Second Conference on Smart Monitoring,
Assessment and Rehabilitation of Civil Structures

SMAR 2013

Impact of high-strength material on seismic design response factors and economics of multi-story buildings

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ABSTRACT: Five types of 60-story reference structures with varying concrete strengths, ranging from 45 to 110 MPa, are considered in this study to investigate the impact of high-strength materials on the seismic design response factors and economics of buildings. The reference structures are designed and detailed such that very close periods of vibration are obtained from different designs. A large number of inelastic pushover analyses (IPAs) and incremental dynamic analyses (IDAs) are carried out using detailed fiber-based simulation models of the reference structures. Significant profit is achieved with increasing concrete strength due to increasing useable areas and decreasing material quantities. The results indicate a possibility of increasing the design response factors for buildings designed to high-strength material, which enable designers to achieve more cost-effective designs. This systematic study provides practical insights into the economics and seismic response of high-strength multi-story buildings and enables the effective verification of essential factors used in design.

1 INTRODUCTION

The inelastic seismic behavior of structures is taken into account in the elastic design methods by the seismic design response factors, namely the force reduction factor (R) and deflection amplification factors (C_d). These factors are used to reduce the seismic forces and amplify deformations to arrive at cost-effective and safe designs. Seismic codes rely on reserve strength and ductility, which improves the capability of the structure to absorb and dissipate energy, to justify the reduction in seismic design forces using the R factor (Mwafy and Elnashai 2002). The satisfactory performance of buildings designed to modern codes in full-scale tests and in previous earthquakes supports this design concept, especially with regard to life safety. Calibrating the R and C_d factors is necessary to prevent excessive inelastic deformations and loss of life, particularly in the event of a strong earthquake. Moreover, seismic design response factors introduced in seismic codes do not offer a uniform margin of safety and cost effectiveness for buildings with different structural systems and material properties (FEMA 2009; Mwafy 2011). This reflects the pressing need for verifying the seismic design response factors of buildings designed to different material strengths using reliable assessment methodologies. The objective of this study is to investigate the impact of high-strength materials on the economics and design response factors of multi-story buildings through IPAs and IDAs. Extensive results of over 1600 inelastic analyses performed using detailed fiber-based simulation models and twenty input ground motions are employed to verify the seismic design response factors and provide insights into the economics of high-strength multi-story buildings.

2 APPROACH OF VERIFYING SEISMIC DESIGN RESPONSE FACTORS

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. The 'significant yield' in well-designed RC buildings is generally observed at a higher strength level than the level implied in design. FEMA (2009) confirms that the first significant yield of adequately designed structures may occur at lateral load levels that are 30-100% higher than the design forces. This is also confirmed from the results presented below. Mwafy and Elnashai (2002) and Mwafy (2011) suggested the following equation for the evaluation of the R factor:

$$R = R_{c,y} \cdot \Omega_y = \left[(a_g)_c / (a_g)_y\right] \cdot \Omega_y \qquad (1)$$

where, $(a_g)_c$ is the peak ground acceleration (PGA) of the collapse earthquake, $(a_g)_y$ is the PGA at the first indication of significant yield, and Ω_y is the ratio of strength at the first significant yield to design strength, which is also called the first yield overstrength. C_d is equal to R for 5% damping (ASCE-7 2010), which follows the Newmark's equal displacement rule. Over 1600 IDAs are performed in the current study for five 60-story reference structures by scaling and applying each of the selected input ground motions up to the attainment of the collapse limit state. Hence, the PGAs causing the first indication of significant yield and collapse are determined to verify the seismic design response factors.

3 STRUCTURAL SYSTEMS AND MODELLING

Five 60-story reference structures are selected to represent the current high-rise buildings in medium seismicity regions (e.g. Dubai, UAE). Each building consists of two basement stories (B1 and B2), a ground story (L1), and fifty seven typical stories (L2 to L58). The typical height of all floors is 3.2 meters except for the ground story, which is 4.5 meters. The total height for each of the five buildings is 193.3 meters with a similar footprint layout, as shown in Figure 1. The buildings are designed to resist seismic forces according to ASCE-7 (2010). Most recent mapped spectral acceleration parameters for the UAE are used to calculate the seismic loads. Detailed three-dimensional (3D) models are developed for the design of the five reference buildings (denoted as M1, M2, M3, M4 and M5). The five buildings are proportioned and detailed according to various load combinations and the seismic design provisions recommended by the ACI code (ACI-318 2008). The sizes of slabs and beams are kept without changes throughout the building height with constant concrete strength, while the cross-sections of the shear walls, thickness of core walls and concrete grades are reduced along the building height, as shown from Table 1. Every effort was made to obtain the most economical design with comparable fundamental periods for the five buildings. This is undertaken considering acceptable drift limits and stiffness requirements as well as supply-to-demand ratios. For the sake of brevity, additional design information are available elsewhere (Hussain 2012).

The idealization adopted in the current study is performed using ZEUS-NL, which effectively models reinforcing steel, unconfined and confined concrete (Elnashai et al. 2012). A number of cubic elasto-plastic elements capable of representing the spread of yielding and cracking are used to model each structural member. This enables modeling different arrangements of reinforcing steel along the length of each structural member as specified in design. Actual material strengths are employed in the ZEUS-NL models. The concrete response is represented by using a uniaxial constant confinement concrete model, while a bilinear elasto-plastic model is selected to model reinforcing steel. Since the 3D modeling and analysis of high-rise structures are computationally demanding, particularly with the wide range of reference buildings and input ground motions considered in the present study, a 2D idealization is adopted. It is assumed for each building that four framing systems resist seismic forces in the transverse directions, while a one frame resists lateral forces in the longitudinal direction, as shown in Figure 1. Each of the framing systems in the transverse direction is loaded with 25% of the total mass of the building. Results obtained from the previous studies carried out on a comparable building layout indicated that the transverse direction is slightly more vulnerable than the longitudinal direction (Mwafy 2011). Therefore, the present study only focuses on the framing systems in the transverse direction to reduce the number of analyses. Additional information concerning the modeling of the reference structures for inelastic analysis is presented elsewhere (Mwafy 2011).

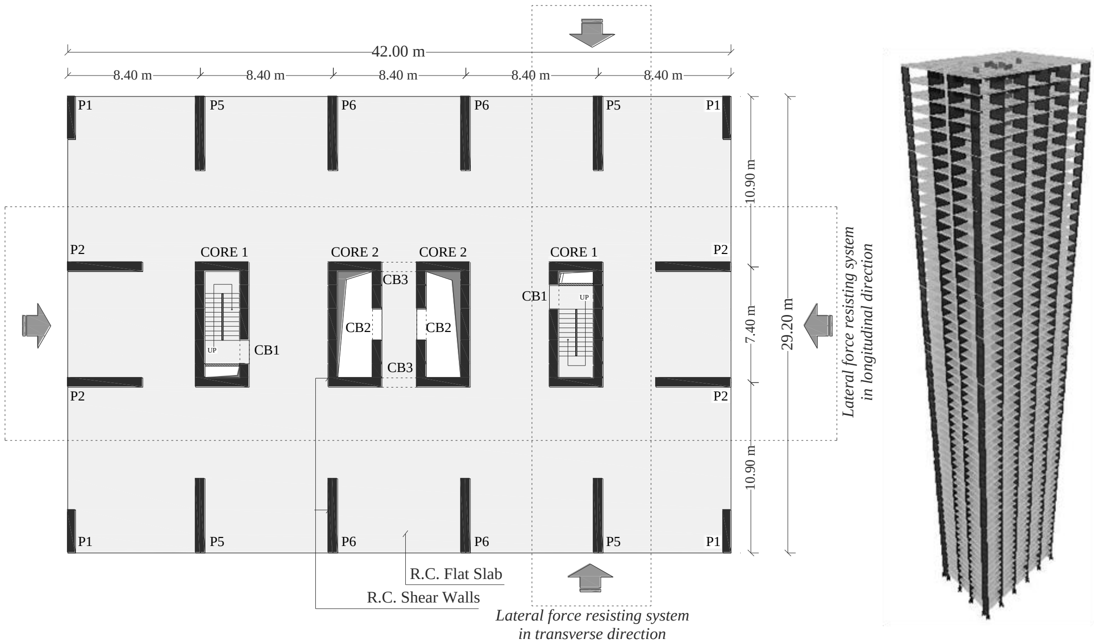


Figure 1. Layout and 3D model of the reference structures.

4 COST ASSESSMENT

The construction cost of the five reference structures is compared in terms of steel, concrete and formwork. The cost of the foundation is not evaluated, while other architectural and finishing costs are considered to be constant between all reference structures. The vertical element sizes decrease with increasing concrete strength, and consequently the volume of concrete and the area of formwork decrease. It is noteworthy that the material costs depend to a large extent on the cost of reinforcing steel. A reduction in the volume of concrete from the reference building M1, which is designed using the lowest concrete strength, to building M5, which employs the highest concrete strength, is only 12%. A substantial reduction of steel reinforcement (up to 37%) is noticed between the lower strength concrete building M1 and higher strength structure M5, as shown from Figure 2. For buildings designed to lower strength concrete, heavy steel reinforcement is required in the shear walls at the lower half of the structure. As the strength of concrete increases, the capacity of concrete to resist axial loads increases, and hence decreases the need for reinforcing steel. The total profits gained from increasing the salable area along with the saving in construction cost due to increasing material strength are depicted in Figure 3. All results are presented relative to building M1, which has the lowest concrete strength. The results indicate that increasing the concrete strength generally results in the most cost effective design. Although the unit cost of concrete increases with increasing strength, the reductions in section sizes and steel ratios result in the most economical design. The net profit, which is calculated from salable area after deducting all construction expenses and cost of land, consistently increases with increasing concrete strengths. The total profit gained from using the highest material strength increased by up to $4.77 million which is 4.95% higher when compared with the building that has the lowest concrete strength.

Table 1. Dynamic characteristics, material properties and sizes of main structural members.

Ref.	Period, Sec.	Member		Dimensions/ Material	Story B1-L8	L9-L18	L19-L28	L29-L38	L39-L48	L49-L58
M1	6.879	Walls	P2, P5, P6	Cross section	650 x 4750	600 x 4750	500 x 4750	400 x 4750	300 x 4750	275 x 4750
				f_c'	50	40	35	35	35	35
		Core	C1	Thickness	550	450	350	300	275	275
				f_c'	40	35	35	35	35	35
			C2	Thickness	600	500	400	350	300	275
				f_c'	40	35	35	35	35	35
M2	6.829	Walls	P2, P5, P6	Cross section	600 x 4750	550 x 4750	450 x 4750	350 x 4750	275 x 4750	250 x 4750
				f_c'	57	50	40	40	35	35
		Core	C1	Thickness	500	400	300	275	250	250
				f_c'	50	40	40	35	35	35
			C2	Thickness	550	450	350	300	275	250
				f_c'	50	40	40	35	35	35
M3	6.810	Walls	P2, P5, P6	Cross section	550 x 4750	500 x 4750	400 x 4750	325 x 4750	250 x 4750	225 x 4750
				f_c'	65	57	50	50	40	35
		Core	C1	Thickness	450	350	275	250	225	225
				f_c'	57	50	50	40	40	35
			C2	Thickness	500	400	300	275	250	225
				f_c'	57	50	50	40	40	35
M4	6.831	Walls	P2, P5, P6	Cross section	500 x 4750	450 x 4750	350 x 4750	300 x 4750	225 x 4750	200 x 4750
				f_c'	75	65	57	57	50	40
		Core	C1	Thickness	400	300	250	225	200	200
				f_c'	65	57	57	50	50	40
			C2	Thickness	450	350	275	250	225	200
				f_c'	65	57	57	50	50	40
M5	6.886	Walls	P2, P5, P6	Cross section	450 x 4750	400 x 4750	300 x 4750	250 x 4750	225 x 4750	200 x 4750
				f_c'	95	85	75	75	57	40
		Core	C1	Thickness	300	275	250	225	200	200
				f_c'	85	75	75	57	57	40
			C2	Thickness	400	300	275	250	225	200
				f_c'	85	75	75	57	57	40

Steel yield strength = 460 MPa; Vertical steel reinforcement ratio of walls and cores vary from 1.0% to 4.6% along the height. Flat slab thickness = 0.28mm; Concrete strength is in MPa and dimensions are in mm.

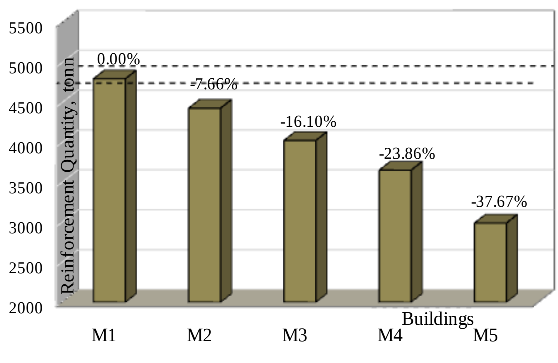


Figure 2. Comparison of the saving in reinforcement quantity relative to building M1.

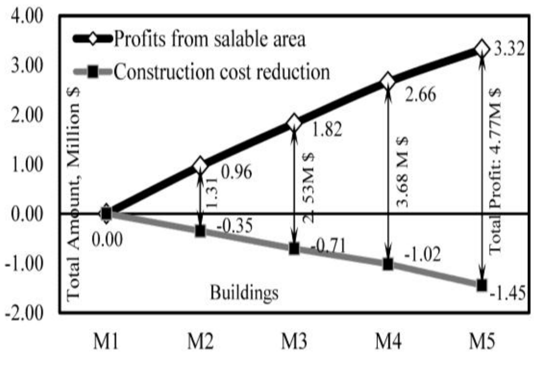


Figure 3. Profits from increasing salable area and saving of material relative to building M1.

5 EARTHQUAKE LOADS AND LIMIT STATES

Twenty input ground motions are selected to represent the most critical seismic scenario for the reference region based on the study of Mwafy et al. (2006), as shown from Table 2. These records were initially scaled to a design PGA of 0.16g based on the recommendations of the abovementioned hazard study. The selected records also fit the Uniform Hazard Spectrum (UHS) of Dubai for 10% probability of exceedance in 50 years. Furthermore, realistic definitions of limit states are needed to obtain accurate predictions of the seismic design response factors. The first yield overstrength (Ω_y) is evaluated from both IPAs and IDAs, and the most conservative values are used. The other component of the response modification factor ($R_{c,y}$), which is ground motion dependent, is evaluated based on two important limit states, namely at yield and collapse. For adequately designed buildings, a RC structure reaches 'significant yield' when one of its most highly stressed sections reaches its yield strength. This is assumed when the strain in the main longitudinal tensile reinforcement exceeds the steel yield strain. The primary collapse criterion is defined using an inter-story drift (IDR) ratio. ASCE-41 (2006) considers the collapse prevention criterion in concrete wall structures is at IDR of 2%. An IDR collapse limit of 2.5% for ductile concrete wall structures was recommended in a number of previous studies (e.g. Ghobarah et al. 1999). Considering that the code recommended drift limits are usually on the conservative side, IDR of 2.5% is therefore adopted based on the values recommended by Ghobarah (1999). 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-318 2008). A large number of inelastic response history analyses are conducted for the reference structures up to the satisfaction of the yield and collapse limit states (about 1600 runs). In addition to monitoring global response parameters, the formation of plastic hinges in different structural members are screened to provide the required indication of first significant yield. The PGAs at yield and collapse are recorded to get the ground motion dependent component of the response modification factor for all ground motions and reference structures.

Table 2. Characteristics of selected natural and artificial input ground motions.

Ref.	**Earthquake**	**Station**	**Comp.**	**Date**	**Magn.** (Mw)	**Site class**	**Epic. Dist.** (Km)	**Duration** (sec)	**PGA** (m/s2)	**a/v** g/ms^{-1}	**a/v class.**
R1	Loma Prieta	Emeryville	260	18/10/1989	6.93	v. dense	96.5	39	2.45	0.57	
R2	Manjil	Tonekabun	N132	20/06/1990	7.42	v. dense	131	40	1.22	0.76	
R3	Bucharest	Bldg res. Institute	EW	04/03/1977	7.53	stiff	161	18	1.73	0.60	
R4	Chi-Chi	ILA013	EW	20/09/1999	7.62	v. dense	135	117	1.36	0.52	
R5	Izmit	Ambarli-Termik	EW	17/08/1999	7.64	stiff	113	150	1.80	0.60	
R6	Loma Prieta	Golden G. Bridge	270	18/10/1989	6.93	v. dense	100	38	2.29	0.61	
R7	Kocaeli	Bursa Tofas	E	17/08/1999	7.51	stiff	95	139	1.06	0.49	
R8	Chi-Chi	ILA030	E	20/09/1999	7.62	stiff	136	90	1.16	0.43	
R9	Chi-Chi	TAP005	E	20/09/1999	7.62	stiff	156	134	1.34	0.49	
R10	Chi-Chi	TAP010	E	20/09/1999	7.62	stiff	151	144	1.19	0.50	Low
R11	Chi-Chi	TAP017	E	20/09/1999	7.62	stiff	148	151	1.12	0.53	
R12	Chi-Chi	TAP021	E	20/09/1999	7.62	stiff	151	125	1.15	0.47	
R13	Chi-Chi	TAP032	N	20/09/1999	7.62	v. dense	144	90	1.13	0.64	
R14	Chi-Chi	TAP090	E	20/09/1999	7.62	stiff	156	125	1.28	0.41	
R15	Chi-Chi	TAP095	N	20/09/1999	7.62	stiff	158	123	0.96	0.52	
R16	BEQ3							60	1.57	0.61	
R17	BEQ4	Artificially generated to match site specific uniform hazard spectrum						60	1.57	0.55	
R18	BEQ5	(refer to Mwafy et al., 2006)						60	1.57	0.61	
R19	BEQ6							60	1.57	0.60	
R20	BEQ7							60	1.57	0.61	

a/v: PGA/PGV, a/v classification (<0.8 Low &>1.2 high), shear wave velocity (Vs_{30}) of very dense soil = 360-760 m/s, for stiff soil = 180-360 m/s

6 EVALUATION OF SEISMIC DESIGN RESPONSE FACTORS, Ω, R & C_d

Structural overstrength (Ω_o) and first yield overstrength (Ω_y) factors of the reference structures are initially evaluated using IPA. Previous studies 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 (e.g. Mwafy et al. 2006). The pushover analysis results are generally on the conservative side when compared with IDA results (Mwafy 2011; Hussain 2012). This is mainly due to the sensitivity of high-rise wall structures to higher mode effects, which amplify the base shear during time history analysis. Hence, higher overstrength is evaluated from IDA compared with IPA. To be on the conservative side in the evaluation of the R factors, the Ω_y

factors calculated from IPA are utilized. The first indication of yielding in walls and in horizontal members as well the global yielding and ultimate capacity are shown in Figure 4. The first indication of local yielding is noticed in horizontal members and followed by vertical members. This is in a good agreement with the strong-column weak-beam code principle of having energy dissipation concentrated in horizontal elements. It is also interesting to note the difference between the capacity envelope of buildings M1 and M5. The response of the latter building is less ductile than the former building, which is confirmed from the steeper post-peak branch of the building M5. The gradual strength degradation observed in building M1 is indeed more favorable than the rapid loss of strength shown in the response of building M5. The large amounts of steel reinforcement in shear walls of building M1 provide considerable flexural resistance under lateral loads, thus slightly higher levels of overstrength are observed in buildings M1 than in building M5, as shown from Figure 4.

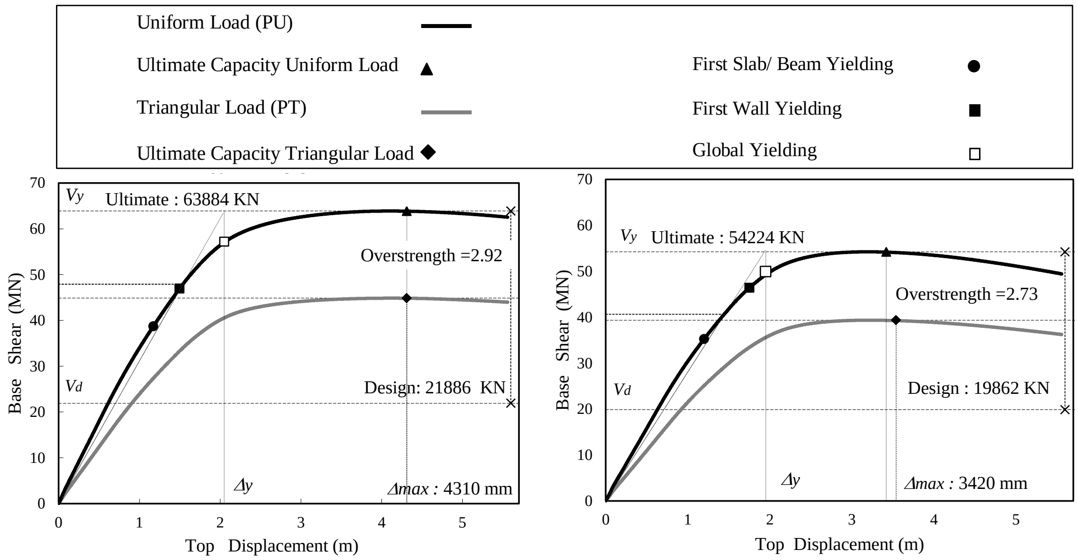


Figure 4. Sample of the capacity curves of the M1 (left) and M5 (right) buildings.

Incremental dynamic analysis is performed for each of the five reference structures using the twenty input ground motions shown in Table 2. Each record is incrementally scaled from a PGA of 0.08g to 1.20g using a scaling factor of 0.08g. The global response parameters of the five reference structures are obtained from over 1600 analyses. The results at the yield and collapse limit states obtained from all ground motions and structures through IDAs are summarized in Figure 5. These results are used to estimate the seismic design response factors, R and C_d, using Eqn. 1. The collapse-to-yield PGA and IDR ratios are presented in Figure 5. The collapse-to-yield PGA ratios are used along with the first yield overstrength (Ω_y) to estimate the R factors. The calculated collapse-to-yield PGA ratios show a clear trend of increase as the building material strength increases. This is mainly because the first indication of collapse occurs at higher PGA levels while the first yield is not significantly affected with increasing material strength. This implies that the impact of earthquakes decreases as the building material strength increases from M1 to M5.

The collapse-to-yield IDRs are always lower than the collapse-to-yield PGA ratios, as shown in Figure 5. The difference between these two median ratios is in the range of 9% to 21% for building M1 to M5, respectively. This difference increases with the increase in material strength. This reflects the adequate conservatism in equating the deflection amplification factor C_d, with the R factor (ASCE-7 2010). The margin of safety increases with increasing material strength. A summary of the force reduction factors evaluated using the twenty earthquake records employed in the present study is shown in Figure 6. This figure depicts the design R factors (R Code), median R values obtained from different input ground motions and those calculated from the median collapse and yield PGAs shown in Figure 5. It is clear that the median R factors of the reference structures are significantly higher than the values suggested by the design code (ASCE-7 2010). The results reflect the high safety margins of the R factor recommended in the design code regarding concrete wall structures. The R factor safety margin increases with increasing material strength from the M1 to M5 buildings. The results confirm the increase in safety margins with increasing material strength.

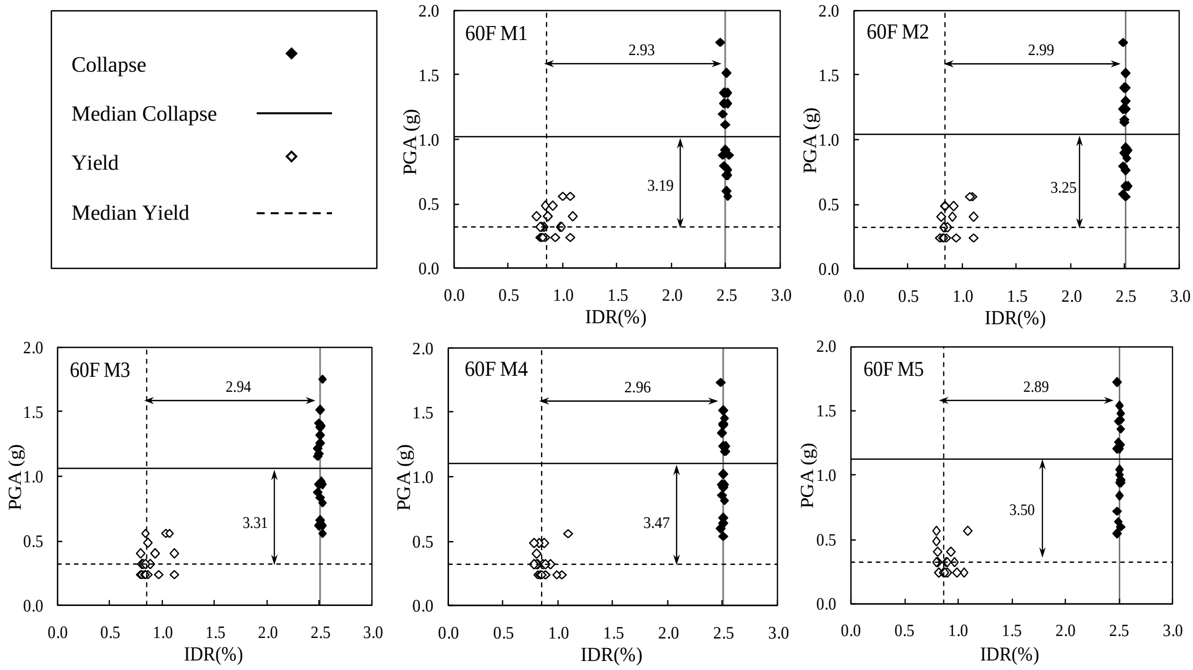
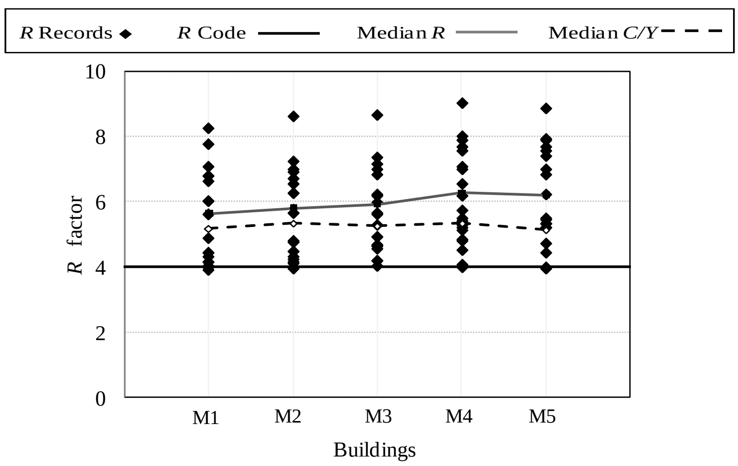


Figure 5. IDA results at yield and collapse along with collapse-to-yield PGA and IDR ratios.



(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 Figure 5)

Figure 6. R factors of the reference buildings obtained from IDAs using twenty input ground motions.

CONCLUSIONS

This paper focused on investigating the impact of increasing material strength on the economics and seismic design response factors, namely the overstrength factor (Ω), force reduction factor (R) and deflection amplification factor (C_d). It is concluded that the cost effectiveness depends to a large extent on the cost of steel reinforcement. With increasing concrete strength, the steel cost reduced by up to 37%. Although the unit cost of concrete increases with increasing strength, the reductions in reinforcement ratios and section sizes as well as increasing salable areas resulted in the most cost effective design. The net profit consistently increased with increasing concrete strength. For the reference 60-story buildings, the total profit gained from using the highest material strength increased by $4.77 million which is 4.95% higher when compared to the building that has the lowest concrete strength. IDA results provided insight into the inelastic seismic response of the reference structures and enabled the assessment of the seismic response factors. Collapse was observed at a higher PGA level as the building material strength increased, which implies a lower seismic risk. The collapse-to yield IDRs were lower than the collapse-to-yield PGA ratios. The difference between these two ratios increased with the increase in material strength. This reflected the adequate conservatism in equating C_d with R and the increase in the safety margin with increasing material strength. The median R factors were significantly higher than the values adopted by the design code. The R factor safety margin increased with increasing the material strength of the buildings. The results indicated a possibility to increase the R factors, particularly for the high-strength concrete buildings, which enable the designers to arrive at more cost-effective designs. The presented systematic assessment study confirmed the significance of verifying the design provisions using a reliable assessment methodology and a wide range of reference structures and input ground motions.

ACKNOWLEDGMENTS

This work was partially supported by the UAE University under research grant code 31N007.

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