

In this section, the performance investigations of the three model buildings are presented. The buildings' structural performance and collapse probabilities are investigated using nonlinear pushover analysis and IDA. Pushover analysis is used to validate the nonlinear modeling, evaluate the buildings' strength, and estimate the period-dependent ductility. Furthermore, IDA is performed to investigate the seismic performance and collapse probability of the model buildings. In addition, IDA results are used to construct fragility curves for three performance levels, namely immediate occupancy (IO), life safety (LS), and collapse prevention (CP). Finally, the construction and repair costs of the three buildings are estimated and compared to assess the seismic design level impact on life-cycle cost. The life-cycle cost calculation is based on the building code (i.e., IBC'12¹⁸) expected service life of 50 years and the seismic hazard that corresponds to 2% probability of exceedance in 50 years. This seismic hazard level represents the high estimate for the MCE in Dubai, UAE.

4.1 | Nonlinear static pushover analysis results

The seismic performance of the reference buildings is initially investigated using displacement-control pushover analysis. The pushover analysis is performed using IDARC-two-dimensional (2D) program with linear displacement profiles representing the fundamental vibration mode. The resulting monotonic backbone curves obtained for the building designed for high seismicity (B1), moderate seismicity (B2), and low seismicity (B3) are shown in Figure 3. The vertical axis is the base shear capacity normalized by the building seismic weight, and the horizontal axis is the roof drift ratio (%). In the pushover analysis, the applied displacement profile is increased monotonically until all the three buildings reached 20% loss in their base shear strength. It can be observed that B1 reached a maximum of

TABLE 4 Concrete shear wall design summary

Floors	Parameter	B1	B2	B3
7–12	L	7,000 mm	5,200 mm	4,600 mm
	T	500 mm	350 mm	300 mm
	L_{BE}		–	–
	R_{BE}	–	–	–
	$C_{BE,w}$	2T16 @ 100 mm	2T16 @ 100 mm	2T12 @ 100 mm
	$C_{BE,L}$	6T16 @ 100 mm	3T16 @ 100 mm	3T12 @ 100 mm
	R_{HW}	T20 @ 400 mm	T16 @ 450 mm	T16 @ 450 mm
	R_{VW}	T22 @ 200 mm	T22 @ 400 mm	T16 @ 450 mm
	T	550 mm	400 mm	350 mm
	L_{BE}	–	–	–
4–6	R_{BE}	–	–	–
	$C_{BE,w}$	2T16 @ 100 mm	2T16 @ 100 mm	2T12 @ 100 mm
	$C_{BE,L}$	6T16 @ 100 mm	3T16 @ 100 mm	3T12 @ 100 mm
	R_{HW}	T20 @ 300 mm	T20 @ 400 mm	T16 @ 450 mm
	R_{VW}	T22 @ 100 mm	T22 @ 200 mm	T16 @ 400 mm
	T	600 mm	450 mm	400 mm
	L_{BE}	1,200 mm	600 mm	600 mm
	R_{BE}	28-T28	14-T28	5-T20
	$C_{BE,w}$	3T16 @ 100 mm	3T16 @ 100 mm	2T16 @ 100 mm
	$C_{BE,L}$	6T16 @ 100 mm	3T16 @ 100 mm	3T16 @ 100 mm
1–3	R_{HW}	T20 @ 250 mm	T20 @ 350 mm	T20 @ 350 mm
	R_{VW}	T28 @ 100 mm	T22 @ 200 mm	T20 @ 200 mm

$C_{BE,L}$ = boundary element confinement reinforcement parallel to wall; $C_{BE,w}$ = boundary element confinement reinforcement perpendicular to wall; L = wall Length; L_{BE} = boundary element length; R_{BE} = boundary element reinforcement; R_{HW} = shear wall horizontal reinforcement; R_{VW} = shear wall vertical reinforcement; T = wall thickness.

7% roof drift ratio without significant base shear strength deterioration compared to B2 and B3 which reached 3.5% and 4% roof drift ratios, respectively. This indicates a desirable ductile behavior for the three buildings. Furthermore, the maximum base shear strength normalized by the building weight (i.e., base shear coefficient) is 0.46 for B1, 0.24 for B2, and 0.15 for B3. The base shear capacity of B2 is approximately 160% higher than B3. Similarly, B1 base shear capacity is 190% greater than B2 and more than 300% higher than B3. From the pushover analysis results shown in Figure 3, the period-based ductility (μ_T) is estimated for the three model buildings as the ratio between ultimate roof displacement (δ_u) and effective yield roof displacement ($\delta_u/\delta_{y,eff}$). The calculated period-based ductility (μ_T) is more than 8 for B1, B2, and B3, which confirms the ductile response of the special RC shear walls. For easy comparison, Figure 3d shows the backbone curves for the three buildings superimposed in the same plot. It highlights the

significant difference in strength and ductility between the three buildings. Furthermore, as shown in Figure 4, the collapse mechanism at 20% strength loss is governed by plastic hinge formation at first and second floors and initiation of cracking in the third floor for B1. On the other hand, B2 and B3 final damage state correspond to plastic hinging at the first floor only. The shear walls in building B1 were well designed and detailed for higher load level. Therefore, the critical section was able to sustain significant damage allowing the inelastic response to transfer to a new extended zone. Thus, resulting in a better seismic response and ductility. The structural damage indices before collapse are shown in Table 5 for B1, B2, and B3. It is noteworthy that if push-over analysis was carried out until 3.5% roof drift ratio (minimum of B1, B2, and B3), B1 would have the least damage, while B3 would have the highest. Thus, when comparing the seismic performance at the three performance levels specified in ASCE41-13³⁸ (i.e., IO, LS, and CP) and their corresponding drift ratios, B1 will exhibit a relative enhancement in seismic performance and savings in repair costs compared to B2 and B3. Similarly, B2 seismic response will surpass that of B3.

4.2 | Nonlinear IDA results

IDA is performed to examine the seismic behavior of the three buildings under the arbitrary nature of earthquake events. It is carried out using the 22 far-field earthquake records set specified in FEMA P695³² and obtained from Reference³³. For each ground motion record, all the three components, two horizontal and one vertical, are considered in the analysis. The ground motion records are from large magnitudes events ranging from 6.5 to 7.6 and originating from distant sources more than 10 km away from fault rupture. These records are scaled using FEMA P695³² methodology such that at the structure's fundamental period the median spectral acceleration of the record set matches the MCE response spectrum of the highest seismicity estimate of Dubai, UAE. The IDA is performed using IDARC-2D²⁴ program with concentrated plasticity fiber-based numerical model incorporating material and geometric nonlinearities. In the IDA, the intensity of each ground motion record is amplified by increasing the spectral acceleration that corresponds to the fundamental period of the structure by 0.1 g increments until collapse occurs. If the collapse is not reached numerically, the model buildings are assumed to have collapsed when the drift ratio reaches 10%.

As expected, the IDA curves presented in Figure 5 show relatively higher collapse resistance for B1, reasonably moderate collapse resistance for B2 and low collapse resistance for B3. The IDA results are used to evaluate the seismic performance of the model buildings according to the methodology outlined in FEMA P695.³² FEMA P695 methodology has been effectively applied to investigate and quantify the seismic performance of many structural

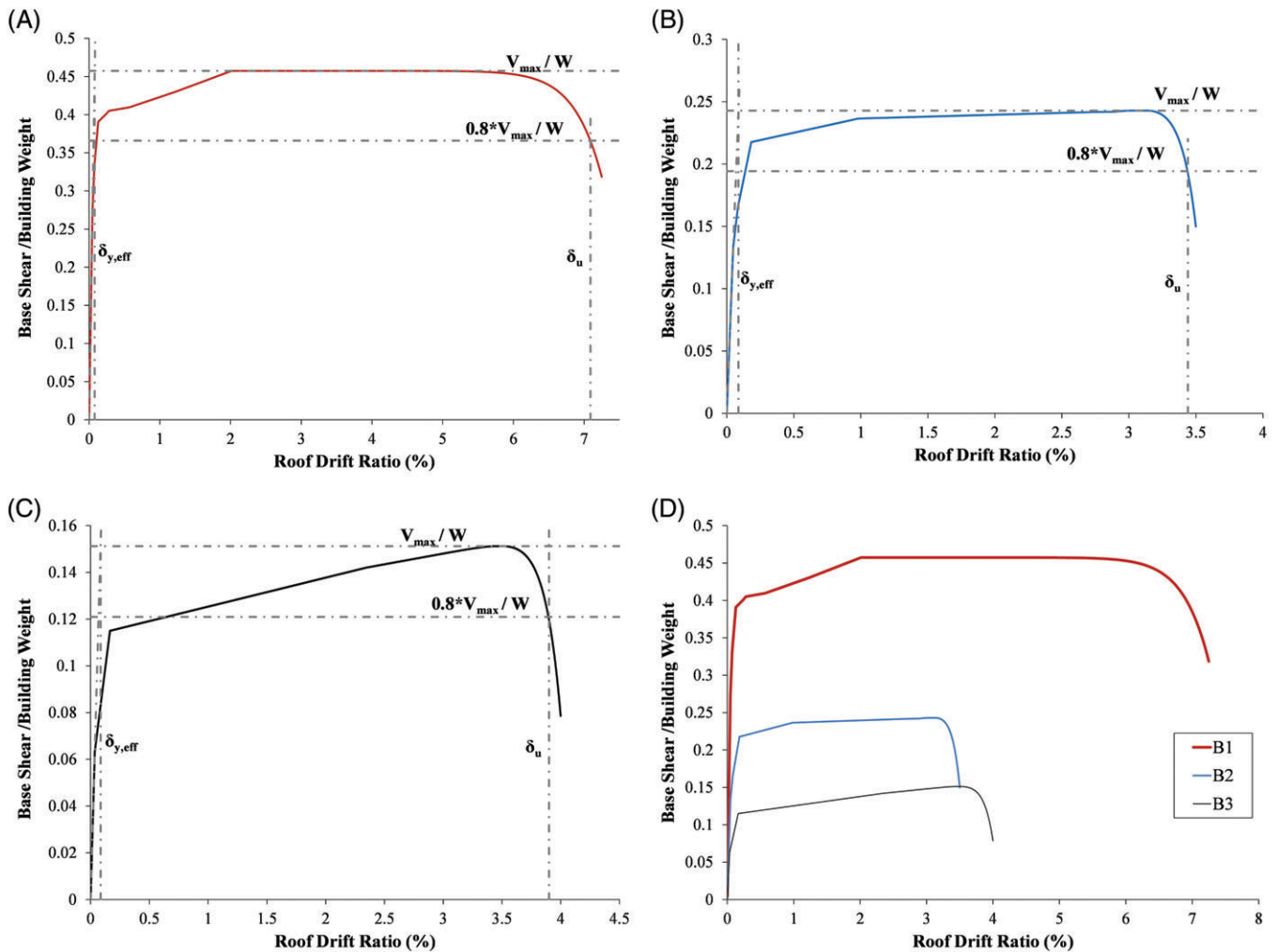


FIGURE 3 Nonlinear static pushover curves: (a) B1; (b) B2; (c) B3; (d) B1, B2, and B3

systems; such as moment resisting frames and shear walls.^{4,34} Moreover, it was applied for evaluating the seismic performance of steel plate shear walls³⁵ and buildings with a central core.³⁶ It was also implemented to assess the seismic safety of RC shear walls.³⁷ FEMA P695 seismic collapse evaluation procedure requires the estimation of the median collapse intensity ($\hat{S}_{CT}$) which is the spectral acceleration at which half of the records (22 components; since 2D analysis is used) cause the structure to collapse. In other words, it is the spectral acceleration corresponding to 50% probability of collapse. Then, the collapse margin ratio (CMR) is calculated as the ratio between $\hat{S}_{CT}$ and the MCE spectral acceleration at the fundamental period of the building (S_{MT}). CMR is the critical parameter that will define the collapse safety and seismic performance of the model buildings. However, the CMR shall be adjusted to account for the different uncertainty sources, such as frequency content (spectral shape) of the used ground motion records and the elongation in the fundamental period of the structure that occurs before collapse. The IDA results are adjusted as per FEMA P695 procedures to consider the different sources of uncertainties such as: (a) record-to-record uncertainty (β_{RTR}); (b) design-requirement-related uncertainty (β_{DR}); (c) test data

uncertainty (β_{TD}); and (d) nonlinear modeling uncertainty (β_{MDL}). The collapse fragility of the buildings is defined using a random variable (S_{CT}) which is calculated using Equation (1).

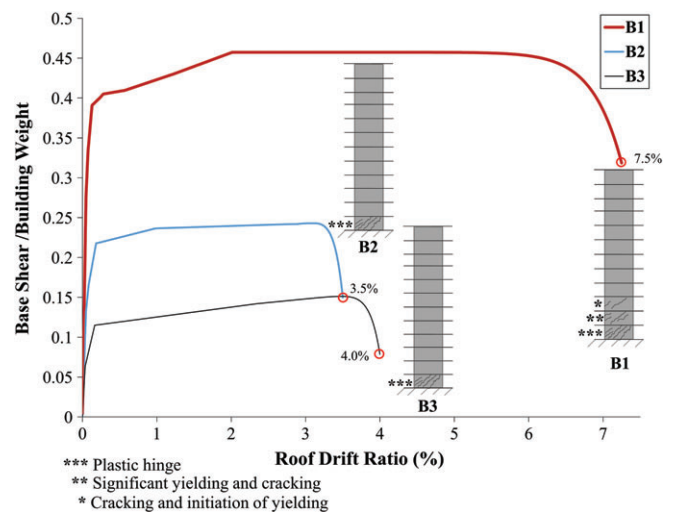


FIGURE 4 Shear walls' damage state at the end of pushover analysis: (a) B1; (b) B2; (c) B3

TABLE 5 Structural damage indices

Floors	B1		B2		B3	
	Col-wall damage	Weighting factor	Col-wall damage	Weighting factor	Col-wall damage	Weighting factor
12	0	0	0	0	0	0
11	0	0	0	0	0	0
10	0	0	0	0	0	0
9	0	0	0	0	0	0
8	0	0	0	0	0	0
7	0	0	0	0	0	0
6	0	0	0	0	0	0
5	0	0	0	0	0	0
4	0.001	0	0	0	0	0
3	0.012	0.007	0	0	0	0
2	0.183	0.162	0	0	0	0
1	0.707	0.832	0.359	1	0.356	1
Overall structural damage		0.618		0.359		0.356

$$S_{CT} = \hat{S}_{CT} \cdot \lambda_{TOT} \quad (1)$$

S_{CT} depends on λ_{TOT} which is a log-normally distributed random variable with a unity median and lognormal standard deviation that is equal to (β_{TOT}) . The parameters λ_{TOT} and β_{TOT} are calculated as shown in Equations (2) and (3), respectively.

$$\lambda_{TOT} = \lambda_{RTR} \cdot \lambda_{DR} \cdot \lambda_{TD} \cdot \lambda_{MDL} \quad (2)$$

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2} \quad (3)$$

The parameters λ_{RTR} , λ_{DR} , λ_{TD} , and λ_{MDL} are also log-normally distributed with lognormal standard deviation β_{RTR} , β_{DR} , β_{TD} , and β_{MDL} , respectively. Furthermore, the uncertainty parameters β_{RTR} , β_{DR} , β_{TD} , and β_{MDL} are obtained from FEMA P695.³² The record-to-record uncertainty (β_{RTR}) is given a fixed value of 0.4 for the far-field record set. From Table 3 in FEMA P695 (2009), a value of 0.2 is chosen for the design-related requirements uncertainty (β_{DR}). This is based on a medium completeness and robustness since the buildings are designed and detailed to resist lateral loads. Additionally, the confidence in “basis of design requirements” is “high” as the buildings are designed according to the IBC’12 requirements. The materials, components, assembly, and system behavior, are well understood and documented for special RC shear walls. Therefore, the test data uncertainty (β_{TD}) is given a value of 0.2 from Table 3 in FEMA P695 corresponding to a “medium” completeness and robustness and “high” confidence in test results. Finally, nonlinear modeling uncertainty (β_{MDL}) is also equal to 0.2, from Table 5 in FEMA P695,³² which corresponds to “medium” representation of collapse characteristics and “high” accuracy and robustness of models. This is because the nonlinear modeling is done using IDARC-2D²⁴ which accounts for plastic hinge and crack formation in all structural elements, but the analysis is limited to 2D. Combining the different uncertainty parameters gives a total system collapse uncertainty (β_{TOT}) of 0.53. Furthermore, the spectral shape factor (SSF) which depends on the

fundamental period of the structure and the period-based ductility obtained from the pushover analysis is used to account for the frequency content of the ground motion records. After adjusting the IDA results to consider the different uncertainty sources, the adjusted collapse margin ratio (ACMR) is calculated by multiplying the CMR by the SSF for each building. The SSF is 1.53 for B1, 1.55 for B2, and 1.61 for B3, as given in Table 7 in FEMA P695 (2009). Figure 6 shows the collapse fragility curves for the three buildings. The adjusted median collapse intensity (S_{CT}) is 1.07 g, 0.69 g, and 0.52 g for B1, B2, and B3, respectively. It is interesting to note that there is a 155% increase in adjusted median collapse intensity (S_{CT}) for B1 compared to B2 and more than 200% increase compared to B3. Similarly, B2 median collapse intensity (S_{CT}) is 33% higher than B3. Furthermore, when the three buildings are subjected to the ground motion records set scaled to match the highest seismicity level, B1 exhibited the least probability of collapse, while B3 had the highest collapse probability. Additionally, the ACMRs were 1.47, 0.95, and 0.71 for B1, B2, and B3, respectively. This indicates that B1 has 47% collapse margin of safety, while B2 is a borderline case and B3 has no identifiable clear collapse margin. Using the adjusted IDA results, the fragility curves are constructed for the three damage states, namely IO, LS, and CP as shown in Figure 7. These damage states are corresponding to particular drift ratios as recommended by ASCE41-13³⁸ for RC structural walls (i.e., 0.5% for IO, 1% for LS, and 2% for CP). From the shown fragility curves, it is clear that B1 exhibits the best response at the three performance levels compared to B2 and B3. This is maintained through the flatter curves which indicate a lower probability of exceeding IO, LS, or CP damage states. Moreover, at the MCE level spectral acceleration, the collapse probability is 23% for B1, 54% for B2, and 73% for B3. Similarly, the probabilities of exceeding LS damage state is 64%, 85%, and 96% for B1, B2, and B3, respectively. It is clearly noted that the building designed for high seismicity

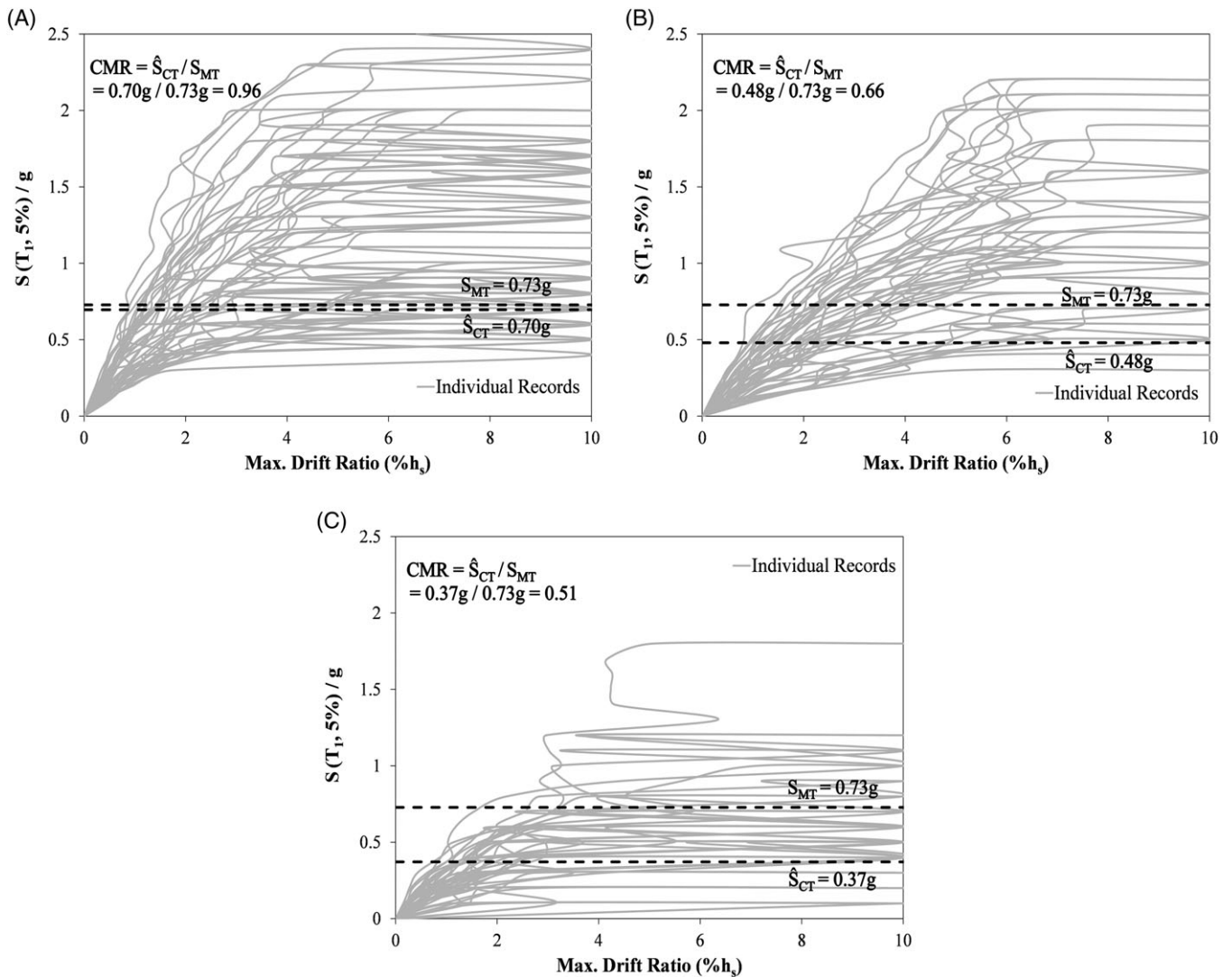


FIGURE 5 Incremental dynamic analysis response plots: (a) B1; (b) B2; (c) B3

(i.e., B1) has the most favorable seismic response and least probability of collapse at MCE.

4.3 | Cost comparison

In quantifying the impact of the diverse seismic hazard estimates, not only the seismic performance matters but also the implications on the construction and life-cycle cost are crucial. The skeleton construction cost of each model building is composed of (a) structural material cost and (b) non-structural cost. For the structural cost, the bill of quantities for the main construction materials (i.e., concrete and reinforcement) and the corresponding costs are summarized for the three buildings in Table 6.

The concrete and steel unit cost has been obtained through communications with major contracting companies in Dubai, UAE. It can be noted that the difference in materials cost is mainly due to the shear walls since the three buildings share the same gravity system. As expected, B1 has the highest volumes of materials and cost compared to the other two buildings (B2 and B3). However, as shown in Table 6, the increase in material cost for B1 is approximately 19% relative

to B2 and 26% relative to B3. Likewise, B2 has material cost that is around 6% higher than B3. The non-structural cost is estimated as a percentage of total construction cost; this was recommended by a major developer for office buildings in Dubai. This percentage was taken as 55% of the average standard construction cost. The average standard construction cost is adopted from the Bruce Shaw Handbook³⁹ as \$1,150/m² of construction floor area. The non-structural cost is considered mainly for the evaluation of the life-cycle cost purposes (i.e., estimation of the non-structural repair cost). It should be noted that the same type of non-structural components was assumed in the three buildings to ensure a meaningful comparison. Thus, it is possible to highlight and quantify the relative impact of the seismic design level on the repair costs associated with the expected earthquake damages in both structural and non-structural components.

The life-cycle (repair) cost due to potential earthquake damages has a significant impact on the net cost of the buildings. A comprehensive assessment of the buildings' construction and life-cycle costs shall be a consideration factor when

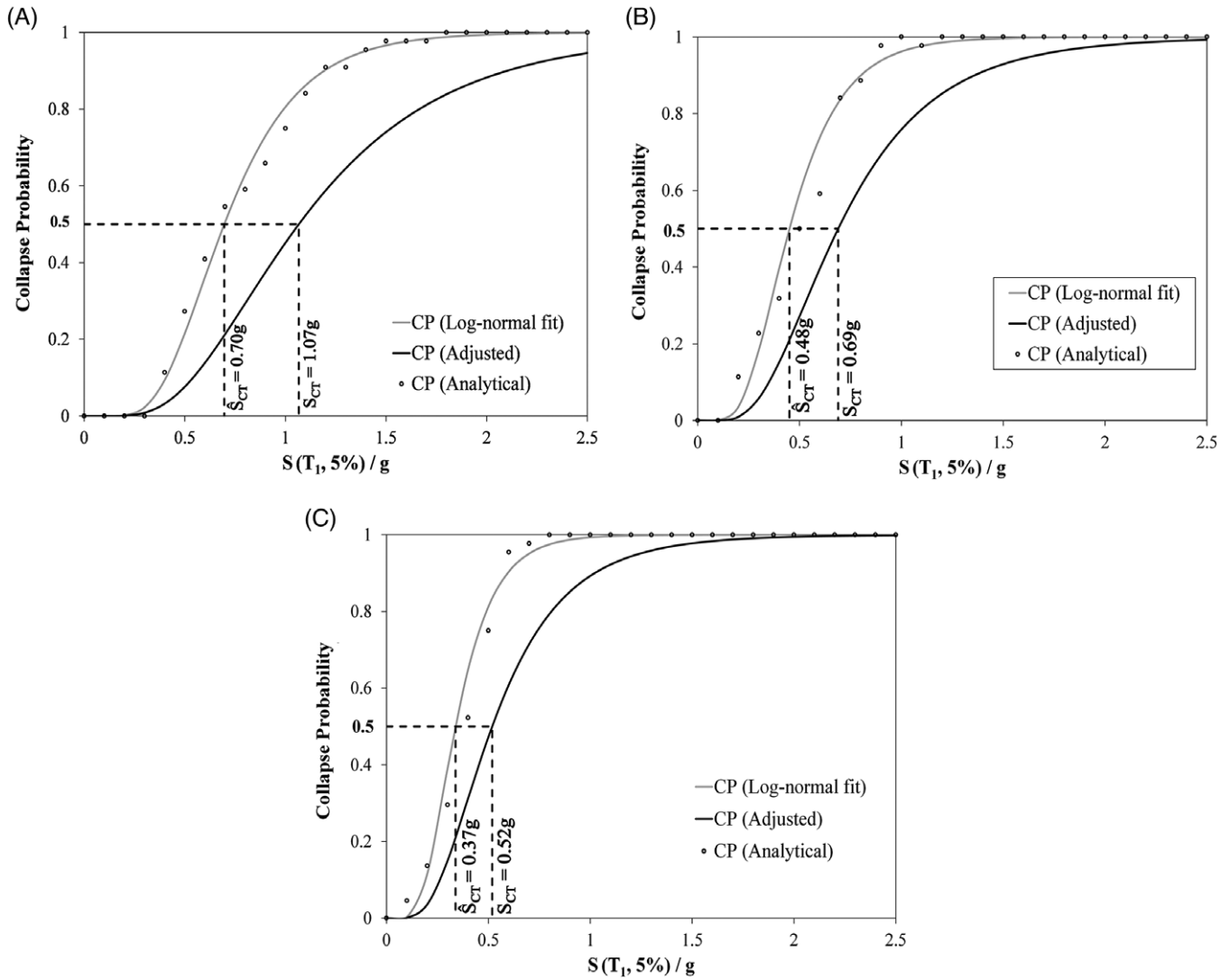


FIGURE 6 Collapse fragility curves: (a) B1; (b) B2; (c) B3

selecting the seismic design level to be adopted for achieving an optimum trade-off between performance and cost. The total life-cycle cost for each building includes the structural and non-structural repair costs that would be expected following the design earthquake event. The repair costs are evaluated according to HAZUS loss estimation methodology⁴⁰ and based on the damage percentages given by SEAOC Blue Book⁴¹ at the different structural performance levels. SEAOC is a well-established standard that is widely used in the earthquake engineering research community (e.g., References 2, 5 and 42–43) for performance-based seismic design and assessment of buildings. Table 7 shows the corresponding structural and non-structural damage percentages at each performance level. The cost of repairs is estimated as follows:

$$\begin{aligned}
 &\text{Structural/non-structural repair cost (\$)} \\
 &= \text{probability of exceeding a limit state at MCE level} \\
 &\times \text{structural/non-structural damage \% for the limit state} \\
 &\times \text{structural/non-structural construction cost (initial investment)}
 \end{aligned}$$

For each building, the repair cost is calculated at the three performance levels (i.e., IO, LS, and CP) and then combined to get the total expected repair cost. The total repair cost is added to the construction cost (initial investment) to estimate the total cost associated with each seismic design level. Construction, total repair, and total costs are shown in Figure 8. For brevity and clarity, Figure 8 does not show a breakdown of the repair cost at the different performance levels (i.e., IO, LS, and CP). As clearly demonstrated, opting to design for the high seismicity level has marginal impact on the construction cost. The total construction cost is only 12.5% and 2.9% higher for B1 and B2, respectively, in comparison to B3. However, the choice of the high seismic design level (i.e., B1) or the moderate seismicity level (i.e., B2) produces approximately 50% and 31% saving in total repair cost, respectively, in comparison to B3. Therefore, designing for the high or moderate seismicity levels results in net cost savings when potential significant ground shaking occurs. The highest net cost is for B3 and it reflects its expected poor seismic performance in

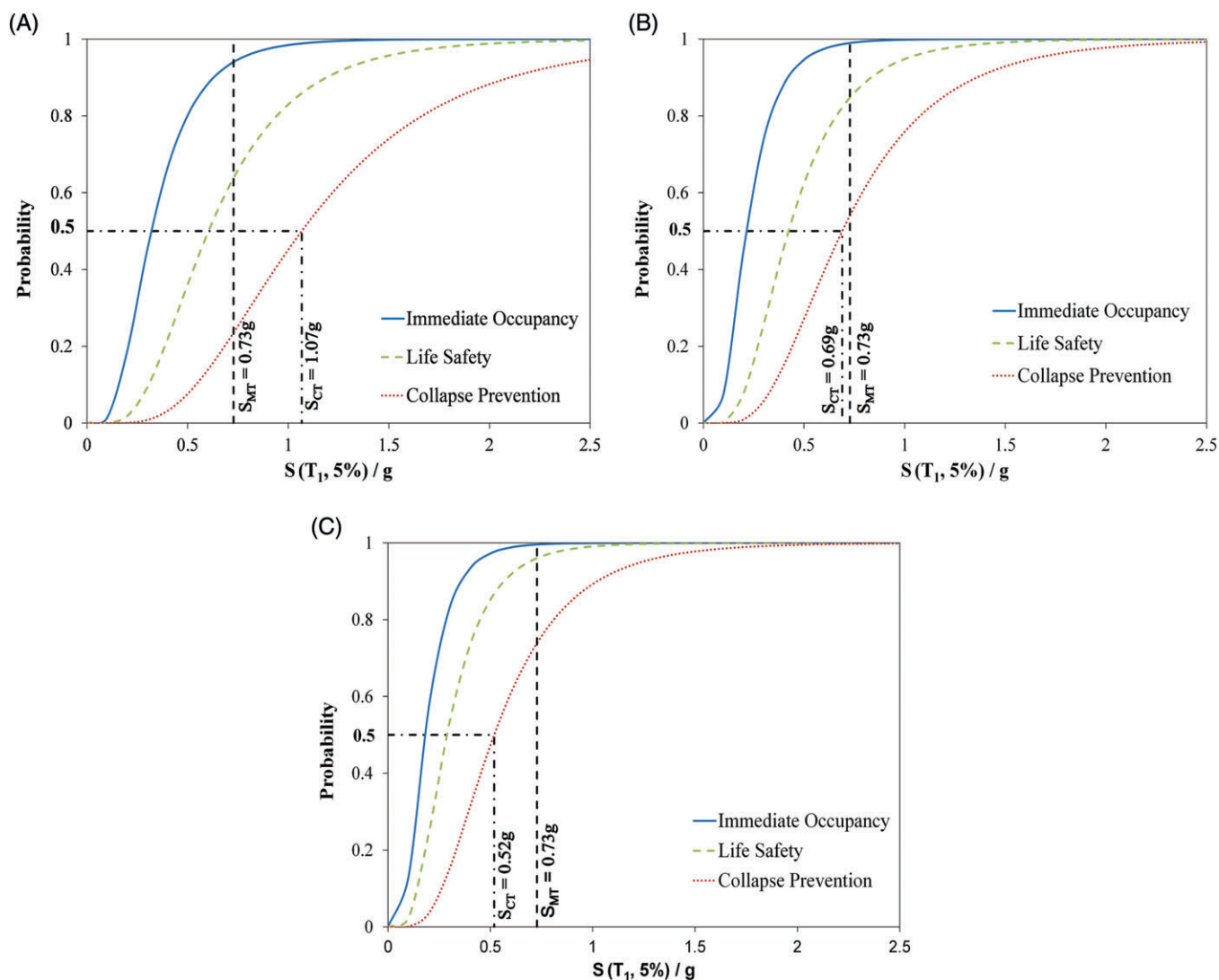


FIGURE 7 Fragility curves: (a) B1; (b) B2; (c) B3

TABLE 6 Summary of bill of quantities and construction material cost

	Concrete volume (m ³)			Steel quantity (ton)			Construction material cost			
	Columns	Slabs	Walls	Columns	Slabs	Walls	Concrete	Steel	Total cost	Material cost increase (%)
B1	802	2,175	1,400	43	525	220	\$297,904	\$536,764	\$834,668	26
B2	802	2,175	747	43	525	90	\$253,449	\$448,204	\$701,654	6
B3	802	2,175	574	43	525	48	\$241,705	\$419,679	\$661,384	0

TABLE 7 Damage percentages

Damage state	Structural damage (%)	Non-structural damage (%)
Immediate occupancy	30	5–40
Life safety	60	20–50
Collapse prevention	80	40–80

major earthquake events (MCE level) and the associated increase in total repair cost. The high repair cost also foresees an increase in the building downtime. This indicates that although B1 has the highest initial investment, its superior performance during major earthquake events would

reduce the life-cycle cost of the building and result in net cost savings. In addition to the savings due to the reduction in repair cost, extra savings would result from the shorter downtime and the faster revenue generation which are conservatively not accounted for in this study.

5 | SUMMARY AND CONCLUSIONS

Three 12-story office buildings proposed to be located in Dubai, UAE were designed and detailed in accordance with IBC'12 code and relevant standards. The buildings had special RC shear walls located along the building perimeter as their lateral force resisting system. Furthermore, the three

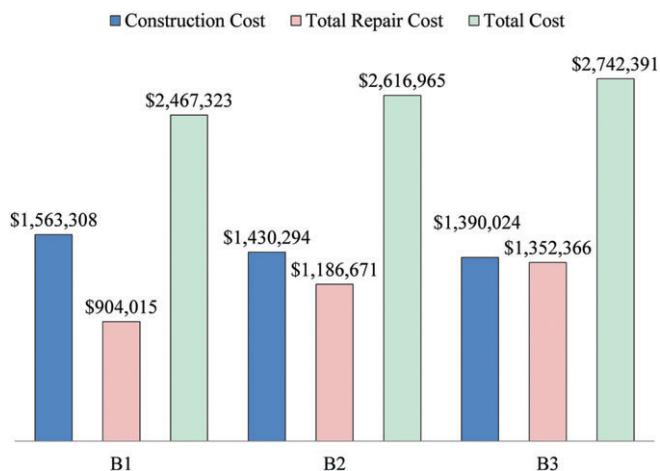


FIGURE 8 Overall cost comparison

buildings were designed for three seismic levels representing the high, moderate and low seismicity estimates of Dubai. Nonlinear pushover analysis was performed to validate the nonlinear simulation and evaluate the strength and deformation capacities. Additionally, the period dependent ductility was calculated from the pushover curves for the three designs. Furthermore, IDA was conducted to investigate the seismic performance of the buildings and evaluate their ACMR's. The seismic performance was assessed based on the IDA response plots and the damage states' fragility curves. For a comprehensive assessment of the buildings, the seismic design level impact on the total construction and life-cycle costs was investigated.

Based on pushover analysis results, it was observed that the three buildings had acceptable ductile behavior and over-strength capacity. Conforming to conventional engineering understanding, B1 had the highest ductility and over-strength. The IDA results provided a better insight into the seismic response of the buildings. It was observed that B1 had the highest margin of safety against collapse, while B2 was a borderline case. However, B3 had virtually no collapse margin of safety against potential major earthquake events. This was reflected in the probability of collapse at the MCE spectral accelerations for the three buildings. For instance, at MCE, there was a 23%, 54%, and 73% probability of collapse for B1, B2, and B3, respectively. From an economic standpoint, B1 compared to B2 and B3 required more material and had the highest construction cost, but had the least total repair cost at LS and CP performance levels.

Upon review of the presented results, the performance could be ranked as superior for B1, moderate but acceptable for B2 and marginal for B3. Taking the construction cost in considerations, it would be very reasonable to opt for increasing the initial investment by approximately 12.5% by choosing to design for high seismicity (i.e., B1 design) instead of low seismicity (i.e., B3 design). This will not only reduce the collapse probability and enhance the seismic response but will also significantly decrease the down time (and its direct and indirect cost) of the structure as well as the

associated potential repair costs. Additionally, it may very well be deemed reasonable to choose a moderate level of seismicity (i.e., B2 design) which will require a mere 2.9% increase in construction cost but will noticeably enhance the expected performance of the structure. It can be concluded that in light of the level of uncertainty, designing for higher seismicity is not always an uneconomical decision. Given the enhancements in the structural performance and the implications on downtime and potential life-cycle costs, it may be sound decision to be taken. However, the optimum design selection will vary depending on the objectives and particulars of the considered project. It might be appealing to increase initial investment and reduce potential repair costs (i.e., choose B1 or B2). Nevertheless, in some cases, it might not be a feasible option due to budget and cash flow restrictions.

The presented results are limited to a certain height and configuration for buildings with special RC shear walls. Additionally, the performance evaluation was based on the outcomes of the collapse assessment procedures suggested by FEMA P695. Although the implications were very realistic, further investigations are needed to account for the effect of building height, different structural configurations, irregularities and different lateral force resisting systems on the performance, construction, and life-cycle costs of buildings with different seismic design levels.

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REFERENCES

1. Croucher M, Badam R. The National 2013. Retrieved from <http://www.thenational.ae/uae/expo-2020/celebrations-and-school-holiday-across-uae-for-dubais-expo-2020-win>. Accessed November 30, 2015.
2. Mwafy A. Assessment of seismic design response factors of concrete wall buildings. *Earthq Eng Eng Vib*. 2011;10(1):115–127.
3. Shama A. Site specific probabilistic seismic hazard analysis at Dubai Creek on the west coast of UAE. *Earthq Eng Eng Vib*. 2011;10(1):143–152.
4. AlHamaydeh M, Aly N, Galal K. Impact of seismicity on performance and cost of RC shear wall buildings in Dubai, United Arab Emirates. *J Perform Constr Facil*. 2017;31(5):1–14.
5. AlHamaydeh M, Aly N, Galal K. Effect of diverse seismic hazard estimates on design and performance of RC shear wall buildings in Dubai, UAE. *The 2015 World Congress on Advances in Structural Engineering and Mechanics (ASEM15)*; August 25–29. Korea: Incheon; 2015.
6. AlHamaydeh M, Ibrahim N, Kaloti D, Alhasan O, Aisha A. Structural Design of Steel Plate Shear Wall Systems in Dubai, UAE. *CYRUS Institute of Knowledge (CIK) Conference, West-Meets-East: Exploring Sustainable Development, Innovation, and Entrepreneurship Opportunities*; January 9–11. Dubai, UAE: CYRUS Institute of Knowledge (CIK); 2014.
7. AlHamaydeh M, Galal K, Yehia S. Impact of lateral force-resisting system and design/construction practices on seismic performance and cost of tall buildings. *Earthq Eng Eng Vib*. 2013;12(3):385–397.
8. AlHamaydeh M, Abdullah S, Hamid A, Mustapha A. Seismic design factors for RC special moment resisting frames in Dubai, UAE. *Earthq Eng Eng Vib*. 2011;10(4):495–506.
9. AlHamaydeh M, Al-Shamsi G. Development of analytical fragility curves for representative buildings in Dubai, United Arab Emirates. *The International Conference on Earthquake Engineering; Skopje Earthquake—50 Years of European Earthquake Engineering (SE-50EEE)*; May 29–31. Macedonia: Skopje; 2013.

10. AlHamaydeh M, Abdalla J, Abdalla S, Al-Rahmani A, Mostafa A. Inelastic seismic demands for reinforced concrete frames in Dubai. In: The 14th European Earthquake Engineering Conference (14EEEC); Aug 30–Sep 3, 2010, Ohrid, Macedonia.
11. AlHamaydeh M, Yehia S, Aly N, Douba A, Hamzeh L. Design alternatives for lateral force-resisting Systems of Tall Buildings in Dubai, UAE. *Int J Civ Environ Eng.* 2012;6(2):143–146.
12. Al Haddad M, Siddiqi GH, Al Zaid R, Arafah A, Necioglu A, Turkelli N. A basis for evaluation of seismic hazard and design criteria for Saudi Arabia. *Eathq Spectra.* 1994;10(2):231–258.
13. Grünthal G, Bosse C, Sellami S, Mayer-Rosa D, Giardini D. Compilation of the GSHAP regional seismic hazard for Europe, Africa and the Middle East. *Ann Di Geofis.* 1999;42(6):1215–1223.
14. Sigbjornsson R, Elnashai A. Hazard assessment of Dubai, United Arab Emirates, for close and distant earthquakes. *J Earthq Eng.* 2006;10(5):749–773.
15. Pascucci V, Free M, Lubkowski Z. Seismic hazard and seismic design requirements for the Arabian Peninsula region. In: 14th World Conference on Earthquake Engineering. Chinese Association of Earthquake Engineering; Beijing, China; 2008.
16. Aldama-Bustos G, Bommer JJ, Fenton CH, Stafford PJ. Probabilistic seismic hazard analysis for rock sites in the cities of Abu Dhabi, Dubai and Ra's al Khaymah, United Arab Emirates. *Georisk Assess Manag Risk Eng Syst Geohazards.* 2009;3(1):1–29.
17. International Conference of Buildings Officials. *Uniform Building Code (UBC)*. International Conference of Building Officials: Whittier, CA; 1997.
18. International Code Council. *International Building Code (IBC)*. Country Club Hills, Illinois; 2012.
19. American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI). *Minimum Design Loads for Buildings and Other structures (ASCE7)*. Reston, Virginia; 2010.
20. Li M, Luu H, Wu C, Mo Y, Hsu T. Seismic performance of reinforced engineered cementitious composite shear walls. *Eathquakes Struct.* 2014;7(5):691–704.
21. United States Geological Survey (USGS) 2015. <http://geohazards.usgs.gov/designmaps/www/>. Accessed May 31, 2015.
22. Abu Dhabi Department of Municipal Affairs and International Code Council. *Abu Dhabi International Building Code (ADIBC)*. Abu Dhabi Department of Municipal Affairs: Abu Dhabi, UAE; 2011.
23. Computers and Structures, Inc. ETABS 2015. Berkeley, CA.
24. Reinhold M, Roh H, Sivaselvan M, et al. *IDARC2D Version 7.0: a program for the inelastic damage analysis of structures*. Tech. Repo. No. MCEER-09-0006. University at Buffalo, The State University of New York; 2009.
25. Al SM. *Estimation of Seismic Response Demands for R/C Framed Structures: An Insight Into the Nonlinear Seismic Behavior*. Saarbrücken, Germany: VDM Verlag; 2008.
26. Al Satari M, Anderson J. Nonlinearity effects on the seismic behavior of RC framed structures. *The 76th SEAOC Annual Convention*; September 26–29. Structural Engineers Association of California (SEAOC): Squaw Creek, CA; 2007.
27. Al Satari MH. *Estimation of Seismic Response Demands for RC Framed Structures*. University of Southern California: Los Angeles, California; 2005.
28. AlHamaydeh M, Najib M, Alawnah S. INSPECT: a graphical user interface software package for IDARC-2D. *SoftwareX.* 2016;5:243–251.
29. AlHamaydeh M, Najib M, Elayyan L. INSPECT-Lite: A GUI Pre-Processor Package for IDARC-2D Ver 7.0. *The 2015 World Congress on Advances in Structural Engineering and Mechanics (ASEM15)*; August 25–29. Korea: Incheon; 2015.
30. American Concrete Institute. *Building Code Requirements for Structural Concrete and Commentary (ACI 318)*. American Concrete Institute (ACI): Farmington Hills, MI; 2011.
31. Integrated Engineering Software (IES) Inc. Quick Concrete Wall 2015. Bozeman, MT.
32. Federal Emergency Management Agency (FEMA P695). *Quantification of Building Seismic Performance Factors*. Washington, DC: Applied Technology Council; 2009.
33. Pacific Earthquake Engineering Research Centre (PEER), PEER Ground Motion Database, University of California, Berkeley, CA; 2016. <http://peer.berkeley.edu/nga/>. Accessed February 20, 2016.
34. Shafei B, Zareian F, Lignos D. A simplified method for collapse capacity assessment of moment-resisting frame and shear wall structural systems. *Eng Struct.* 2011;33(4):1107–1116.
35. Purba R, Bruneau M. Seismic performance of steel plate shear walls considering two different design philosophies of infill plates. II: assessment of collapse potential. *J Struct Eng.* 2014;141(1):1–12.
36. Lee J, Han S, Kim J. Seismic performance evaluation of apartment buildings with central core. *Int J High-Rise Build.* 2014;3(1):9–19.
37. Gogus A, Wallace J. Seismic safety evaluation of reinforced concrete walls through FEMA P695 methodology. *J Struct Eng.* 2015;141(10):1–17.
38. American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI). *Seismic Rehabilitation of Existing Buildings (ASCE 41)*. Reston, Virginia; 2013.
39. Bruce Shaw Knowledge Centre 2015. <http://www.bruceshaw.com/knowledgecentre/chapters/middle-east/uae-average-construction-costs>. Accessed November 11, 2015.
40. Federal Emergency Management Agency. *Hazus-MH MR5 Technical Manual. Multi-Hazard Loss Estimation Methodology Earthquake Model*. Federal Emergency Management Agency: Washington, DC; 2010.
41. Structural Engineers Association of California (SEAOC) *Blue Book. Recommended Lateral Force Requirements and Commentary*. 7th ed. CA: Sacramento; 1999.
42. Krawinkler H. Challenges and progress in performance-based earthquake engineering. *International Seminar on Seismic Engineering for Tomorrow*. Tokyo, Japan; 1999.
43. Ghobarah A. Performance-based design in earthquake engineering: state of development. *Eng Struct.* 2001;23(8):878–884.

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