

Impact of Seismicity on Performance and Cost of RC Shear Wall Buildings in Dubai, United Arab Emirates

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Abstract: Unfortunately, available probabilistic seismic hazard studies are reporting significantly varying estimates for seismicity of Dubai. Given Dubai's rapid economic growth, it is crucial to assess the impact of the diverse estimates on the performance and cost of buildings. This research investigates and quantifies the impact of the high and moderate seismicity estimates of Dubai on the seismic performance and construction and repair costs of buildings with 6, 9, and 12 stories. The reference buildings are made up of reinforced concrete with special shear walls as their seismic force-resisting system. The seismic performance is investigated using nonlinear static and incremental dynamic analyses. Construction and repair costs associated with earthquake damages are evaluated to quantify the impact. Results showed that designing for higher seismicity yields significant enhancement in overall structural performance. In addition, the higher seismicity estimate resulted in slight increase in initial construction cost. However, the increase in initial investment is outweighed by significant enhancements in seismic performance and reduction in earthquake damages. This resulted in overall cost savings when reduction in repair and downtime costs are considered. DOI: [10.1061/\(ASCE\)CF.1943-5509.0001079](https://doi.org/10.1061/(ASCE)CF.1943-5509.0001079). © 2017 American Society of Civil Engineers.

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Introduction

The economy of the United Arab Emirates (UAE), specifically Dubai, has been rapidly growing over the last few decades. Significant investments are taking place in the various sectors, especially in the real estate sector. In addition, in November 2014, Dubai was announced to be the host city of the upcoming Exposition (Expo) 2020. As a result, substantial growth is taking place in the real estate sector. Several residential, commercial, and hotel buildings are going to be designed and constructed to accommodate the increase in population size. This sector region (i.e., UAE) suffers from considerable uncertainty in its seismicity level and the design guidelines that should be followed (AlHamaydeh et al. 2012). The seismicity level of the UAE and Dubai has been the matter of several research studies, such as Abdalla and Al-homoud (2004), Aldama-Bustos et al. (2009), and Khan et al. (2013). Nevertheless, unfortunately, there is not much consensus in these research studies about the seismicity levels that should be designed for in the UAE. This could be attributed to the lack of in-depth seismological data and historical recordings of ground motions in this region. Such data would have been useful in providing comprehensive and sound seismic design guidelines (AlHamaydeh et al. 2013). The

minimum seismic design requirements set by the local authorities in Dubai are based on the 1997 Uniform Building Code (UBC 1997). However, the municipality has been proactive in adapting to tremors that were felt and measured in the UAE in April 2013. They sent a circular to consulting offices in May 2013, raising the minimum requirement to Zone 2B for buildings higher than nine stories, and to Zone 2A for buildings between four and nine stories. Therefore, the unprecedented growth in the number of buildings in Dubai, combined with the lack of consensus on seismic design criteria, complicates the vulnerability to earthquakes. It is generally believed that the UAE has low seismicity. Nevertheless, over the last few years, a significant number of regional seismic activities, originating from faults surrounding the UAE, has been recorded by the Dubai Seismic Network (DSN). Additionally, the DSN has recorded some local seismic activities over the period from 2006 to 2014.

The UAE's seismicity is affected by earthquakes originating from near-fault and far-field seismic sources (Mwafy 2011). The most recent seismic hazard study for the UAE, available to the authors, was published in 2013 by Khan et al. (2013). The study provided a comprehensive probabilistic seismic hazard assessment and spectral accelerations for the entire UAE. Furthermore, it implemented a standardized earthquakes catalog for the UAE compiled from the U.S. Geological Survey (USGS), National Geosciences of Iran (USGS 2015), and the National Center of Meteorology & Seismology of UAE (NCMS 2015), which dates back 110 years. Furthermore, Khan et al. (2013) have used seven different ground motion prediction equations incorporating three next-generation attenuation equations owing to the lack of specific equations for the UAE. They attributed the UAE's seismicity to the seven seismic source zones shown in Fig. 1, which shows the location of the UAE relative to the seven distinct source zones contributing to its seismic hazard. These source zones are: (1) North Zagros, (2) South Zagros, (3) Zindab-Minab Transition, (4) Makran Top, (5) Makran Bottom, (6) Oman Mountains, and (7) Arabian Craton (Khan et al. 2013). In addition, the seismic hazard study

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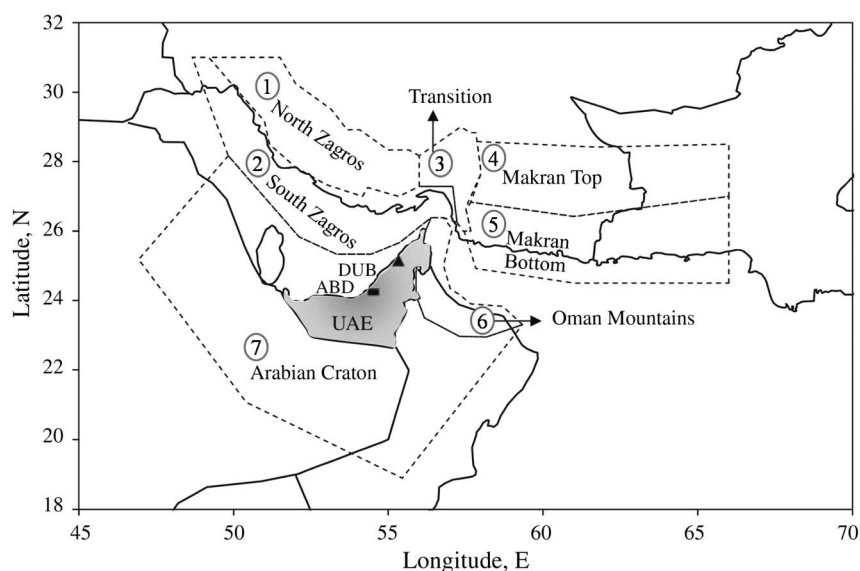


Fig. 1. Seismic source zones affecting UAE (reprinted from Springer *Natural Hazards*, “Probabilistic seismic hazard analysis and spectral accelerations for United Arab Emirates,” 2013, pp. 569–589, Zahid Khan, Magdi El-Emam, Muhammad Irfan and Jamal Abdalla, © Springer Science+Business Media Dordrecht 2013, with permission of Springer)

by Shama highlighted several local crustal faults in the UAE that might affect its seismicity level. These faults are Dibba, Wadi El Fay, Wadi Ham, Wadi-Shimal, Oman, and the West Coast fault (Shama 2011). Unfortunately, the level of seismicity is not clearly set because there is no strong consensus among researchers about the exact seismic level of the UAE or Dubai. On the other hand, reviewing the available probabilistic seismic hazard studies conducted for the UAE and Dubai clearly shows that there are significant variations in the estimated seismicity levels. In fact, results vary from “no seismic hazard” to “very high seismicity.” Table 1 is a summary of peak ground accelerations (PGA) from several probabilistic seismic hazard studies. The reported PGAs vary from less than 0.05 to 0.32 g. This is attributed to the differences in the used source zonation, recurrence parameters, earthquake catalogs, and ground motion prediction equations. The differences are mainly due to the lack of detailed seismological measurement and data in this region, and such data is required to provide a comprehensive and sound seismic hazard study (AlHamaydeh et al. 2013). The variation in seismicity was a driving factor for many research studies related to the impact on design of buildings in Dubai, such as AlHamaydeh et al. (2010), AlHamaydeh et al. (2011), and AlHamaydeh and Al-Shamsi (2013).

The objective of this paper is to investigate the impact of the seismicity hazard level on the performance, construction, repair,

and downtime costs of RC shear wall buildings in Dubai. In this regard, six RC shear wall buildings were designed and detailed following the 2012 International Building Code (IBC 2012) standards. The reference buildings were of 6, 9, and 12 stories. They were chosen to target the main sectors of buildings inventory in Dubai, UAE. These buildings were designed for two different seismic hazard levels, representing high and moderate seismicity estimates of Dubai. The different designs were compared on the basis of their seismic performance and construction and repair costs in order to investigate and quantify the impact of the seismic design level. The seismic performance was evaluated following the methodology outlined in FEMA P695 (FEMA 2009), a technical publication aiming to establish standard procedures for quantifying the seismic performance factors of buildings. The assessment methodology was based on nonlinear pseudostatic and dynamic analyses. The nonlinear response history analyses were performed using a set of ground motion records selected and scaled to represent the highest possible seismic activity in Dubai. As such, this would highlight the implications on design, seismic performance, and construction and repair costs of RC shear wall buildings designed for different seismic hazard levels in Dubai, if the high seismicity estimate turned to be the most realistic. In addition, total construction cost was estimated considering structural and nonstructural components. Finally, the repair cost was evaluated on the basis of the structural and nonstructural damage percentages adapted from the Structural Engineers Association of California (SEAOC) blue book (1999).

Table 1. Diversity in PGA Estimates for Dubai from Several Seismic Hazard Studies

Study by (year)	PGA
	(10% exceedance probability in 50 years) (g)
Al-Haddad et al. (1994)	<0.05
Grunthal et al. (1999)	0.32
Abdalla and Al Homoud (2004)	0.14
Sigbjornsson and Elnashai (2006)	0.16
Pascucci et al. (2008)	0.06
Aldama-Bustos et al. (2009)	<0.05
Shama (2011)	0.17
Khan et al. (2013)	0.12

Details of the Reference Buildings

The six reference buildings considered in this paper were intended to represent typical office buildings located in Dubai, UAE. The buildings had the number of floors ranging from 6 to 12 stories to represent the common buildings in Dubai. They were constructed using RC, with a typical floor plan as shown in Fig. 2. The plan consists of five 6-m (20 ft) bays and a total dimension of 30 × 30 m (100 × 100 ft). Furthermore, the overall structural height varies between 24 and 48 m (78 and 156 ft), with a typical floor height of 4 m (13 ft). The lateral force-resisting system

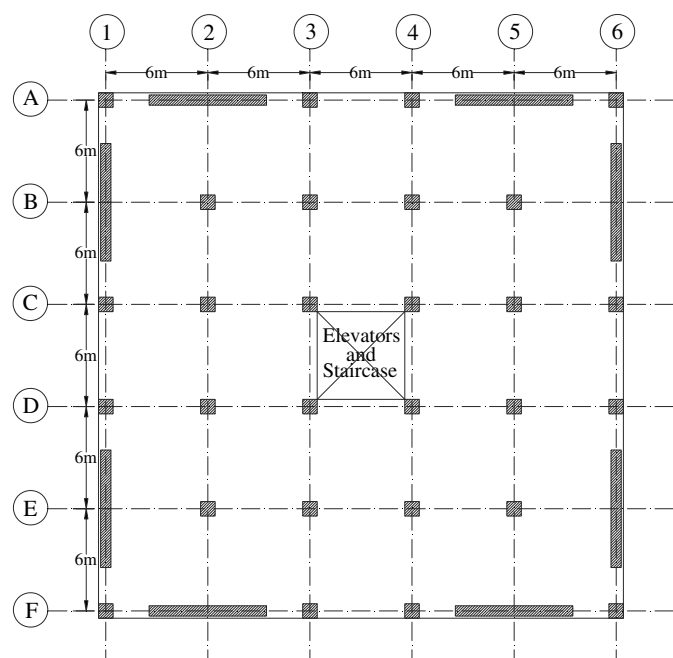


Fig. 2. Typical floor plan view

consisted of special RC shear wall placed along the perimeter of the building. This arrangement ensured that the center of mass was close to the center of rigidity, hence it avoided inherent torsional effects. In addition, placing the walls along the perimeter boosted the building's torsional resistance and reduced the shear demands on walls due to accidental torsion effects.

The gravity system consisted of RC square columns, whereas the floor system comprised cast-in situ flat plate. However, the gravity system was not designed to be part of the lateral force-resisting system. It was only designed to support vertical loads and to satisfy the deformation compatibility requirement. For design purposes, concrete compressive strength (f'_c) was assumed to be 28 MPa (4.0 ksi) for columns and slabs, and 38 MPa (5.0 ksi) for shear walls. Additionally, the yield strength (f_y) of reinforcement was assumed to be 420 MPa (60 ksi). Superimposed dead load (SDL) was 3.6 kPa (75 psf), excluding the self-weight of the concrete slabs. This SDL value is a conservative estimate commonly used for office buildings in Dubai. The breakdown of this estimate is as follows: 2 kPa (for 100 mm of leveling screed and flooring tiles), 1 kPa for partitions (usually movable partitions), and 0.6 kPa allowance for mechanical, electrical, and plumbing overhanging services. Curtain wall (cladding) load on the perimeter of each floor is 0.72 kPa (15 psf). Moreover, for office buildings, the typical floor's live load is 2.4 kPa (50 psf) and the roof live load is 1 kPa (20 psf), as per American Society of Civil Engineers [ASCE 7-10 (ASCE/SEI 2010)].

The six reference buildings were designed and detailed as per IBC 2012 requirements for two different seismic hazard levels, representing high and moderate seismicity estimates in Dubai. The selection of the two seismic design levels was driven by the existing uncertainty in seismic loading, and the substantial variability in reported seismic hazard levels for the UAE and Dubai. Therefore, it was deemed a reasonable choice for the objective of this paper to consider the highest and moderate seismicity levels. This would allow investigating the consequences of the possible alternative seismic loading levels available to designers. The highest seismicity level represents the upper bound, and it

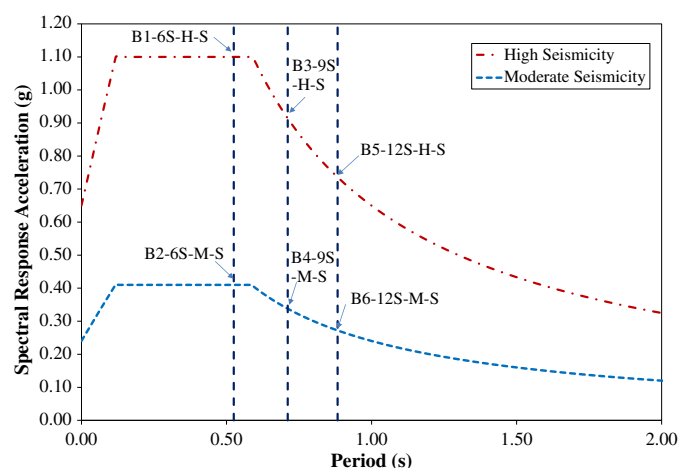


Fig. 3. Elastic design response spectra and proposed reference buildings

was obtained from USGS (2015). The USGS seismic hazard level for Dubai estimate was selected in this study (i.e., $S_s = 1.65$ g and $S_1 = 0.65$ g). Moreover, the moderate seismic design level represents the estimate by the Abu Dhabi International Building Code 2011 (ADIBC 2011) for Dubai. The elastic design response spectra for the two considered seismicity levels, along with the ASCE 7-10 estimated fundamental periods of the studied six buildings are presented in Fig. 3. As shown, at each seismic design level, three buildings with 6, 9, and 12 stories are designed with special RC shear walls. Summary of all buildings' details, including response modification factors (R and C_d), design spectral accelerations and elastic fundamental time periods, and approximate periods' upper limit are given in Table 2. It is worth mentioning that the ground motion input parameters (S_s and S_1) of the two seismicity levels (high and moderate) result in Seismic Design Category (SDC) D for the six considered buildings. Consequently, ordinary RC shear walls are not permitted by the design code (i.e., ASCE 7-10). Thus, all buildings are required to have special RC walls. Furthermore, choosing special detailing for both seismicity levels would allow investigating the direct impact of the seismic design level on the cost and performance of walls with same level of detailing requirements. The buildings are given a legend showing its ID [i.e., Building1 (B1) to Building6 (B6)], number of stories [i.e., 6 story (6S), 9 story (9S), or 12 story (12S)], seismic design level [i.e., high (H) or moderate (M)], and shear wall type [i.e., special (S) or ordinary (O)]. A site Class D is assumed for the six reference buildings. This assumption complies with IBC 2012 recommendations.

Design Summary

The buildings were designed and detailed in accordance with IBC 2012 standards, which refer to ASCE 7-10 for minimum design loads, and the American Concrete Institute [ACI 318-11 (ACI 2011)] for structural concrete requirements. The designs implemented state-of-the-art practices in design and construction followed in Dubai, UAE. For the design purposes, elastic analysis was done using three-dimensional models on ETABS commercial package. To determine the majority of the seismic mass, the gravity system was designed first and fixed for the three buildings.

The gravity system was designed to resist axial forces from all vertical loads in addition to the moments and shears induced from the deformation compatibility requirements. In order to ensure the

Table 2. Buildings' Characteristics

Reference building	Number of floors	Shear wall type	R	C_d	Design S_s (g)	Design S_1 (g)	Natural period (s)	Approximate period upper limit ($C_u \cdot T_a$) (s)
B1-6S-H-S	6	Special	6	5	1.65	0.65	1.03	0.73
B2-6S-M-S	6	Special	6	5	0.42	0.17	2.21	0.77
B3-9S-H-S	9	Special	6	5	1.65	0.65	1.45	1
B4-9S-M-S	9	Special	6	5	0.42	0.17	2.92	1.04
B5-12S-H-S	12	Special	6	5	1.65	0.65	1.8	1.24
B6-12S-M-S	12	Special	6	5	0.42	0.17	3.13	1.29

structural stability of gravity columns, they were designed to resist the induced actions (bending moments and shear forces) from the deformations that would be imposed by earthquake excitations on the building. The bending moments and shear forces were estimated on the basis of the maximum allowable interstory drift by IBC 2012, which is 2%. The stiffness of the columns was estimated using *ETABS* commercial package, by applying a force at the top and bottom of the considered story and getting the corresponding displacement. The shear forces were then calculated by multiplying the maximum allowable displacement and the stiffness of each column. Then, from the shear force, the moment was calculated, as shown in Eqs. (1) and (2)

$$V = K \times d \quad (1)$$

$$M = \frac{L}{2} \times V \quad (2)$$

where V = shear force; d = displacement (calculated using *ETABS* commercial package); M = bending moment; and L = column height. It should be noted that concrete shear capacity was found sufficient to resist the shear forces owing to the imposed deformations by applied seismic forces. Therefore, a minimum lateral reinforcement (i.e., column ties) was provided in columns, with reference to Clause 7.10 in ACI 318-11. The gravity system

components (i.e., flat plates and columns) were designed in accordance with the ACI 318-11 provisions using in-house design spreadsheets. For an optimized design, following common design trends in the UAE, columns cross sections and reinforcement were grouped and changed every three floors. The gravity system was common between buildings with the same number of floors. The gravity columns' design details for the six reference buildings are shown in Fig. 4. Figs. 4(a–c) show the dimensions and reinforcement details of the 6-, 9-, and 12-story buildings, respectively. On the left side of each figure, the columns' cross section dimensions are provided over each group of floors. The right side shows the vertical and horizontal reinforcement of the different columns (i.e., columns around the opening and remaining columns) in each group of floors. The minimum required slab thickness was calculated such that it satisfied the minimum requirements set out in ACI 318-11 Table 9.5(c). For the longest clear span of 5.3 m, the minimum required thickness was 177 mm. This figure was rounded up and 200-mm-thick flat plates are used. The flat plates were reinforced with T16 reinforcement bars spaced at 125 mm, top and bottom in both directions. Additional T20 reinforcement spaced at 125 mm (2 m long) were added on top of columns in both directions.

The lateral system was designed to resist the seismic lateral loads determined according to IBC 2012 static equivalent lateral force (SELF) method. The SELF method was permitted for all

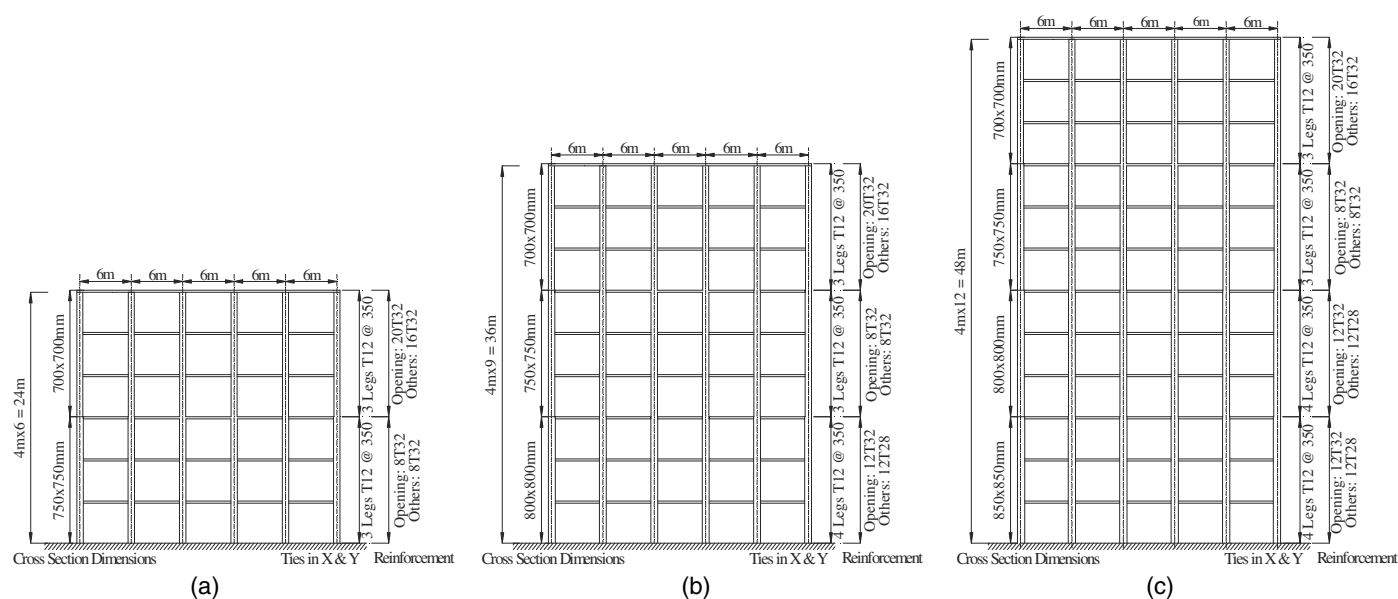


Fig. 4. Gravity columns dimensions and reinforcement details: (a) B1-6S-H-S and B2-6S-M-S; (b) B3-9S-H-S and B4-9S-M-S; (c) B5-12S-H-S and B6-12S-M-S

the six reference buildings. This is because the total height for all buildings did not exceed 48 m (160 ft), the SDC is D and no structural irregularities existed as per ASCE 7-10, Table 12.6-1. Linear static analysis was performed using *ETABS* commercial package to evaluate the induced forces and displacements from seismic forces. Then, shear walls were designed to satisfy strength and drift requirements. Interstory drift ratios were controlled within code, IBC 2012, limits (2%) by varying the shear wall stiffness through changing its in-plan length. Strength requirements were satisfied by designing the shear walls for the induced bending moments and shear forces by the seismic actions using *Quick Concrete Wall*. The thickness and reinforcement of the shear walls were changed every three floors to optimize the design and to match common design practices in Dubai. However, the in-plane length of walls was kept constant throughout the building's height to avoid any vertical structural irregularities. The need for specially detailed boundary elements was checked every three floors using the displacement-based approach. The use of displacement-based approach for checking the boundary elements vertical extent was preferred over the use of stress-based approach because the latter was proven to provide highly conservative requirements for the special detailing (Wallace and Moehle 1992; Thomsen and Wallace 2004). For practical constructability, boundary elements were designed to have the same wall thickness. Additionally, to comply with ACI 318-11 minimum thickness requirements and conform to typical design practices in the UAE, an aspect ratio of at least 25.4:304.8 mm (1:12 in.) was maintained between wall thickness and length. The minimum thickness of the walls depended on the unsupported height and length. Therefore, because the unsupported heights of the walls across the different buildings were constant (i.e., limited by the typical story height), it was necessary to impose a practical criterion on the wall thickness as the length was changed from building to building. This approach guaranteed that the different designs were subjected to the same guidelines, especially for sizing the cross sections of the walls. This would result in a fair response comparison among all designs because they followed similar bases that imitated typical design practices in the UAE. During initial dynamic analyses of the 12-story buildings, B5-12S-H-S and B6-12S-M-S, it was observed that the critical section was not at the base of the wall. This was contradicting the code assumption of having a single critical section at the base of cantilevered shear walls. The critical section resulting in the dynamic analysis was actually shifted from the wall base to the bottom of the lowest floor in the upper quarter of the building (10th floor). The initiated failure mechanism was governed by higher modes effects and the formation of plastic hinges at upper floors. The optimization done initially to the design by reducing dimensions and reinforcement for upper floors magnified the impact of higher modes effects. As a result, it triggered the failure and plastic hinge formation to be initiated at the reduced cross section. Therefore, the critical section became located at the weak spot at higher levels, which resulted in an unfavorable premature collapse mechanism. This observation has been highlighted by previous researchers, such as Tremblay et al. (2001), Bachmann and Linde (1995), and Panneton et al. (2006). It was also experimentally proven by shake table and cyclic loading tests (El-Sokkary et al. 2013). As an example, the modal analysis of the 12-story building, B5-12S-H-S, is shown in Table 3. It can be seen that there are clear separations between the individual modes' characteristics (periods and modal masses). This is generally expected in a flexural cantilever structural type (i.e., shear walls). The first mode effective mass is usually ranging from 50 to 70%, and the second mode period is approximately one-sixth of the first mode. This is consistent with many research studies which investigated the effects of higher modes on response

Table 3. Building B5-12S-H-S Modal Analysis Results

Mode	Frequency (Hz)	Period (s)	Modal participation factor	Relative modal weight (%)
1	0.57	1.7423	6.87	56.29
2	3.47	0.2878	−3.60	15.45
3	9.49	0.1054	1.97	4.62
4	17.09	0.0585	−1.25	1.85
5	25.49	0.0392	0.71	0.60
6	33.57	0.0298	−0.07	0.01
7	37.72	0.0265	2.90	10.01
8	41.63	0.0240	1.17	1.64
9	48.03	0.0208	−0.53	0.33
10	53.54	0.0187	−0.26	0.08
11	57.01	0.0175	−0.19	0.04
12	1,014.94	0.0010	2.76	9.08

of cantilever shear walls (e.g., Humar and Mahgoub 2003; Tremblay et al. 2001). Furthermore, it is clear that relative modal weights (%) and modal participation factors are relatively high at 7th, 8th, and 12th vibration modes, which highlights the impact of higher modes. As a result, the design was revised by keeping the cross section and reinforcement constant for the upper half of the 12-story (B5-12S-H-S and B6-12S-M-S) and 9-story buildings (B3-9S-H-S and B4-9S-M-S). For the 6-story buildings (B1-6S-H-S and B2-6S-M-S), a single cross section was used for all floors, with terminating boundary element at the third floor. This conforms to the state-of-the-art design and construction practices in Dubai, UAE. It also matches the design philosophy adopted in other 12- and 9-story buildings by keeping the same cross section and reinforcement for upper six floors. Summary of the shear walls design details is shown in Fig. 5.

Nonlinear Modeling

The six reference buildings were modeled using lumped plasticity formulations on the computer program *IDARC-2D* (Reinhorn et al. 2009). Because the buildings were symmetric, mass participation of torsional modes of vibration were low. Therefore, torsional effects were negligible and a two-dimensional model was sufficient to simulate the buildings' responses. The shear walls were idealized using macromodels by representing the structural members with equivalent elements possessing all nonlinear characteristics. The members' nonlinear characteristics depend on distribution of plasticity and yield penetration. A lumped plasticity model consisting of two nonlinear rotational springs located at the ends and an elastic member was used for the shear walls. The nonlinearity was concentrated at the locations of the nonlinear rotational springs. The flexural and shear deformations of the shear walls were modeled using the trilinear (three parameter) hysteretic model developed by Park et al. (1987). The trilinear hysteretic models allow controlling the stiffness degradation and strength deterioration due to ductility and energy. In addition, the axial deformations of the shear walls are considered by a linear-elastic spring. For the shear walls, the moment curvature and shear distortion were calculated using the fiber elements procedure of *IDARC-2D*. The wall cross section was divided into number of fibers and then subjected to increments of curvatures. From strain compatibility and equilibrium, the strains were calculated and used to compute the resulting axial forces and bending moments in the section (Reinhorn et al. 2009).

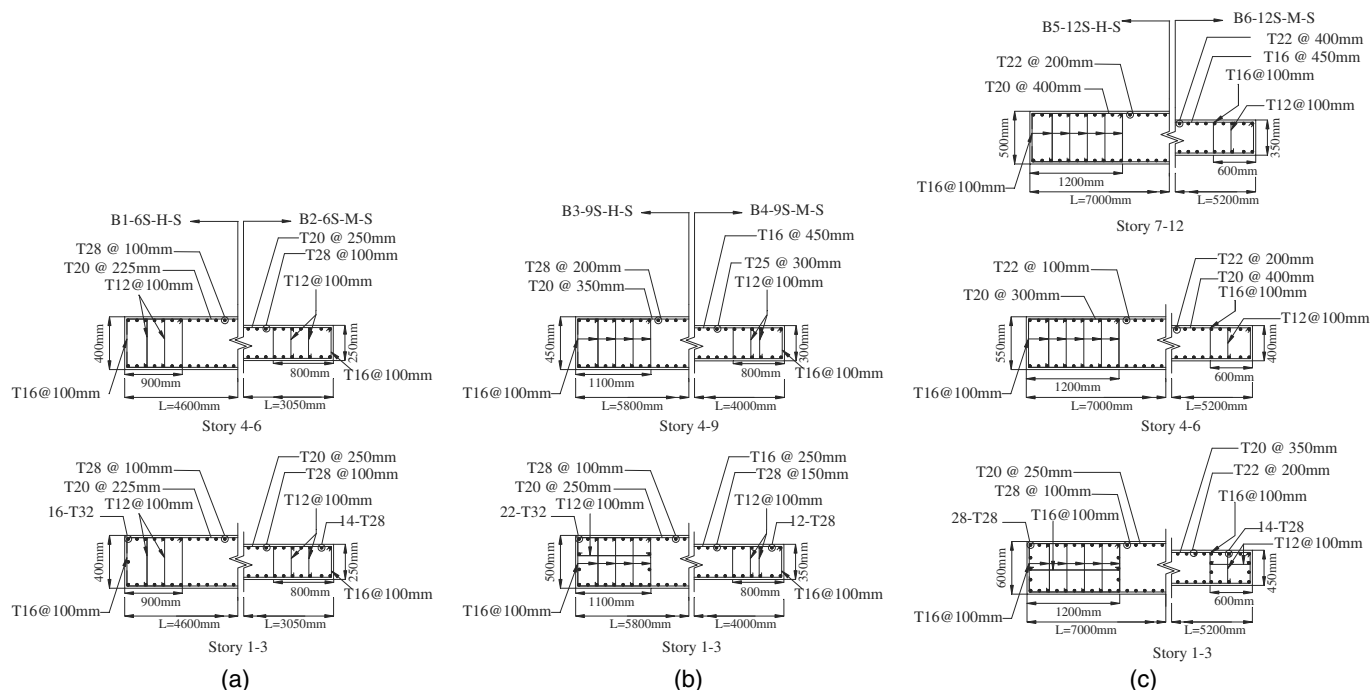


Fig. 5. Reinforced concrete shear walls dimensions and reinforcement details: (a) B1-6S-H-S and B2-6S-M-S; (b) B3-9S-H-S and B4-9S-M-S; (c) B5-12S-H-S and B6-12S-M-S

Results and Discussion

The buildings' seismic performance was evaluated following FEMA P695 methodology (FEMA 2009). This approach is based on nonlinear pseudostatic (pushover) analysis, incremental dynamic analysis (IDA), and fragility analysis. Pushover analysis was used to validate the nonlinear model and estimate the period-based ductility of the buildings. Then, IDA analysis was performed using a suite of far-field ground motion records to estimate the median collapse intensity and collapse margin ratio. The far-field ground motion records were 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 (2015) for Dubai ($S_s = 1.65\text{ g}$ and $S_1 = 0.65\text{ g}$). The selected scaling level simulates the worst, yet possible, seismic hazard scenario from distant sources (e.g., Zagros thrust or Makran subduction zone), as highlighted by Sigbjornsson and Elnashai (2006). Thus, it allows assessing the consequences of the selected seismic design level (i.e., high or moderate) on the seismic performance and construction and repair costs of RC shear wall buildings in Dubai. The calculated collapse margin ratios from the IDA were adjusted to account for uncertainties in design basis, test data, nonlinear modeling, and to consider the spectral shape of the ground motion records. The spectral content was accounted for on the basis of the calculated period-based ductility. Adjusted IDA results were finally used to calculate exceedance probabilities for ASCE-41 (ASCE/SEI 2013) performance levels, collapse prevention (CP), life safety (LS) and immediate occupancy (IO).

Nonlinear Pseudostatic (Pushover) Analysis

Pushover analysis is performed using an inverted triangle displacement profile as a pushing function for all buildings. The intensity is increased monotonically until the ultimate base shear degrades by 20%. The results are used to construct capacity curves (back-bone) for the reference buildings in the form of roof drift ratio versus

base-shear coefficient (i.e., base shear normalized by seismic weight). Pushover capacity curves are used to assess the building's deformation and strength capacities.

The capacity curves of the three buildings designed for the high seismicity estimate (i.e., B1-6S-H-S, B3-9S-H-S, and B5-12S-H-S) are shown in Fig. 6. Normalized base-shear capacities are 0.57, 0.5, and, 0.46 for B1-6S-H-S, B3-9S-H-S, and B5-12S-H-S, respectively. As expected, base-shear capacity is higher for the shorter building (6 story). This is attributed to the higher design forces that resulted from the relatively higher initial stiffness of squat shear walls compared with their slender counterpart. Building B1-6S-H-S reached a maximum roof drift ratio, prior collapse, of 6.5%, whereas B3-9S-H-S reached 6%, and B5-12S-H-S reached 7.25%. The three buildings have period-based ductility calculated,

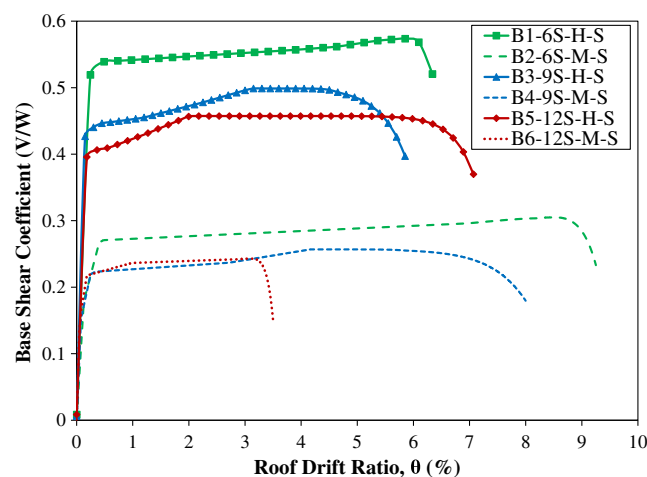


Fig. 6. Pushover capacity curves for the reference buildings (displacement control)

as recommended by FEMA P695, greater than 8. It can also be observed that in the three high-seismicity designs, the capacity curves do not experience severe degradation in strength or deterioration in stiffness. This matches the expected behavior of well-detailed special RC shear walls with confined boundary elements. It is noticed from the final damage states of the buildings, at 20% strength degradation, that static pushover analysis resulted in a failure mode at the base of the shear walls conforming to the design code-assumed critical section. The overall structural damage index reported by *IDARC-2D* is 0.359, 0.426, and 0.618 for B1-6S-H-S, B3-9S-H-S, and B5-12S-H-S, respectively. It is worth mentioning that these damages are concentrated at first-floor shear walls.

Established capacity curves for buildings designed for moderate seismicity (B2-6S-M-S, B4-9S-M-S, and B6-12S-M-S) are presented in Fig. 6. From shown capacity curves, normalized base-shear capacities are 0.31, 0.26, and 0.24 for buildings B2-6S-M-S, B4-9S-M-S, and B6-12S-M-S, respectively. Similar to the high-seismicity design, the 6-story building has the highest normalized base-shear capacity. This is due to its lateral system (shear walls) relatively higher stiffness, which resulted in higher demands. Maximum drift ratios, prior collapse achieved by B2-6S-M-S, B4-9S-M-S, and B6-12S-M-S are 9.25, 8, and 3.5%, respectively. The three designs have period-based ductility greater than 8 calculated, as recommended by FEMA P695. The overall structural damage index reported by *IDARC-2D* is 0.371 for B2-6S-M-S, 0.455 for B4-9S-M-S, and 0.359 for B6-12S-M-S. These damages are triggered at the first-floor shear walls only. Therefore, similar to high-seismicity designs, the pseudostatic pushover analysis results

of moderate-seismicity designs suggests a single critical section at the wall base. This conclusion matches with design code recommendations for regular buildings permitted to be designed following the static method (SELF) by ASCE7-10.

Nonlinear Incremental Dynamic Analysis

The seismic performance of the reference buildings is investigated under the random nature of earthquakes. Standard nonlinear pseudostatic analysis does not inherently fully capture the higher modes effects that usually govern the response of tall and irregular buildings. Consequently, the use of nonlinear dynamic analysis is more appropriate in such cases. Incremental dynamic analysis provides better insight of the expected structural response from the linear range through the nonlinear response, and until it loses stability and collapses (Vamvatsikos and Cornell 2004). The IDA, in this case, is performed using a very fine increment of 0.1 g for the spectral accelerations. The spectral accelerations are increased until all ground motion records caused the buildings to collapse or exceed the CP maximum drift ratio limit of 2%, as specified by ASCE-41. However, the maximum spectral acceleration for all the records is not increased more than 2.5 g. The total number of dynamic analysis runs performed for each reference building is around 1,100 (22 records $\times$ 2 components $\times$ 25 scale factors).

Fig. 7 presents the resulting IDA curves for high-seismicity designs, B1-6S-H-S, B3-9S-H-S, and B5-12S-H-S. The structural response derived from IDA curves can depend, to some extent, on the characteristics of the particular accelerograms used. Thus, the

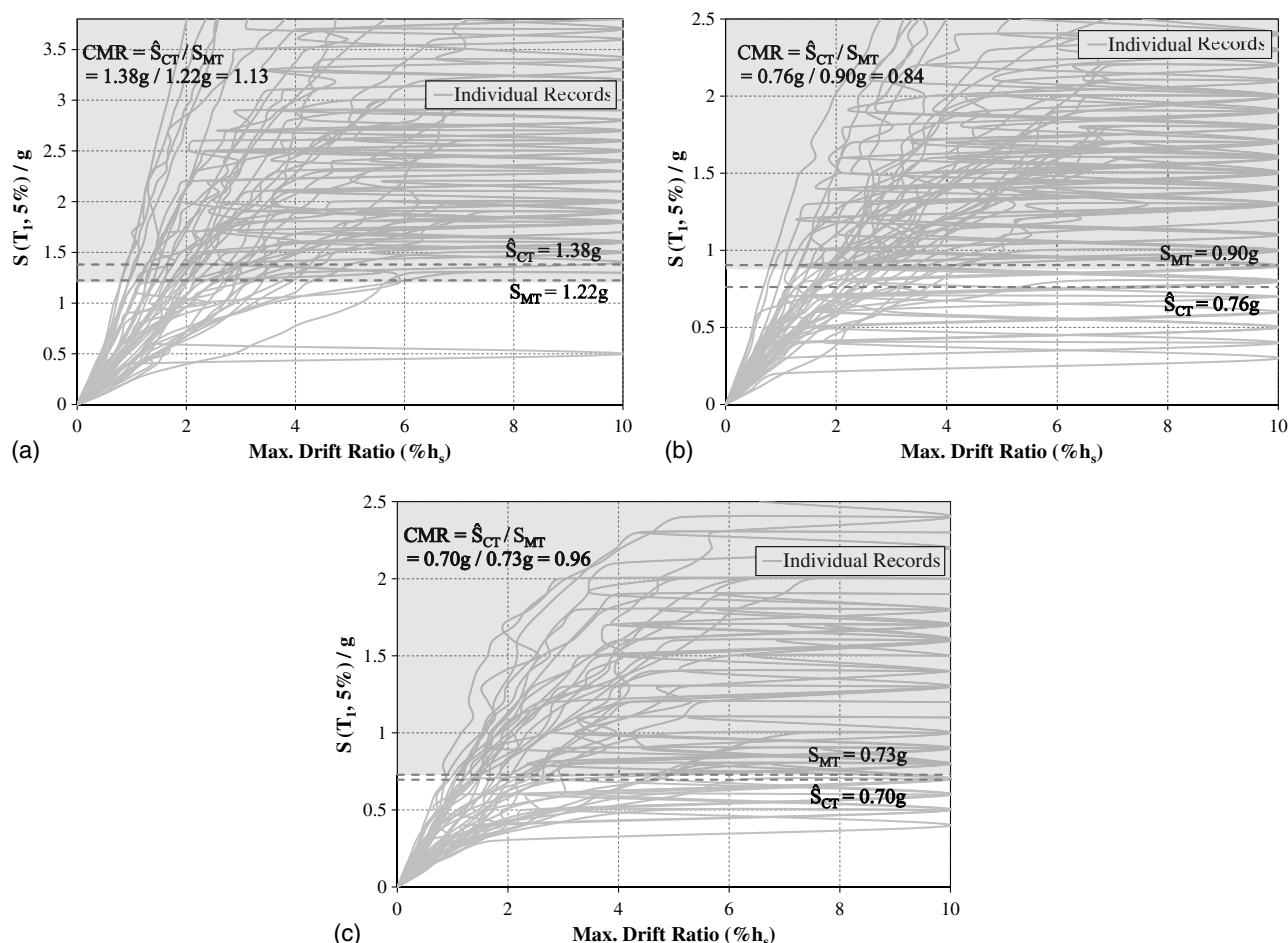


Fig. 7. Incremental dynamic analysis curves: (a) B1-6S-H-S; (b) B3-9S-H-S; (c) B5-12S-H-S

performance is judged on the basis of a suit of ground motion records to segregate this effect. On average, at low drift ratios (approximately up to 1%), the three designs (B1-6S-H-S, B3-9S-H-S, and B5-12S-H-S) exhibit a linear behavior. The same linear behavior results from some of the ground motion records up to the MCE spectral acceleration. At higher spectral accelerations, the structural response starts to vary, showing several patterns of nonlinearities, such as softening, hardening, and weaving. For only few records, the structure seems to soften and move to large drifts rapidly until it reaches collapse. Collapse in these curves, whether resulting from convergence issues, numerical instabilities, or very large drift ratio, is represented using a drift ratio of 10% and a flat line in IDA curves. Majority of the earthquake records caused severe hardening and weaving around the elastic response. The weaving observation conforms to the common equal displacement rule stating that inelastic and elastic displacements are equal for structures with relatively moderate time periods (Vamvatsikos and Cornell 2002). In addition, for some records, the hardening phenomenon in which the structure seems to perform better at higher intensities is somewhat against the common expectation (Vamvatsikos and Cornell 2002). This is because generally the time and pattern of the time history governs the response more than just the intensity. Moreover, the upward scaling done to the records makes the less responsive cycles at the beginning of the time history strong enough to cause damage and yielding of the structural elements. Therefore, some strong ground motion records at some intensity might cause early yielding of a specific floor, usually a low floor. This floor acts as a sacrificial fuse which reduces the response of higher floors (Vamvatsikos and Cornell 2002). Another very interesting observation that is clearly seen in IDA curves shown in Fig. 7 is what is called “structural resurrection.” This phenomenon has been observed by Vamvatsikos and Cornell (2002), and is defined as a severe hardening behavior. In structural resurrection, the building moves all the way to complete collapse (numerical instability or convergence issues) at some intensity. Then, at higher intensities, it shows a lower or higher response, but without collapsing. This happens because the time and pattern of the ground motion record at a particular intensity might be more damaging than at higher intensities. In other words, this particular intensity causes the structure to resonate. This is observed in the three designs, but more often in shorter buildings, B1-6S-H-S (6-story) and B3-9S-H-S (9-story) compared with the B5-12S-H-S (12-story building). The median collapse intensity ($\hat{S}_{CT}$), the spectral acceleration at which 50% (22) of the records caused the structure to collapse, is 1.38, 0.76, and 0.70 g for B1-6S-H-S, B3-9S-H-S, and B5-12S-H-S, respectively. Moreover, it can be observed from Fig. 7 that for building B1-6S-H-S there are a number of flat lines representing collapse from individual ground motion records above the MCE level (S_{MT}). This indicates that the 6-story building designed for high seismicity shows collapse due to several ground motion records, excessively large drift ratios and global instabilities, at spectral accelerations much higher than MCE level. For buildings B3-9S-H-S and B5-12S-H-S, the number of flat lines above MCE is relatively lower compared with B1-6S-H-S. Comparing the median collapse intensity ($\hat{S}_{CT}$) with MCE level (S_{MT}), the collapse margin ratio (CMR) is 1.13, 0.84, and 0.96 for B1-6S-H-S, B3-9S-H-S, and B5-12S-H-S, respectively. Based on CMR, only the 6-story building design (B1-6S-H-S) has an acceptable collapse margin of more than 1. However, the CMR has to be adjusted to account for sources of uncertainties and spectral shape of ground motion records before being used to judge the structural performance. The adjusted values for CMRs are shown subsequently. It should be noted that the MCE level (S_{MT}) refers to the highest seismicity estimate level. This is particularly chosen to achieve the objectives

of this research of investigating the performance and construction and lifecycle cost of the different designs under the high seismic hazard that might be present in Dubai, UAE. This scenario simulates a conservative possible seismic hazard in Dubai and its consequences on performance of buildings designed for two different seismic hazard levels.

Fig. 8 depicts IDA curves for moderate seismicity designs (B2-6S-M-S, B4-9S-M-S, and B6-12S-M-S). Incremental dynamic analysis curves show that, on average, the three designs exhibit a linear behavior at low drift ratios except for some ground motion records in which the structure remains linear up to significantly high spectral accelerations. At higher spectral accelerations, most records show nonlinear structural response, such as softening, hardening, and weaving. Mainly, significant weaving is observed in the three designs and, in some cases, structural resurrection is noticed. It is interesting to highlight that structural resurrection is most observed in the 6-story building (B2-6S-M-S), and least observed in the 12-story building (B6-12S-M-S). Median collapse intensity ($\hat{S}_{CT}$) is 0.78 g for B2-6S-M-S, 0.51 g for B4-9S-M-S, and 0.48 g for B6-12S-M-S. Furthermore, calculated CMRs are 0.64, 0.56, and 0.66 for B2-6S-M-S, B4-9S-M-S, and B6-12S-M-S, respectively. None of the three designs have any margin of safety against structural collapse based on CMRs. However, decision about performance cannot be made completely based on CMRs without doing the proper adjustments. The adjusted CMRs are presented subsequently.

It is worth mentioning that the reference buildings had an elastic response up to drift ratios of around 1% based on the IDA results shown in Figs. 7 and 8. Nonetheless, pushover analysis results depicted in Fig. 6 show that elastic ranges are below the 1% drift ratio. This could be attributed to the limitations of conventional pushover analysis, where a constant mode shape is used to capture the structure's response. As such, the pushover analysis predictions are sensitive to the selected load pattern (Elnashai and Di Sarno 2008). Pushover analysis attempts to capture the dynamic response, and should be treated as such. Therefore, dynamic analysis predictions will vary from pushover analysis results (Krawinkler and Seneviratna 1998).

Fragility Analysis

The seismic vulnerability of the different designs is assessed using fragility curves relating the probabilities of exceeding a performance level to the spectral acceleration demand. Incremental dynamic analysis results are first used to calculate the probabilities of exceeding the three performance levels (i.e., IO, LS, and CP) at every spectral acceleration. Based on the analytically calculated probabilities, a fragility function is derived. The derived fitting fragility curve is based on a cumulative distribution function, following the maximum likelihood method by Baker (2015). This method estimates fragility parameters (i.e., mean and standard deviation/dispersion) to produce a fragility distribution that would most likely reproduce observed data. The fitted fragility functions are then adjusted according to FEMA P695 to account for record-to-record (β_{RTR}), design basis (β_{DR}), test data (β_{TD}), and modeling uncertainties (β_{MDL}). In this case, record to record uncertainty (β_{RTR}) is given a fixed value of 0.4 for the chosen far-field records set. Moreover, there is high confidence in the basis of design requirements because the buildings are designed according to the relevant IBC 2012 standards. Hence, from Table 3-1 in FEMA P695, a value of 0.2 is selected for the design basis uncertainty (β_{DR}), this corresponds to a medium completeness and robustness. For the lateral system used (i.e., special RC shear walls), the materials, components, assembly, and system behavior are well understood and

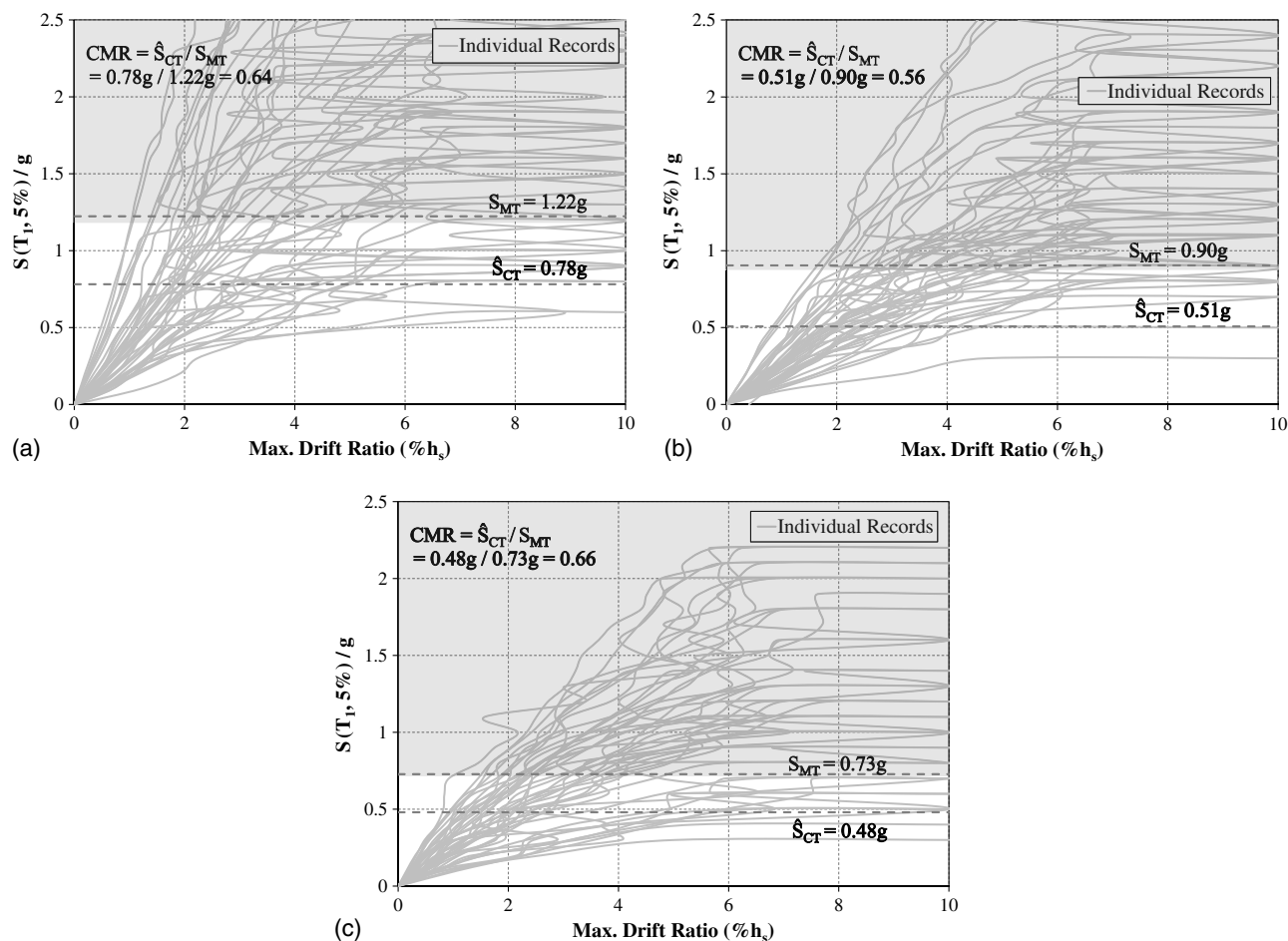


Fig. 8. Incremental dynamic analysis curves: (a) B2-6S-M-S; (b) B4-9S-M-S; (c) B6-12S-M-S

documented. Thus, from Table 3-2 in FEMA P695 (FEMA 2009), the test data uncertainty (β_{TD}) is given a value of 0.2 corresponding to a medium completeness and robustness and high confidence in test results. Finally, corresponding to medium representation of collapse characteristics, high accuracy and robustness of nonlinear analysis models, the nonlinear modeling uncertainty (β_{MDL}) is given a value 0.2. This is obtained from Table 5-3 in FEMA P695 (FEMA 2009). The nonlinear modeling is performed using *IDARC-2D* which accounts for plastic hinge and crack formation in all structural elements. It also accounts for shear, axial, and flexural deformation in structural members. However, the nonlinear analysis using *IDARC-2D* is limited to two dimensions. Combining the different uncertainty parameters, using Eq. (3), gives a total system collapse uncertainty (β_{TOT}) of 0.525

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

The total system collapse uncertainty is used to adjust the fragility curves for the three performance levels. However, for the collapse fragilities, the adjusted probabilities are also multiplied by the spectral shape factor (SSF) to account for the natural period elongation prior collapse. Spectral shape factor is given in Tables 7-1a and 7-1b in FEMA P695 based on seismic design level, fundamental period of the structure, and period-based ductility.

Analytical, lognormal fitted, and adjusted collapse fragility curves for buildings B1-6S-H-S, B3-9S-H-S, and B5-12S-H-S (high seismicity designs) are shown in Fig. 9. Collapse is defined

in FEMA P695 as either when the structure reaches excessive lateral drift (global dynamic instability) or through limit state criteria. In this study, the latter option is adopted in creating collapse fragility curves. Therefore, collapse in this case is defined as exceeding the drift ratio limit of CP limit state of ASCE41. The adjusted median collapse intensity (S_{CT}) is 1.92, 1.1, and 1.07 g for buildings B1-6S-H-S, B3-9S-H-S, and B5-12S-H-S, respectively. adjusted collapse margin ratios (ACMRs) are evaluated using the adjusted median collapse intensity (S_{CT}) and S_{MT} . The calculated ACMR is 1.57 for B1-6S-H-S, 1.23 for B3-9S-H-S, and 1.47 for B5-12S-H-S. This indicates 57, 23, and 47% margin of safety against collapse for buildings B1-6S-H-S, B3-9S-H-S, and B5-12S-H-S, respectively. Similarly, collapse fragility curves for B2-6S-M-S, B4-9S-M-S, and B6-12S-M-S (moderate seismicity designs) are given in Fig. 10. Adjusted median collapse intensity (S_{CT}) is 1.09, 0.75, and 0.69 g for buildings B2-6S-M-S, B4-9S-M-S, and B6-12S-M-S, respectively. Moreover, ACMR, ratio between S_{CT} and S_{MT} , is 0.89 for B2-6S-M-S, 0.82 for B4-9S-M-S, and 0.95 for B6-12S-M-S. This reflects that the three moderate seismicity designs have no margin of safety against collapse.

Fragility curves for IO, LS, and CP are given in Fig. 11 for the six reference buildings. In general, it is observed that at the three performance levels fragility curves are much steeper for the buildings designed for moderate seismicity. This reflects higher exceedance probabilities compared with the buildings designed for high seismicity, which have much flatter fragility curves. Additionally, at MCE level (S_{MT}), B1-6S-H-S has 20% probability of exceeding CP limits state, 66% of exceeding LS limit state, and 95% of

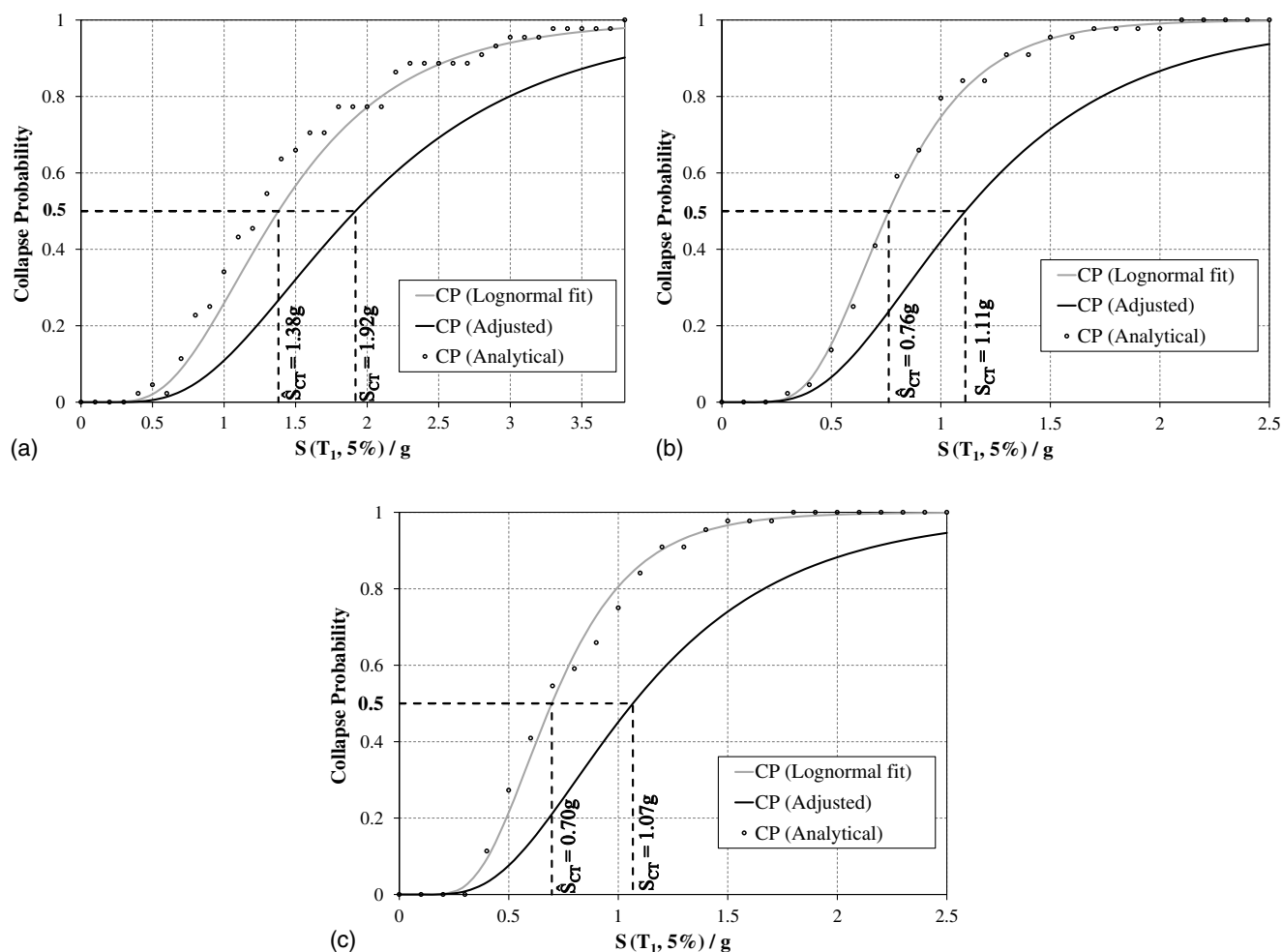


Fig. 9. Collapse fragility curves: (a) B1-6S-H-S; (b) B3-9S-H-S; (c) B5-12S-H-S

exceeding IO performance level. On the other hand, B2-6S-M-S has 58% probability of exceeding CP performance level, 96% probability of exceeding LS, and 100% probability of exceeding IO. Similarly, for B3-9S-H-S at the MCE level (S_{MT}), the probabilities are 34, 71, and 97% of exceeding CP, LS, and IO, respectively, whereas B4-9S-M-S has 64, 97, and 100% probabilities of exceeding CP, LS, and IO damage states, respectively at MCE level. Finally, B5-12S-H-S has a 23% probability of exceeding CP, 64% of exceeding LS, and 94% of exceeding IO at the MCE level. Conversely, B6-12S-M-S has 54% probability of exceeding CP, 85% probability of exceeding LS, and 99% probability of exceeding IO. It is interesting to note that the superior performance of high seismicity designs is more evident at LS and CP performance levels.

Construction and Maintenance Cost Comparison

The impact of the seismic design level on the construction and repair costs is quantified. This quantification provides more tangible measures for the engineers and decision makers. For the objective of this study, focus is on the direct economic losses owing to earthquakes, in terms of repair costs associated with structural and nonstructural damages. Therefore, indirect costs, content repair, or replacement costs are excluded. The total construction cost is estimated on the basis of the cost of structural materials (i.e., concrete and reinforcement steel) and nonstructural components cost.

Structural materials cost is approximated using average market unit prices, whereas nonstructural cost is taken as 55% of the total construction cost. Table 4 presents a summary of reference buildings' construction cost. It should be noted that this cost only represents the skeleton construction cost, considering the superstructure and nonstructural elements. It does not account for substructure cost, other construction costs, and the contractor's profit, and so forth. This is done to isolate other factors and focus on the seismic design level impact on construction and repair costs. Nonstructural components' cost in most office buildings is a significant portion of total cost. Therefore, majority of potential damage and losses from earthquakes are expected to occur in nonstructural components. This is consistent with the recommendation of Taghavi and Miranda (2003) for including the impact of nonstructural components' cost in total construction cost and response assessment.

As expected, there is an increase in the construction cost of the buildings designed for the high seismicity estimate. However, the increase is only 9% for 12-story buildings and 6% for 6- and 9-story buildings.

Repair cost is calculated on the basis of structural and nonstructural damage percentages adapted from the SEAOC blue book. The SEAOC blue book performance levels, its description, and damage percentages are summarized in Table 5. It can be seen that structural performance level (SP2) and nonstructural performance level (NP2) corresponds to IO performance level, SP3:NP3 parallels LS performance level and SP4:NP4 is equivalent to CP performance level.

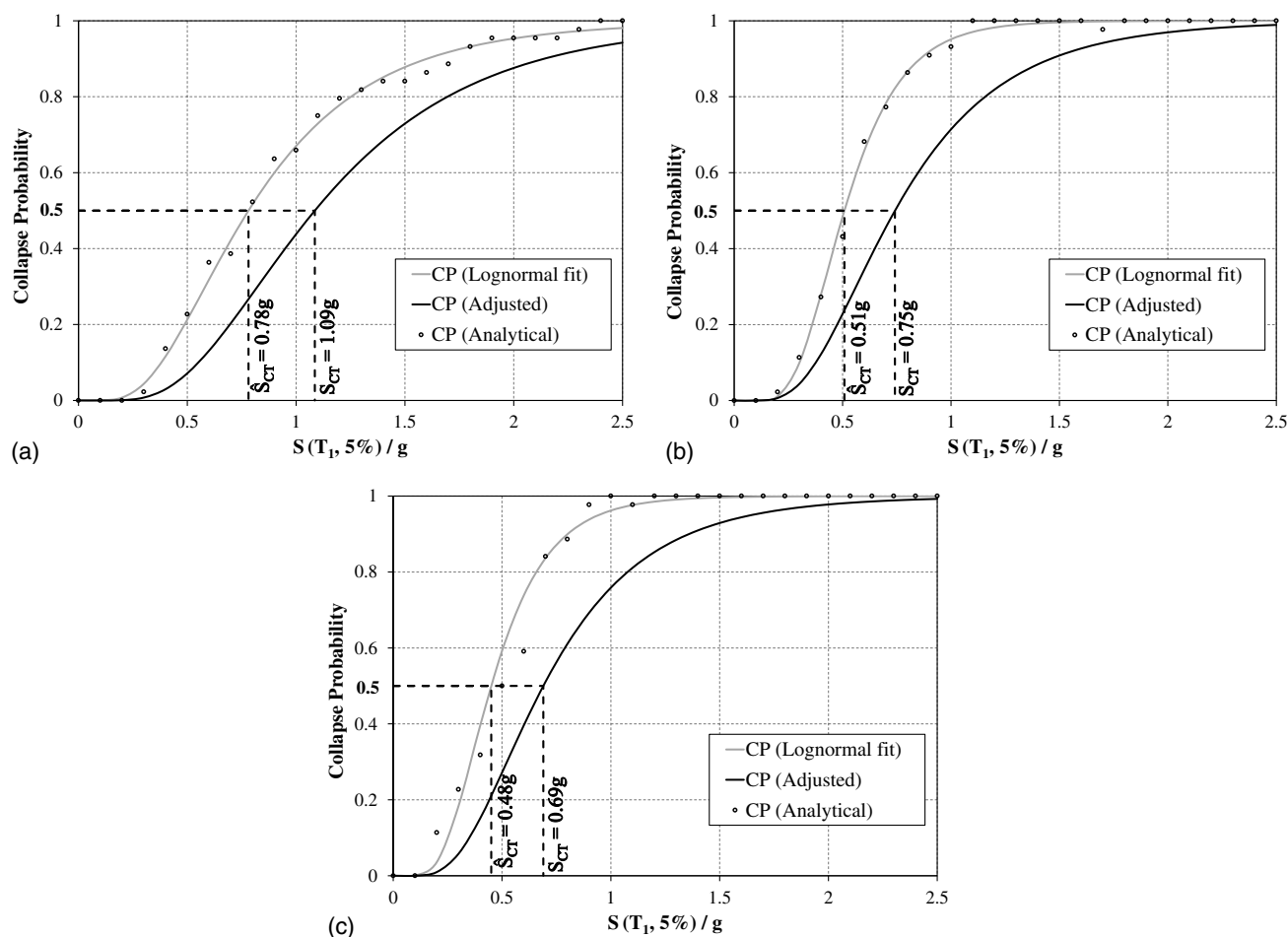


Fig. 10. Collapse fragility curves: (a) B2-6S-M-S; (b) B4-9S-M-S; (c) B6-12S-M-S

From the range of nonstructural damage ratios provided by the SEAOC blue book, the lower bound is chosen. The selection is based on experience because the reference buildings' lateral system consists of shear walls, which are relatively more rigid compared with other structural systems. The structural and nonstructural repair costs are calculated as follows using HAZUS economic losses estimation methodology [Federal Emergency Management Agency (FEMA 2010)]:

Structural repair cost(\$)

= Structural material cost

× Probability of exceeding alimit state at MCE level

× Structural damage ratio at the limit state

Nonstructural repair cost(\$)

= Nonstructural cost

× Probability of exceeding alimit state at MCE level

× Nonstructural damage ratio at the limit state

The calculated repair costs at the three performance levels, along with the total construction costs, are shown in Fig. 12. It is observed that at IO performance level, repair and construction costs are higher for the buildings designed for high seismicity (i.e., B1-6S-H-S, B3-9S-H-S, and B5-12S-H-S). This conclusion is valid for 6-, 9-, and 12-story buildings. Nevertheless, at LS and CP, the

moderate seismicity designs (i.e., B2-6S-M-S, B4-9S-M-S, and B6-12S-M-S) have higher repair and construction costs. The increase in repair costs at IO performance level when the buildings are designed for the higher seismicity estimate is in the range of 5–10%. However, the reduction in repair cost is ranging between 14 and 25% at LS, and between 42 and 63% at CP performance level. This reflects net cost saving associated with designing for a conservative seismic hazard estimate at LS and CP performance levels. The higher repair cost also indirectly reflects an increase in downtime of the structure.

Summary and Conclusions

This paper investigates the seismic hazard level impact on the performance and overall cost of buildings in Dubai. The investigation targets 6-, 9-, and 12-story buildings located in Dubai, and represents the main inventory of buildings. The selected reference buildings are designed and detailed following state-of-the-art design and construction practices in Dubai based on IBC 2012. The buildings' lateral force-resisting system consists of special RC shear walls. The seismic performance and collapse safety of the reference buildings are investigated using nonlinear pseudostatic and dynamic analyses, namely pushover analysis and IDA, based on FEMA P695 methodology. The nonlinear analyses are performed using a fully detailed fiber-based finite element model developed on *IDARC-2D*. A large number of nonlinear analyses are performed to capture the buildings' response to seismic loads. Performance

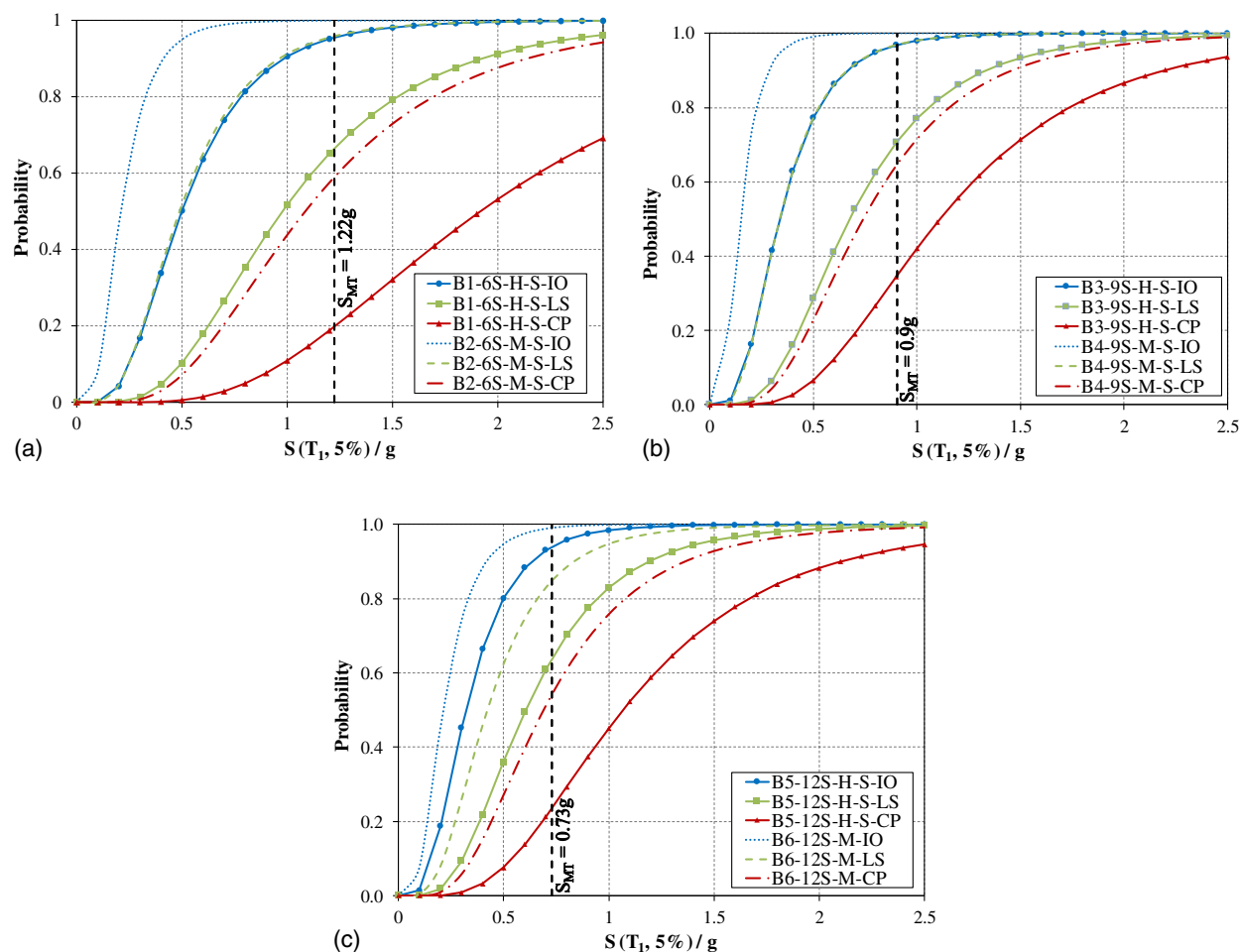


Fig. 11. Immediate occupancy, life safety, and collapse prevention fragility curves: (a) 6-story buildings; (b) 9-story buildings; (c) 12-story buildings

Table 4. Summary of Construction Cost

Reference building	Construction cost (in U.S. dollars)		
	Structural cost	Nonstructural cost	Total construction cost
B1-6S-H-S	397,499	364,320	761,819
B2-6S-M-S	353,069	364,320	717,389
B3-9S-H-S	611,710	546,480	1,158,190
B4-9S-M-S	542,446	546,480	1,088,926
B5-12S-H-S	834,668	728,640	1,563,308
B6-12S-M-S	701,654	728,640	1,430,294

Table 5. SEAOC Blue Book (1999) Performance Levels and Damage Percentages

Structural performance level	Nonstructural performance level	Qualitative description	Structural damage ratio (%)	Nonstructural damage ratio (%)
SP1	NP1	Operational	0	0–10
SP2	NP2	Occupiable	30	5–40
SP3	NP3	Life safe	60	20–50
SP4	NP4	Near collapse	80	40–80
SP5	NP5	Collapsed	100	>70

quantification is based on pushover capacity curves, ACMRs, seismic vulnerability, and probabilities of exceeding IO, LS, and CP at MCE. Finally, the impact of the seismic performance enhancement, due to choice of seismic design level, on the total cost is evaluated.

This considered the net effect of construction, repair and downtime costs to present more realistic and solid conclusions.

Nonlinear response history analysis of preliminary optimized designs revealed unfavorable response for all reference buildings. An undesirable collapse mechanism caused by higher modes effects is noticed. This resulted in imposing greater demands at upper floors with reduced cross sections. Therefore, designs were revised to consider both enhancing the expected seismic performance and optimizing the construction cost. It is noteworthy that initial designs strictly followed the code design approach. Based on the static method outlined in the design code (IBC 2012), cantilevered shear walls are expected to have a single critical section at the base. Yet, the dynamic analysis results showed that the critical section causing the collapse was shifted to upper floors with reduced sections. This is particularly clear in 9- and 12-buildings, which are more flexible structures compared with 6-story buildings.

Based on pushover capacity curves, as expected, all reference buildings showed a ductile response. However, the buildings designed for the high seismicity estimate of Dubai exhibited enhancements in their strength and inelastic deformation capacities. The calculated ACMRs reflected sufficient margin of safety (i.e., more than 20%) against collapse at MCE for the buildings designed for higher seismicity. On the other hand, moderate seismicity designs showed no margin of safety against collapse at MCE level. Furthermore, when the buildings are designed adapting the conservative seismicity estimate, there is a clear enhancement in their seismic vulnerability. This is clear in the flatter fragility curves for high seismicity designs at IO, LS, and CP performance levels.

		Moderate Seismicity			High Seismicity		
		Performance Level	Value	Reference	Performance Level	Value	Impact (% Increase/ Decrease)
6-Story (B2-6S-M-S & B1-6S-H-S)	Construction Cost		\$ 717,389	100%		\$ 761,819	106
	Repair Cost	IO	\$ 124,137	100%	IO	\$ 130,593	105
		LS	\$ 273,317	100%	LS	\$ 205,500	75
		CP	\$ 248,346	100%	CP	\$ 92,746	37
9-Story (B4-9S-M-S & B3-9S-H-S)	Construction Cost		\$ 1,088,926	100%		\$ 1,158,190	106
	Repair Cost	IO	\$ 190,058	100%	IO	\$ 204,512	108
		LS	\$ 421,721	100%	LS	\$ 338,188	80
		CP	\$ 417,631	100%	CP	\$ 240,706	58
12-Story (B6-12S-M-S & B5-12S-H-S)	Construction Cost		\$ 1,430,294	100%		\$ 1,563,308	109
	Repair Cost	IO	\$ 244,459	100%	IO	\$ 269,623	110
		LS	\$ 481,712	100%	LS	\$ 413,779	86
		CP	\$ 460,501	100%	CP	\$ 220,614	48

Fig. 12. Impact of seismicity on construction and repair costs

The probabilities of exceeding IO, LS, and CP at MCE are significantly reduced when the high seismic design level is chosen, compared with the moderate design level. These clear enhancements in seismic performance resulting from the higher seismic design level is associated with limited increase (i.e., 6-9%) in initial construction cost. However, the total cost, considering the net effect of construction and repair costs, is reduced for higher seismicity designs at LS and CP performance levels. Therefore, the increase in initial investment associated with designing for conservative seismicity level is outweighed by the enhancement in seismic performance, and the reduced repair and downtime costs. Under the idealized conditions and modeling assumptions, these conclusions are applicable for typical buildings ranging between 6 and 12 stories. The focus of this study was given to the impact of seismic design level on overall cost, considering cost of superstructure and nonstructural components. In fact, nonstructural components cost contributes to the majority of total construction and potential repair costs. Hence, impact of seismic design level on other costs such as foundation cost is beyond the scope of this study.

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