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Assessment of seismic design response factors of concrete wall buildings

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Abstract: To verify the seismic design response factors of high-rise buildings, five reference structures, varying in height from 20- to 60-stories, were selected and designed according to modern design codes to represent a wide range of concrete wall structures. Verified fiber-based analytical models for inelastic simulation were developed, considering the geometric nonlinearity and material inelasticity of the structural members. The ground motion uncertainty was accounted for by employing 20 earthquake records representing two seismic scenarios, consistent with the latest understanding of the tectonic setting and seismicity of the selected reference region (UAE). A large number of Inelastic Pushover Analyses (IPAs) and Incremental Dynamic Collapse Analyses (IDCAs) were deployed for the reference structures to estimate the seismic design response factors. It is concluded that the factors adopted by the design code are adequately conservative. The results of this systematic assessment of seismic design response factors apply to a wide variety of contemporary concrete wall buildings with various characteristics.

Keywords: concrete wall buildings; seismic design; response factors; fiber-based modeling; inelastic pushover analysis; incremental dynamic analysis

1 Introduction

Seismic design response factors, namely overstrength (Ω_o), force reduction factor (R) and deflection amplification factors (C_d), are used to reduce the seismic forces and amplify deformations to arrive at cost-effective and safe designs. For instance, increasing the R factor from 4 to 5 reduces the design force, and potentially the cost of the structural system, up to about 25%. On the other hand, restricting the R and C_d values is necessary to prevent excessive inelastic deformations and loss of life, particularly in the event of a major earthquake. Seismic design response factors introduced in seismic codes do not offer a uniform margin of safety and cost effectiveness for different seismic regions to account for the diversity of structural systems, construction practices and quality control (FEMA, 2009b; Mwafy and Elnashai, 2002). Moreover, modern design codes do not fully address all structural systems currently used in different parts of the world. The capability of these systems to meet the intended seismic design objectives is also not adequately understood.

The number of earthquake reported within or near to the UAE is on increase. About three events per year were recorded between 2000-2006 compared to 0.65 events per year from 1924-1999 (Aldama-Bustos *et al*., 2009). The UAE seismicity is characterized by earthquakes that originate from both distant and local seismic sources (Mwafy *et al*., 2006). Given these seismological features along with the large investment in the construction sector, unified building codes have been recently enforced in Abu Dhabi, UAE. The new design provisions are based on the International Building Code (ICC, 2009) with minimum amendments in the initial implementation phase. This reflects the pressing need for verifying the seismic design response factors using reliable assessment methodologies.

The objective of this study is to verify response factors used in the seismic design of reinforced concrete (RC) multi-story buildings through inelastic pushover analysis (IPA) and incremental dynamic collapse analysis (IDCA). The methodology employed to design and develop verified fiber-based simulation models for five example structures (20 to 60 stories high) is presented and IPA and IDCA results are used to verify the design response factors.

2 Verification approach for seismic design response factors

The methodology adopted in previous studies on the

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assessment of seismic design response factors may be grouped into two main categories based on the analytical models considered: (i) those using single degree of freedom "SDOF" models (e.g., Borzi and Elnashai, 2000), and (ii) studies focused on more realistic multi-degrees of freedom (MDOF) systems representing actual structures (e.g., Mwafy and Elnashai, 2002). Although the second group results in a more reliable estimation of the seismic design response factors, few studies matching this category are available in the literature, particularly for multi-story shear wall buildings. The present study belongs to the second category.

Structures are designed for forces consistent with the yield limit state, while collapse may occur under earthquakes with a spectrum higher than the elastic design spectrum. Moreover, the 'significant yield' in well-designed RC buildings is generally observed at a high intensity level compared with the yield level implied in design. This is confirmed from the results presented hereafter. FEMA P-750 (FEMA, 2009a) confirms that the first 'significant yield' of adequately designed structures may occur at lateral load levels that are 30 to 100 percent higher than the prescribed design forces. Mwafy and Elnashai (2002) therefore suggested the following definition for evaluating the R factor of a particular structure under the effect of a specific earthquake:

$$R = \left[(a_g)_c/(a_g)_y\right]\cdot\Omega_y \tag{1}$$

where $(a_g)_c$ is the peak ground accelerations (PGA) of the collapse earthquake, $(a_g)_y$ is the PGA at the first indication of significant yield and Ω_y is defined in the present study as the ratio of strength at significant yield to design strength. For inherent damping of 5 percent of critical and fundamental periods greater than the transition period, T_s, which are applicable to the reference structures, C_d is considered equal to R (ASCE, 2006a). Equating C_d to R is based on Newmark's equal displacement rule, which assumes that the elastic and inelastic displacements are approximately equal. Extensive IDCAs are therefore performed for the reference structures by scaling and applying each input ground motion used in the current study up to the attainment of collapse. Hence, the PGAs causing the first indication of significant yield and collapse are determined. The wide ranges of structures and input ground motions employed in the present investigation are presented in the following sections.

3 Selection and design of reference structures

RC high-rise buildings with cast-in-situ flat slabs and shear walls are widespread in different parts of the world. Shear walls are the main gravity and lateral force-resisting elements. The foundations consist of a mat supported on a system of piles. The buildings were designed in highly populated areas of the UAE to resist seismic forces according to the Uniform Building Code (ICBO, 1997) or the International Building Code (ICC, 2009) with a design PGA of 0.15g for 10% probability of exceedance in 50 years. Five RC buildings, ranging from 20 to 60 stories, were selected to represent a wide range of regular multi-story buildings in this region. The buildings were designed and detailed for the purpose of the current study. The five buildings have the same layout shown in Fig. 1. Each building consists of two basements, a ground story, and a number of typical stories. The typical height of all stories is 3.2 m except for the ground story, which is 4.5 m. The permanent loads used in design include the self-weight in addition to a superimposed dead load of 4.0 kN/m^2. The live load is 2.0 kN/m^2 except for stairs and exit ways, which is 4.8 kN/m^2.

Wind loads were estimated according to ASCE/SEI 7-05 (ASCE, 2006a) based on a basic wind speed of 45 m/s (100 mph) and an exposure category C. Seismic design loads were also calculated according to the ASCE/SEI 7-05 (ASCE, 2006a) code. Note that ASCE/

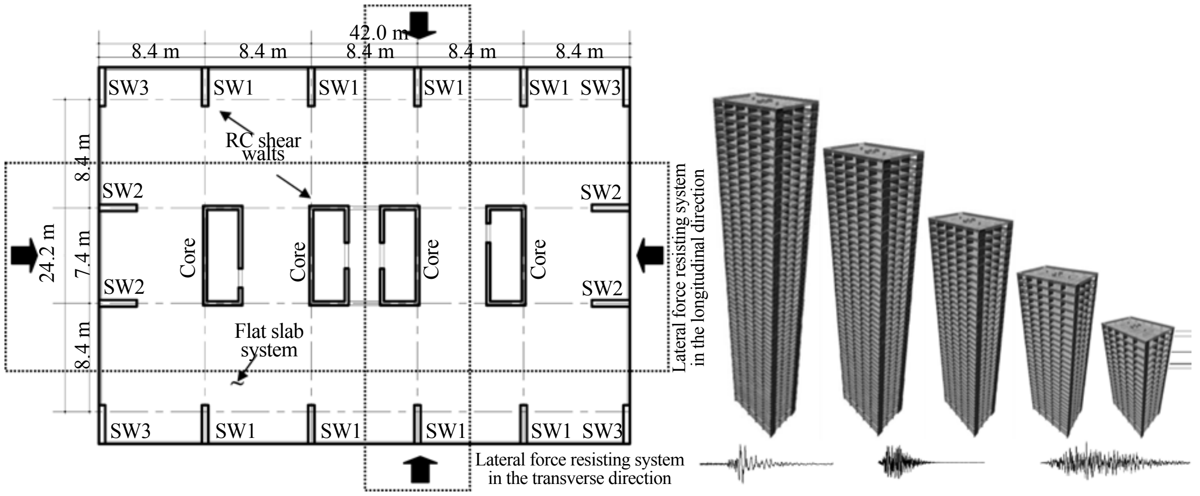

Fig. 1 Building layout and 3D analytical models

SEI 7-05 requires the use of spectral seismic design maps to quantify seismic hazards on the basis of contour lines. These maps were not available for the UAE at the time of the present study. Equivalent mapped spectral values were therefore estimated based on a site class D (stiff soil) and used to calculate the seismic loads according to ASCE/SEI 7-05 (ASCE, 2006a). Selection of suitable mapped spectral values was achieved by matching the ASCE/SEI 7-05 (ASCE, 2006a) elastic design spectrum with the corresponding UBC (ICBO, 1997) spectrum, as shown in Fig. 2. The seismic zone 2A was recommended in a number of hazard assessment studies for Dubai, and hence is used with a site class S_D (stiff soil profile) to obtain the UBC spectrum (Mwafy *et al*., 2006).

Table 1 shows the characteristics of the five reference structures along with the base shear-to-weight ratio (V/W) calculated using the ASCE/SEI 7-05 provisions. Seismic loads generally govern the design of the five reference structures compared with wind loads. A lower bound on the base shear obtained from modal response spectrum analysis is adopted as required by ASCE/SEI 7-05. Note that the V/W ratio significantly decreases as building height increases. The difference between the design base shear of the five buildings is, however, limited to about 30% due to the significant increase in weight with increasing building height.

Detailed three-dimensional (3D) models are developed for the five reference buildings using the structural analysis and design program ETABS (CSI, 2008), which is commonly used by structural engineers in the building industry since it can handle large building models. The five buildings were proportioned and detailed according to various load combinations and design provisions recommended by the ACI building code (ACI, 2005). Yield strength of reinforcing steel is 460 MPa. Constant concrete cylinder strength of 36 MPa (cube strength of 45 MPa) is used for floor slabs and beams, while it varies along the height of shear walls from 36 MPa to 56 MPa. The cross-sections of walls and the corresponding reinforcement also vary along the building height. The floor slab systems consist of 0.28 m cast-in-place flat slabs with a beam at the perimeter.

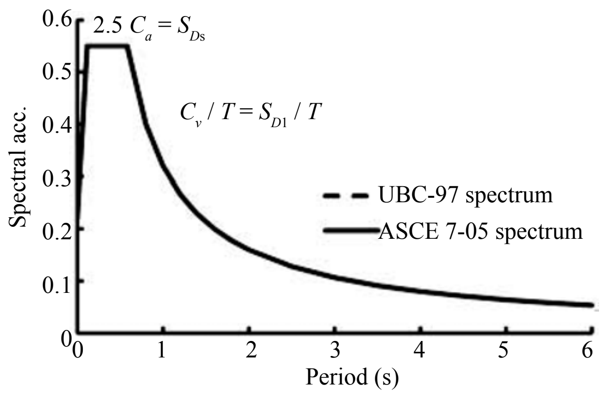

Fig. 2 Matching the ASCE 7-05 elastic design spectrum to UBC-97

Table 1 Characteristics of the designed buildings

Building reference	Total height (m)	Total design weight, W (MN)	Response modification coefficient, R	System overstrength factor, Ω_o	Deflection amplification factor, C_d	Design base shear-to-weight ratio (V/W)
20SB	65.3	354				0.0425
30SB	97.3	539				0.0319
40SB	129.3	743	4.0	2.5	4.0	0.0257
50SB	161.3	945				0.0218
60SB	193.3	1149				0.0190

Figure 1 shows the 3D models employed in the design of the buildings. Table 2 summarizes the sizes and concrete strength (f_c') of vertical structural members. Additional design information of the five reference structures, including detailed structural drawings, are presented elsewhere (Mwafy, 2009).

4 Analytical modeling for inelastic analysis

The IPAs and IDCAs for the reference structures were performed using ZEUS-NL (Elnashai *et al*., 2010), which is a modern inelastic analysis platform utilizing the fiber modeling approach. The adopted modeling approach enables monitoring the stress-strain response of each frame element over two Gauss sections through the integration of the nonlinear stress-strain response of different fibers in which the section is subdivided. Three cubic elasto-plastic frame elements capable of representing the spread of yielding and cracking were used to model each structural member. This enables modeling three different arrangements of reinforcing steel along the length of each structural element as specified in design. Two rigid arms are also utilized to connect the slab/beam ends with the framing wall (the length between the centerline and the face of the vertical element). RC rectangular, *T*, flexural wall, hollow core and steel rectangular cross sections are used from the ZEUS-NL library to model slabs, connecting beams, shear walls, cores and rigid arms, respectively.

Table 2 Sizes of vertical structural members and concrete cylinder strength (f_c') of the reference structures

Building	Member	Characteristics	Story number						
			2nd bas.–3	4–8	9–18	19–28	29–38	39–48	49-58
20SB	Shear walls	Thickness × Length	350×3000	350×3000	300×3000				
		f_c'	36	36	36				
	Core	Thickness	300	250	200				
		f_c'	36	36	36				
30SB	Shear walls	Thickness × Length	350×4000	350×4000	300×4000	250×4000			
		f_c'	40	40	36	36			
	Core	Thickness	300	300	250	200			
		f_c'	40	40	40	36			
40SB	Shear walls	Thickness × Length	400×5000	400×5000	350×5000	300×5000	250×5000		
		f_c'	48	48	40	36	36		
	Core	Thickness	350	350	300	250	200		
		f_c'	48	48	40	40	36		
50SB	Shear walls	Thickness × Length	450×5000	450×5000	400×5000	350×5000	300×5000	250×5000	
		f_c'	48	48	40	40	36	36	
	Core	Thickness	400	400	350	300	250	200	
		f_c'	48	48	40	40	36	36	
60SB	Shear walls	Thickness × Length	500×5000	500×5000	450×5000	400×5000	350×5000	300×5000	250×5000
		f_c'	56	56	48	40	40	36	36
	Core	Thickness	450	450	400	350	300	250	200
		f_c'	56	56	48	40	40	36	36

Note: concrete cylinder strength is in MPa and dimensions are in mm

The concrete behavior is represented by using a uniaxial constant confinement concrete model, while the reinforcing steel behavior is simulated by a bilinear elasto-plastic model, as shown in Fig. 3 (Elnashai *et al*., 2010). Actual rather than nominal material strengths were employed in the ZEUS-NL models for assessment of the reference structures. A normal distribution is adopted for both the concrete and steel strength values with coefficients of variation of 12% and 6%, respectively.

The 3D modeling and inelastic response history analysis of high-rise buildings are computationally demanding, particularly with the wide range of buildings and input ground motions employed in the present study. A 2D idealization is therefore adopted for the IPAs and IDCAs carried out to assess the response factors of the reference structures. It is assumed that four lateral-force-resisting systems are in the transverse direction of each building. Each of these framing systems consists of two external shear walls (SW1) and an internal core, as shown in Fig. 1. Note that only one framing system effectively resists the lateral forces in the longitudinal direction. The latter system consists of all internal cores and the shear walls SW2. Other walls are conservatively assumed to not participate in resisting the lateral forces. Therefore, in total, ten framing systems are modeled using ZEUS-

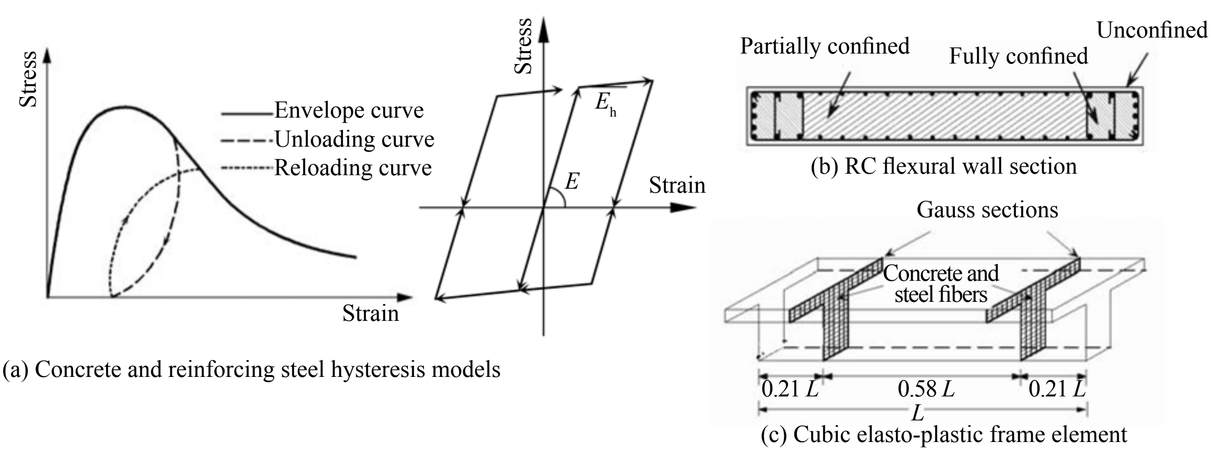

Fig. 3 Ingredients of the ZEUS-NL fiber-based models: (a) material models, (b) sample of concrete sections, and (c) elasto-plastic frame element

NL to represent the lateral-force-resisting systems of the five buildings (one model for each building in both the longitudinal and transverse directions).

Hysteretic damping, which is responsible for the dissipation of the majority of energy introduced by the earthquake action, is accounted for in the inelastic fiber formulation of the inelastic frame elements. A relatively small quantity of stiffness-proportional damping (0.75%) is also added to the inelastic models based on comparisons between the response of the five investigated buildings at different damping levels as well as the information collected from the literature for multi-story buildings (e.g., Priestley and Grant, 2005; Willford *et al*., 2008). Table 3 shows a comparison between the non-cracked periods of vibration of the five buildings in the transverse direction from the ETABS 3D and the ZEUS-NL 2D models. The results obtained from the ZEUS-NL models indicate that the periods of the structure are compatible with those obtained from ETABS. The difference between the 3D and the 2D models is less than 20%, which is due to employing actual material strength in the ZEUS-NL models rather than the nominal values used in the 3D design models. Also, the rebars, which are accurately represented in the ZEUS-NL models, increase the structure stiffness. The free vibration results validate the ZEUS-NL analytical models used in the inelastic analyses presented in subsequent sections.

5 Estimation of lateral capacity and overstrength

5.1 Lateral capacity

Although several improvements have been recently suggested to advance the pushover technique, newly developed procedures such as the adaptive and modal pushover procedures still do not guarantee satisfactory results when compared with IDCA with increasingly erratic input ground motions and structural irregularity (Huang and Kuang, 2010). The more enhancements involved in new procedures may also have an impact on simplicity. Although the conventional pushover procedure is more applicable to structures mainly vibrating in their fundamental mode, it has also been proven valuable for capacity estimates of multi-story buildings and highway bridges (e.g., Mwafy *et al*., 2006; Mwafy *et al*., 2007). Nonlinear static analyses are performed in the present study to evaluate the structural overstrength (Ω_o) and first yield overstrength (Ω_y) factors of the reference structures. Predefined lateral load patterns are applied incrementally in a step-wise manner on the ten framing systems, which represent the five reference structures in the longitudinal and transverse directions. Two lateral load patterns are employed in IPA (e.g., ASCE, 2006b). The first pattern is a uniform distribution, representing lateral forces that are proportional with the mass. The second one is an inverted triangular distribution, resembling the first mode shape. Due to the significant influence of higher modes of vibration on the seismic response of high-rise buildings, Mwafy *el al.* (2006) investigated a third lateral load distribution representing the second mode shape of a 54-story building. The study concluded that the uniform lateral load distribution can be used to obtain a conservative estimate of initial stiffness and lateral capacity of high-rise buildings. This conclusion is in line with the findings from other studies carried out on high-rise buildings (e.g., Ventura and Ding, 2000). Mwafy and Elnashai (2001) also concluded that the uniformly distributed load gives a better prediction of the ultimate strength of structures influenced by higher modes compared with the inverted triangular load. Based on these conclusions, it was decided to use the uniform lateral load to conservatively estimate the lateral capacity of the reference structures. It is important to note that IPA was not employed in the present study to estimate seismic demands such as the interstory drift, story shear or the plastic hinge distributions. These demands are exclusively predicted using IDCAs, as explained below. The author suggests that more effort is needed to improve IPA for the demand prediction of high-rise buildings, maintaining the simplicity which is the most advantageous feature of this technique when compared with IDCA.

The response of the 3D models developed for the design of the reference structures indicated a comparable lateral stiffness in their two orthogonal horizontal directions. Figure 4 shows sample results from IPAs of the 20- and 30-story buildings in the longitudinal and transverse directions. The first indication of yielding in walls and horizontal members as well the global yielding

Table 3 Comparison between calculated vibration periods of the reference structures in transverse direction by using ZEUS-NL and ETABS models along with those used in the design

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Building reference		20SB		30SB		40SB		50SB		60SB	
Analysis models		ETABS	ZEUS	ETABS	ZEUS	ETABS	ZEUS	ETABS	ZEUS	ETABS	ZEUS
Mode No.	1st	1.78	1.55	2.89	2.56	3.82	3.69	5.14	5.10	6.55	6.06
	2nd	0.44	0.35	0.76	0.65	1.06	0.98	1.44	1.38	1.84	1.68
	3rd	0.19	0.14	0.34	0.28	0.49	0.44	0.67	0.63	0.87	0.77
Design periods (ASCE 7-05)		1.57		2.11		2.62		3.09		3.54	

and ultimate capacity are also shown on the graphs. The global yield is estimated from an elastic-perfectly plastic idealization of the real capacity envelop, where the initial stiffness is evaluated as the secant stiffness at 75% of the ultimate strength, as shown from Figs. 4 and 5. The starting point of the post-elastic branch is considered as the global yield threshold. The results presented in Fig. 4 confirm that the differences in response between the transverse and longitudinal directions of the buildings are insignificant. This observation suggests that the number of incremental response history analyses can be reduced by focusing on one direction. Although ten framing systems were modeled using ZEUS-NL to represent the lateral force-resisting-systems in the longitudinal and transverse directions, the five frames representing the structural systems in the transverse direction are employed in subsequent sections to estimate the seismic design response factors based on the results presented above. The number of frame elements of the selected models is 44% of those used to idealize the reference structures in the longitudinal direction, and hence a considerable savings in time and computer resources is achieved by employing the analytical models that represent the transverse direction in the extensive IDCAs presented hereafter.

It is shown from Figs. 4 and 5 that the first yield gradually shifts from walls to horizontal members as the building height increases. The behavior of the 20-and 30-story buildings is governed by localized yielding at the base of shear walls. First yielding in horizontal members of these buildings is observed at higher lateral load levels compared with yielding in shear walls, particularly for the 20-story building. This is unlike the response of other structures. Clearly, shifting the first indication of yield to be at a higher base shear level and in slabs/beams rather than in shear walls, as shown from the responses of the 40- to 60-story structures, is more favorable.

5.2 Overstrength

The seismic design forces of the five reference structures are depicted in Figs. 4 and 5 (also refer to Table 1). Note that the structural overstrength (Ω_o), defined as the ratio of the ultimate capacity to the seismic design force, is generally higher than the reserve strength intended by the design code. This is mainly attributed to the contribution of several sources to structural overstrength (FEMA, 2009a). Figure 6 depicts a comparison between the structural overstrength (Ω_o) of the five reference structures in the longitudinal and transverse directions along with the value recommended by the design code (ASCE, 2006a) for RC shear wall structures. The results indicate that Ω_o slightly decreases

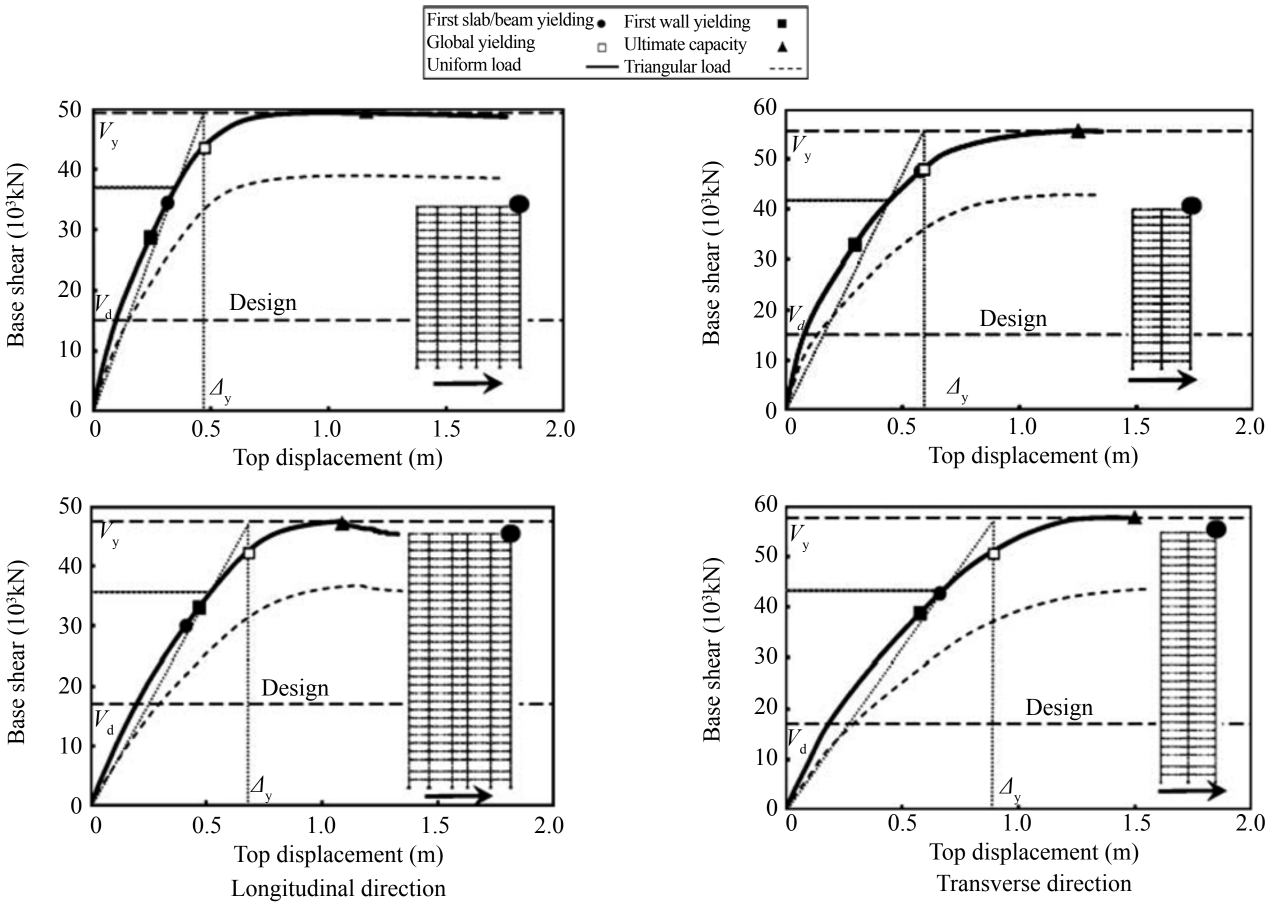


Fig. 4 Comparison between the capacity versus demand of the 20-story and 30-story buildings in the longitudinal and transverse directions

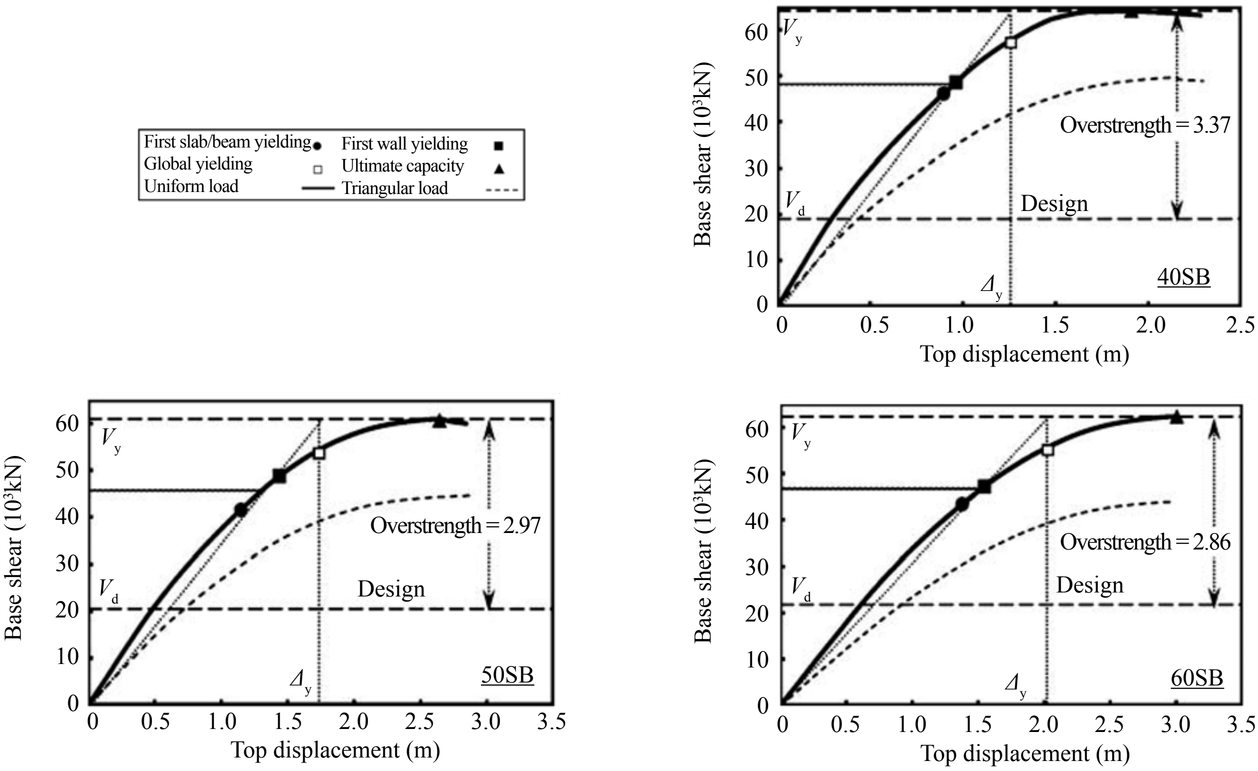


Fig. 5 Capacity envelops of 40, 50 and 60-story structures in the transverse direction

as building height (or period) increases. The calculated overstrength values in the transverse direction are also higher than those in the longitudinal direction due to the higher capacity of the former direction, as shown in Fig. 4. The lowest calculated reserve strength is for the 60-story building, while the highest value is for the 20-story structure. It is noteworthy that the IPA conservatism increases as the building height increases.

Furthermore, IDCAs are carried out herein to validate the structural overstrength (Ω_o) and the first yield overstrength (Ω_y) factors of the reference structures, as discussed below. The calculated Ω_o factors using the median base shear of the Set 1 earthquake records (shown in Fig. 8 and discussed in the next section) at the collapse limit state (2.5% interstory drift) are 7.4, 7.6, 9.3, 9.7, and 13.0 for the 20- to 60-story buildings, respectively. The calculated values using the Set 2 ground motions are even higher. The IPA and IDCA results confirm that the lowest overstrength factor is 2.5, which is consistent with the value recommended by the design code. It is clear from this discussion that IPAs are used in the present study along with the more reliable IDCA to obtain conservative estimates of the overstrength factors from both analysis procedures. The results indicate that it is possible to increase the design overstrength factors by 20% (Ω_o = 3.0) for the 20-story structure. Seismic forces generally play a less important role in the determination of cross-section sizes and reinforcement than gravity loads for low- and medium-rise buildings. This is also shown from the trend of the results presented in Fig. 6.

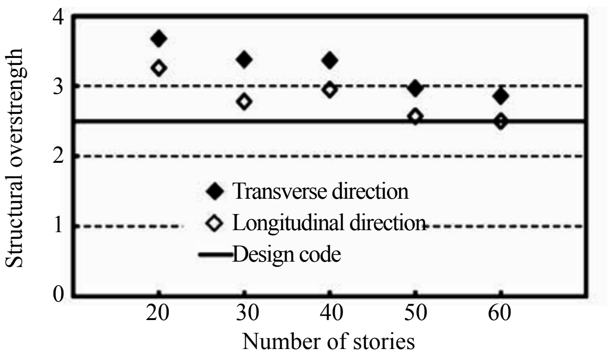


Fig. 6 Comparison between the calculated and code recommended overstrength (Ω_o) of RC shear wall structures ranging from 20 to 60 stories

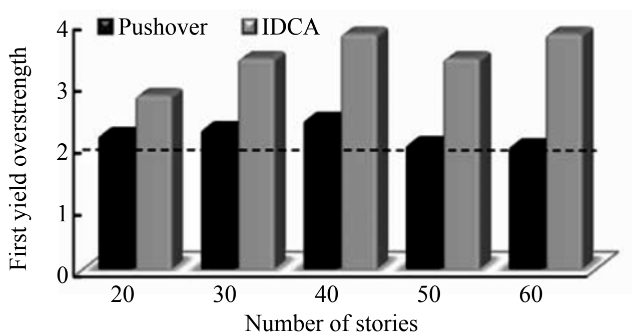


Fig. 7 Calculated overstrength (Ω_y) of RC shear wall structures ranging from 20 to 60 stories by IPA and IDCA

Therefore, a conservative system overstrength factor of 3.0 can be used for shear wall buildings lower than 20 stories. On the other hand, the results shown in Fig. 6 validate the code recommended overstrength factor for higher RC shear wall structures (30 to 60 stories). Note that the actual overstrength should be higher than the values obtained from IPA due to the beneficial effects of some parameters that contribute to overstrength such as nonstructural elements.

Figures 4 and 5 show that the first indication of yielding is significantly higher than the force level used in design, which suggests that the significant yield level can be safely reduced to the design level. The overstrength (Ω_y), which is defined in Eq. (1) as the ratio of strength at significant yield to design strength, is depicted in Fig. 7. The Ω_y factors obtained from IPA and IDCA do not show the trend observed in Fig. 6 for the Ω_o factors as the building height increases. The minimum calculated Ω_y factor is above 2.0, which is conservatively adopted to estimate the force reduction factor (R) of the reference structures using Eq. (1).

6 Incremental dynamic collapse analysis

6.1 Input ground motions

Two sets of natural and synthetically generated records were selected to verify the seismic design response factors of the five reference structures. The selected records represent two distinct seismic scenarios: (i) severe distant earthquakes of magnitude 7.4 with 100 km epicentral distance and (ii) moderate events of magnitude 6.0 and a distance to source of 10 km. These two seismic scenarios were recommended in a previous seismic hazard assessment study for Dubai (Mwafy *et al*., 2006). This study also recommended uniform hazard spectra (UHS) and two sets of artificial time series simulated using available geophysical information and engineering models to represent these two seismic scenarios. Ten synthetic accelerograms (five representing severe distant earthquakes, R6-R10, and five for moderate close events, R16-R20) were selected from the abovementioned study. Due to the lack of available real records for the UAE, ten natural accelerograms were selected from the Pacific Earthquake Engineering Research Center (PEER, 2009) and the European strong-motion (Ambraseys *et al*., 2004) databases. Five of the selected natural records (R1-R5) represent severe distant events, while moderate close earthquakes are represented by R11-R15. The natural records were selected based on their distance to source and spectral amplification to fit the UHS for 10% probability of exceedance in 50 years.

Tso *et al.* (1992) studied the significance of the a/v ratio as a measure for the frequency content, magnitude and epicentral distance of the ground motion. It was suggested that the low a/v ratios (< 0.8 g/m s^{-1}) signify earthquakes with low predominant frequencies, broad response spectra, medium-to-high magnitude, long epicentral distance, long duration and site period. On the other hand, high a/v (> 1.2 g/m s^{-1}) ratios represent high predominant frequencies, narrow band spectra, small-to-medium magnitude, short epicentral distance, short duration and site period. The selected input ground motions correlate well with the abovementioned two ranges. The elastic response spectra of the 20 selected natural and artificial records are shown in Fig. 8 with a PGA of 0.16g. This intensity level represents the design PGA recommended for Dubai for a 10% probability of exceedance in 50 years. The UHS for Dubai for 10% probability of exceedance in 50 years is also shown in Fig. 8. It is clear that the median spectrum of the two selected ground motion sets adequately represents the two seismic scenarios recommended for Dubai (Mwafy *et al*., 2006).

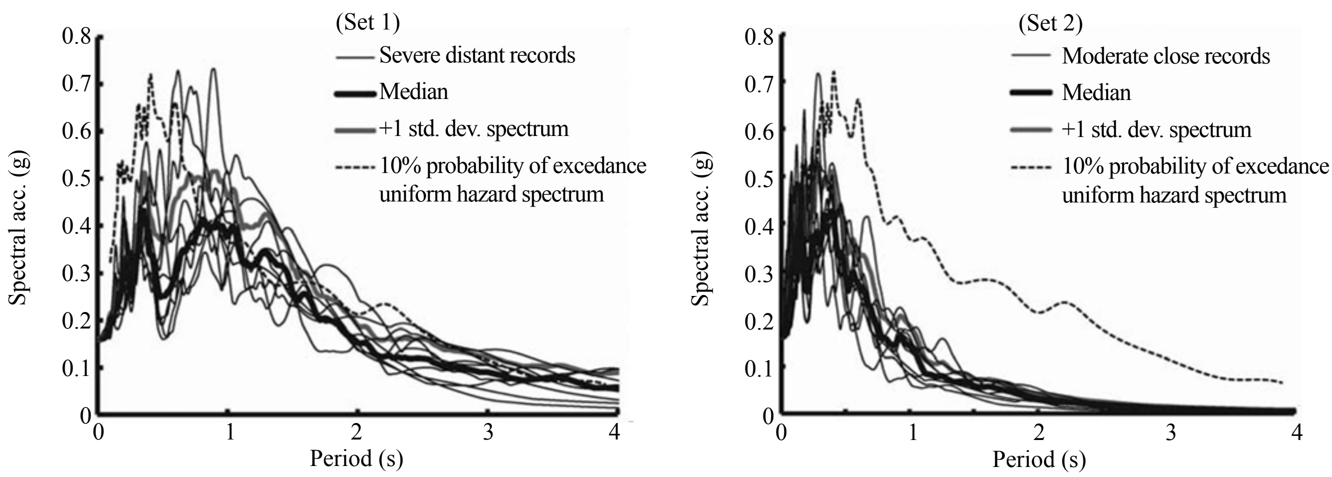


Fig. 8 Response spectra of 20 natural and artificial records representing severe distant (left) and moderate proximate (right) events along with the median spectrum, spectrum representing one standard deviation above the median, and uniform hazard spectrum for Dubai for 10% probability of exceedance in 50 years

6.2 Performance criteria

First yield is typically assumed when the strain in the main longitudinal tensile reinforcement exceeds the steel yield strain. An RC structure reaches ‘significant yield’ when one of its most highly stressed sections reaches its yield strength. Four performance levels are described by the NEHRP Recommended Provisions (FEMA, 2009a). These are termed the operational, immediate occupancy, life safety, and collapse prevention levels. The structure sustains nearly complete damage at the collapse prevention performance level. The lateral-force-resisting system loses most of its original stiffness and strength and a little margin against collapse remains. A reliable collapse criterion consistent with the definition of the collapse prevention performance level is therefore needed due its significant role in the determination of the R factors, as shown in Eq. (1).

Interstory drift ratio (IDR), which is considered in the present study as the primary collapse criterion, ranging from 2.0% to 2.5% is typically employed by seismic codes and guidelines to define the collapse prevention (or near collapse) limit state. At values in excess of this limit, P-Δ effects are significant and lead to reduced lateral resistance, leading to failure. ASCE/SEI 41-06 (2006b) considers the collapse prevention criterion in concrete wall structures, which corresponds to extensive crushing and buckling of reinforcement in walls and coupling beams, at IDR of 2%. The SEAOC blue book (1999) recommends an interstory drift ratio of 2.1% for concrete shear walls at a performance level SP4, which represents the near collapse performance level. The latter guideline also suggests an interstory drift ratio greater than 2.5% for the collapse performance level. Moreover, an IDR collapse limit of 2.5 for ductile concrete wall structures has been recommended in a number of previous studies (e.g., Ghobarah, 2004). Note that the code recommended drift limits tend to be on the conservative side and do not necessarily imply actual collapse. An interstory drift limit of 2.5% is therefore adopted in the present study based on the values recommended by SEAOC (1999) and Ghobarah (2004). The selected criterion is sufficient to restrict P-Δ effects and limit the extensive structural damage in concrete wall structures, particularly those designed to modern seismic provisions (ACI, 2005; ICC, 2009).

To verify the seismic design response factors using IDCAs, the selected input ground motions (refer to Fig. 8) are scaled to different intensity (PGA) levels. The analysis is carried out for the five reference structures up to the satisfaction of the two performance levels discussed above, namely the yield and collapse limit states. A large number of inelastic response history analyses are conducted for the reference structures

Table 4 Summary of IDCAs at the first indication of yield

		20SB		30SB		40SB		50SB		60SB	
		PGA	IDR	PGA	IDR	PGA	IDR	PGA	IDR	PGA	IDR
Set 1	Max.	0.40	0.96	0.72	0.98	0.48	1.12	0.48	1.15	0.32	0.90
	Min.	0.12	0.67	0.16	0.74	0.24	0.83	0.16	0.62	0.24	0.65
	Median	0.20	0.79	0.30	0.88	0.32	0.98	0.32	0.86	0.24	0.75
Set 2	Max.	1.92	0.79	1.92	1.08	2.56	1.05	2.56	0.96	2.88	1.01
	Min.	0.56	0.60	0.96	0.78	0.96	0.71	1.28	0.72	0.96	0.66
	Median	0.88	0.68	1.44	0.96	1.28	0.84	1.44	0.80	1.92	0.84
Median of 20 records		0.48	0.76	0.84	0.93	0.72	0.92	0.88	0.83	0.64	0.77

PGA: Peak ground acceleration (g); IDR: Interstory drift ratio (%)

Table 5 Summary of IDCAs at the first indication of collapse

		20SB		30SB		40SB		50SB		60SB	
		PGA	IDR	PGA	IDR	PGA	IDR	PGA	IDR	PGA	IDR
Set 1	Max.	1.46	2.55	1.40	2.55	1.28	2.54	1.92	2.53	1.76	2.51
	Min.	0.47	2.47	0.57	2.50	0.70	2.45	0.78	2.44	0.59	2.44
	Median	0.75	2.50	0.95	2.52	0.94	2.50	0.93	2.50	1.09	2.50
Set 2	Max.	5.35	2.50	11.07	2.51	11.20	2.55	9.49	2.50	16.34	2.55
	Min.	2.47	2.49	3.40	2.49	3.57	2.47	4.30	2.47	5.17	2.44
	Median	3.58	2.50	4.27	2.50	4.64	2.50	6.43	2.50	7.83	2.50
Median of 20 records		1.97	2.50	2.40	2.50	2.42	2.50	3.11	2.50	3.47	2.50

PGA: Peak ground acceleration (g); IDR: Interstory drift ratio (%)

(about 1500 runs). In addition to monitoring global response parameters such as top displacement, base shear and IDRs from the response history analyses, the formation of plastic hinges in different structural members are screened during the entire multi-step analyses to provide the required indication of the level of structural damage.

7 Evaluation of seismic design response factors, *R* and C_d

The results obtained from the IDCAs are used to estimate the seismic design response factors, R and C_d, using Eq. (1). Tables 4 and 5 summarize the IDCA results at the yield and collapse limit states, respectively. For the sake of brevity, only the maximum, minimum and median results obtained for Set 1 and Set 2 ground motions are presented. The results at the design and at twice the design earthquake intensity confirmed a significant difference between the responses of the reference structures under the effect of the two ground motion sets employed in the present study. Consequently, the IDCA results are presented separately for the two sets of records along with the median results from the twenty input ground motions.

Figure 9 summarizes the IDCA results obtained for the five reference structures under the effect of the two sets of input ground motions. The collapse-to-yield PGA and interstory drift ratios are also presented in this figure. As explained in Eq. (1), the collapse-to-yield PGA ratios are used along with the calculated overstrength (Ω_y) to estimate the R factors. It is clear that although yield and collapse are calculated at significantly higher PGA values under the effect of Set 2 ground motions when compared with Set 1, the calculated collapse-to-yield PGA ratios are comparable from the two sets of records. This ratio also does not show a clear trend as

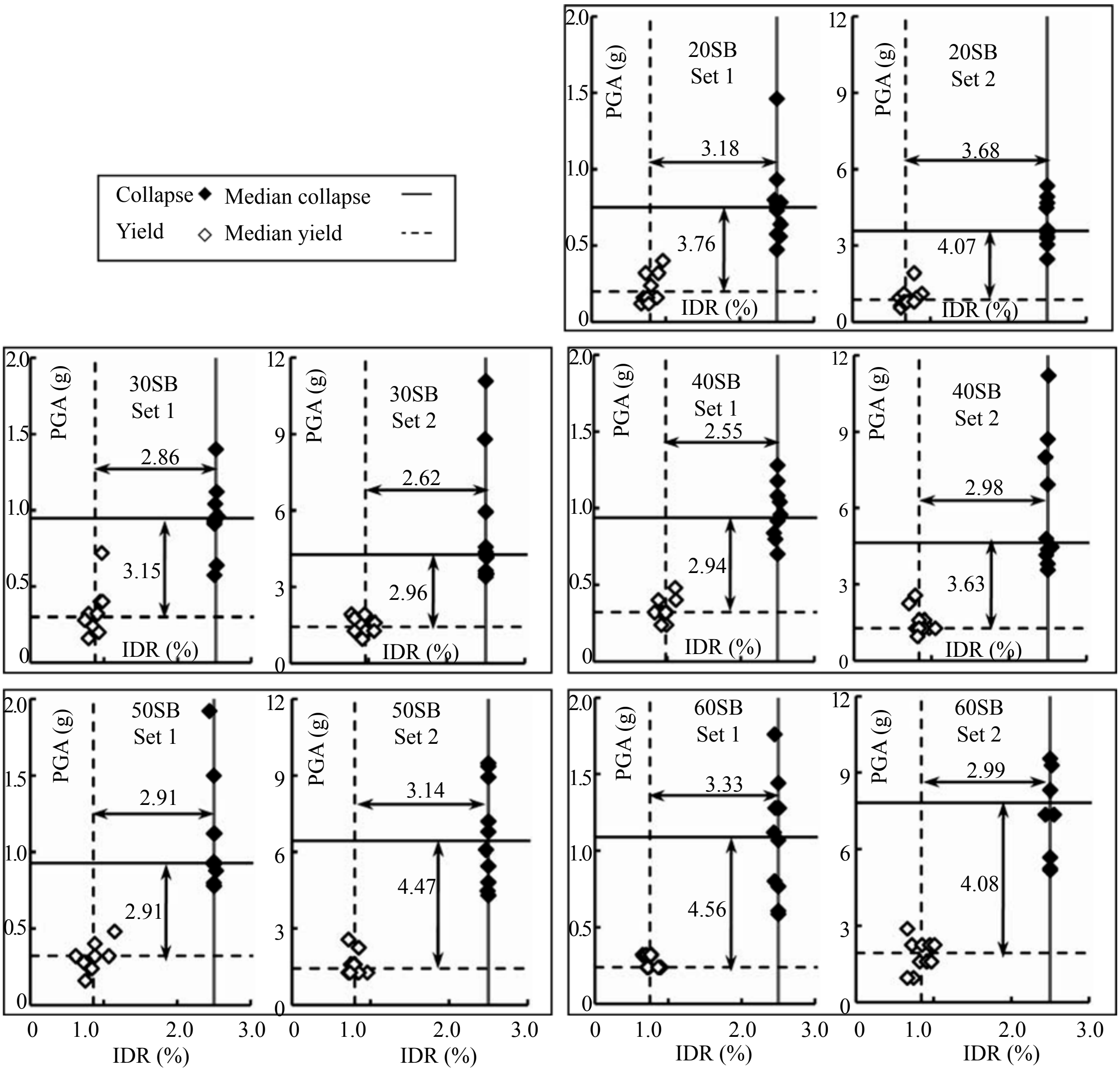
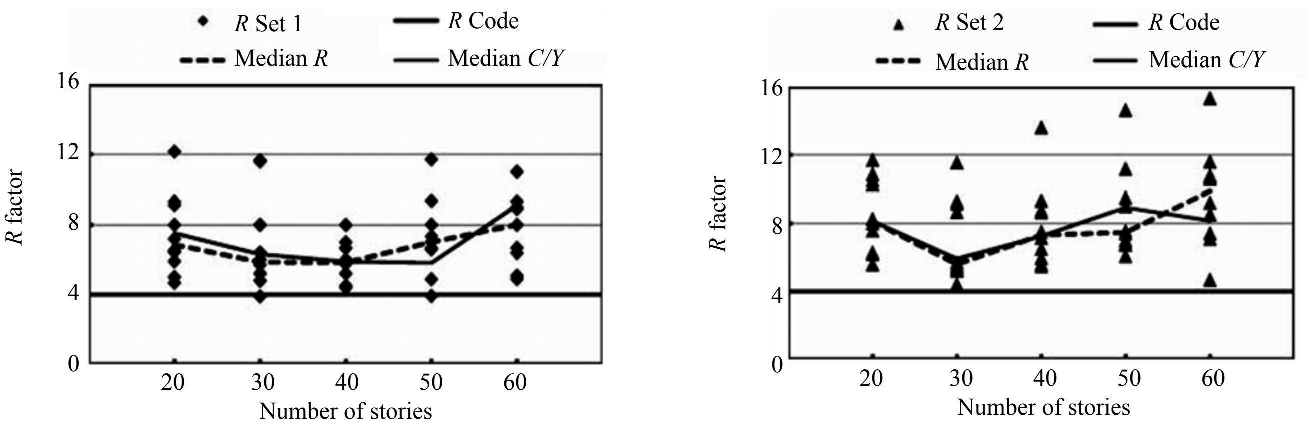


Fig. 9 IDCA results at yield and collapse along with collapse-to-yield PGA ratios and IDRs

the building height increases. Also, the PGA values that cause yield are not affected by the increased building height (or period), as shown in Table 4. However, Table 5 confirms the relationship between the building height and the PGA at collapse. Clearly, collapse is observed at higher PGAs for higher buildings. This implies that the impact of the two earthquake scenarios decreases as the building height increases. This observation is more pronounced under the effect of Set 2 due to the insignificant contribution of the fundamental mode of vibration as the building height increases under the effect of these ground motions, which have low amplifications in the long period range.

It is shown from Fig. 9 that the collapse-to-yield IDRs are always lower or equal to the collapse-to-yield PGA ratios. The difference between these two ratios is less than 20% for the 20 to 40-story buildings, while the difference exceeds 40% for the 50 and 60-story structures. This reflects the adequate conservatism in equating the deflection amplification factor, C_d, with the R factor. Figure 10 shows a summary of the force reduction factors evaluated using the two sets of earthquake records employed in the present study. The design R factors (R Code) and median values obtained from different input ground motions as well as those calculated from the median collapse and median yield PGA shown in Fig. 9 are also presented. It is clear from Fig. 10 that the median R factors of the reference structures are significantly higher than the values suggested by the design code (ASCE, 2006a). The results reflect the high safety margins of the design code regarding concrete wall structures. Mwafy and Elnashai (2002) calculated the R factors of a set of buildings designed to Eurocodes and ranging from 8 to 12 stories. The calculated-to-design R values were higher for short period buildings compared with longer period structures. The results presented in Figs. 9 and 10 do not follow the same trend since the estimated R factors are comparable for the five reference structures. This is due to the comparable overstrength and collapse-to-yield PGA ratios of the concrete wall structures investigated in this study.



(Note: Median R is the median of the R factors obtained from different input ground motions, while the median C/Y is based on the median collapse and median yield PGA, as indicated in Fig. 9)

Fig. 10 Seismic design response factors of concrete wall buildings obtained from ground motions Set 1 (left) and Set 2 (right)

The calculated seismic design response factors using a wide range of input ground motions representing two distinctive seismic scenarios confirm the less dependence of this factor on the ground motions used in analysis, as shown from Figs. 9 and 10. This observation emphasizes the need to employ a diverse and wide range of natural and artificial records representing all seismic scenarios anticipated to reliably verify seismic response. The seismic design response factors, namely the Ω_o, R and C_d factors, confirm the satisfactory safety margins and design factors adopted by the IBC and ASCE/SEI 7-05 (ASCE, 2006a; ICC, 2009) for the design of multi-story concrete wall buildings.

8 Conclusions

This study focused on verifying the seismic design response factors, namely the overstrength (Ω_o), force reduction (R) and deflection amplification (C_d) factors, of multi-story buildings with shear walls since they are widespread in different parts of the world and represent concentrated economic and human assets. The approach used to design and develop verified fiber-based simulation models for five reference structures, which represent a wide range of buildings, and the methodology adopted to verify the seismic design response factors using inelastic pushover analyses (IPAs) and incremental dynamic collapse analyses (IDCAs) were discussed in this paper. The capacity curves of the reference structures were traced to evaluate the structural overstrength. First indication of yielding gradually shifted from walls to horizontal structural members as the building height increased. The Ω_o factor was equal to or higher than the reserve strength intended by the design code (2.5). Overstrength increased as the building height (or period) decreased, which reflects the possibility to increase Ω_o by 20% for the 20-story structures. The overstrength factor calculated using IDCAs was significantly higher than estimated from IPAs.

IDCA results provided insight into the inelastic seismic response of the reference structures and enabled the seismic response factors to be assessed under the effect of two sets of ground motions representing two different seismic scenarios relevant to the studied area. Collapse was observed at a higher PGA level as the building height increased. This implies a higher seismic risk for medium-rise buildings compared with high-rise structures due to the lower contribution of the fundamental mode of vibration to the seismic response of the latter buildings, particularly under the effect of moderate close earthquakes. The collapse-to-yield interstory drift ratio (IDR) was lower than the collapse-to-yield PGA ratio. This confirms an adequate safety margin when equating the C_d and R factors. The R factors were comparable for the two sets of ground motions employed in this study. The median R factors of various structures were also comparable and notably higher than the design values, which confirm that the seismic design response factors adopted by the design code are adequately conservative. Although the results indicated a possibility to increase the R factors, the significant impact of severe distant earthquakes on the response of multi-story buildings in the UAE suggests retaining the design code values without changes. This study shows the importance of a systematic assessment of seismic design response factors using a wide range of ground motions and reference structures and the need to cover other classes of buildings and infrastructure to improve public safety.

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