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ASSESSMENT OF SEISMIC DESIGN COEFFICIENTS FOR DIFFERENT STRUCTURAL SYSTEMS OF BUILDINGS

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Abstract

The verification of the approaches and coefficients used for the seismic design of the diverse structural systems of multi-story buildings represents a priority, particularly with the unprecedented escalation in the number of buildings in earthquake-prone regions. The seismic design modification factors of modern multi-story buildings with three different structural systems are verified in this paper using a systematic assessment approach. Ten reference structures of low, medium and high-rise buildings are selected to represent the contemporary building inventory in metropolitan areas. The selected structures are reinforced concrete (RC) buildings of 2, 8, 18, 26, 40, 50, 56, 66, 80 and 100-story. The benchmark buildings have six various layouts and three different lateral force resisting systems, namely flat slab-columns (FSC), shear walls (SW) and tube in tube (TIT) structural systems. The ten reference buildings are fully designed and detailed according to the latest U.S. building codes. The seismic design response parameters, namely overstrength (Ω), force reduction (R) and deflection amplification (C_d) factors, are estimated through a large number of inelastic pushover analyses (IPAs) and dynamic time history analyses (THAs) using detailed fiber-based analytical models and a large set of earthquake records. The effective stiffness of different structural systems is also compared with the code recommended values and realistic stiffness values are proposed for design to reflect the anticipated cracking at the life safety performance level. The large inelastic analysis results obtained from the seismic assessment of ten multi-story buildings at different performance levels up to collapse reflect the urgent need to decrease the design Ω factor by 10% and increase the C_d factor by 10% for the low-rise FSC system. For the SW and TIT systems, the R factor can be safely increased by 10% to arrive at more cost effective design, with the potential of increasing it further after a thorough assessment of the proposed changes in seismic loads.

Keywords: seismic design coefficients; modern RC buildings; structural systems; inelastic dynamic analysis; effective stiffness

1. Introduction

Multi-story buildings have appeared to satisfy the increasing demands of urbanization. The rapid growth of urban population, the high cost of land, and a desire to avoid continuous urban spread led to a significant increase in the construction of multi-story buildings. Medium to high-rise buildings pose the most significant concerns in the potential consequences from natural hazard events since they represent concentrated economic and human assets. The accurate and cost-effective design of this class of structures to resist anticipated earthquake loads have a high priority, particularly with the unprecedented increase in the number of multi-story buildings in seismic regions. The determination of the structural system of a multi-story building involves the selection and arrangement of the main structural members to resist the various combinations of gravity and horizontal loads such as earthquake and wind loads. The main factors considered in selecting the structural system include the building height, nature and magnitude of lateral loads, architectural geometry, and material and method of construction. The structure height represents to some extent the most important factor in choosing the appropriate lateral force resisting system. A wide range of structural systems have been recommended for reinforced concrete (RC) buildings according to building height [e.g. 1, 2]. While the structural system must fulfill the imposed architectural requirements for modern buildings, it should also satisfy the required performance under different loading conditions. Hence, in seismic regions, the coefficients used in the design of different structural systems of multi-story buildings should be meticulously verified under expected earthquake loads.

Modern seismic codes commonly use the approach of reducing the seismic forces using a reduction factor to reach the design force level [e.g. 3, 4]. The seismic design coefficients, namely overstrength (Ω), force reduction (R) and deflection amplification (C_d) factors, are adopted by the design provisions to arrive at safe structures and economic designs. Therefore, these seismic coefficients play a key role in the safety and economy of structures. It has been confirmed that these design coefficients depend on several parameters such as the structural system and building characteristics [e.g. 5, 6]. The seismic coefficients recommended in design codes still do not provide a consistent safety margin for different seismic zones with the variation of structural systems and construction practices [5, 7]. This confirms the need for verifying the seismic design coefficients for a wide range of reference buildings representing different structural systems and building heights using reliable assessment approaches.

The main objectives of this study are therefore to evaluate the response factors commonly used in seismic design for a wide range of structures with different building heights and structural systems and to verify these design coefficients with those recommended by modern building codes in order to arrive at a reliable and cost-effective estimation of seismic design forces.

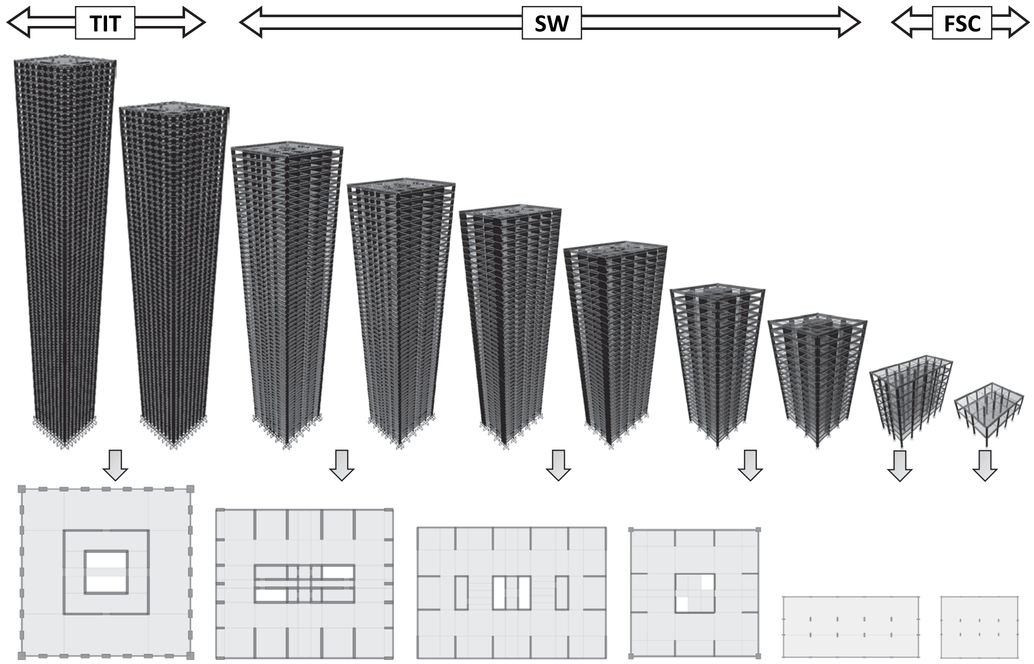
2. Methodology

2.1 Description of Reference structural systems

Ten reference structures of low, medium and high-rise buildings are selected based on the building inventory of a medium seismicity urban area, represented in the present study by Dubai, UAE [8]. The selected structures are RC buildings of 2, 8, 18, 26, 40, 50, 56, 66, 80 and 100-story. The ten buildings have six different layouts and three different structural systems, as shown in Table 1. The reference buildings are fully designed and detailed as per the latest international building codes and construction practice adopted in the study region in order to arrive at the optimum concrete cross-sections and steel reinforcement for all structural members [3, 9]. The yield strength of the reinforcing steel is 460 MPa, while the cylinder concrete strength of vertical structural members decreases along the building height from 64 to 32 MPa (cube strength of 80 to 40 MPa). Floor slabs are designed using a constant cylinder concrete strength of 32 MPa throughout the building height. The cross-sections of shear walls and the associated reinforcing steel vary throughout the building height. The floor slabs of different buildings consist of cast-in-place flat slab systems with varied thickness according to spans of building layout. Different cross-sectional dimensions, reinforcement, and material strength values are adopted to obtain the most efficient and economical design for the reference buildings. Detailed three-dimensional (3D) finite element (FE) models are developed for the design of the reference buildings using the structural analysis and design software ETABS [10]. Fig. 1 depicts the developed ETABS models of the selected buildings along with the corresponding structural system and building layouts. All design information of the ten reference buildings obtained from the comprehensive design process is stored in spreadsheets and AutoCAD drawings. More information regarding the design process and reinforcement details of the ten reference buildings are presented elsewhere [11]. The design results are used to idealize the reference buildings for multi-degree-of-freedom inelastic simulations, as discussed below.

Table 1 – Selected structures to represent modern buildings with different heights and structural systems

No.	Reference	building	Structural system
1	2St	2-story	flat slab-columns (FSC)
2	8St	8-story	
3	18St	18-story	shear walls (SW)
4	26St	26-story	
5	40St	40-story	
6	50St	50-story	
7	56St	56-story	
8	66St	66-story	
9	80St	80-story	tube in tube (TIT)
10	100St	100-story	



TIT: Tube in tube; SW: shear walls; FSC: flat slab-columns

Fig. 1 – Design models of ten reference buildings with the corresponding structural systems and building layouts

Several observations are noted during the design process. The floor slab design of is strongly affected by considering the seismic loads in design due to the stress concentration at the connections between floor slabs and vertical elements, particularly the stiff shear walls of high-rise buildings. The results indicate that the slab reinforcing steel is highly increased at the slab connection with shear walls and core walls when seismic loads are considered. Moreover, although the design provisions recommended by the design codes are fully implemented in the design of high-rise buildings, the design results confirmed that the design should be verified using inelastic dynamic analysis, particularly for the TIT system to avoid the excessive drift demands from higher modes of vibration. The latter observation is consistent with the recent recommendations for the performance based seismic design of high-rise buildings [12].

2.2 Modeling technique and selection of earthquake records

Detailed fiber-based analytical models are developed for the ten reference buildings. The developed models are used to conduct a large number of inelastic pushover analyses (IPAs) and dynamic time history analyses (THAs). The above-mentioned analyses are conducted using ZEUS-NL [13], which is a contemporary platform for inelastic analysis using the fiber modeling approach. This inelastic analysis platform has been extensively verified through experimental testing carried out in Europe and the U.S., and hence has been adopted in several research projects covering complex structures [e.g. 14, 15]. The adopted idealization effectively accounts for reinforcing steel, unconfined concrete and confined concrete. This idealization enables tracing the stress-strain response at certain Gauss sections through the integration of the non-linear stress-strain response of different fibers in which the section is subdivided. The modeling uncertainty is reduced by utilizing the fiber modeling approach since assumptions such as the moment–curvature relationships needed by other idealization approaches such as the lumped plasticity modeling technique are avoided. Each of the structural elements in the current study is modeled using a number of elasto-plastic frame elements capable of representing the spread of inelasticity within the element cross-section and along the member length through the fiber modeling idealization. This approach enables modeling different arrangements of reinforcing steel along the element length as specified in design (at the two edges and at the mid-span). The slab/beam ends are connected with shear wall/core wall (the length between the centerline and the edge of the vertical element representing the shear wall/core wall) by rigid arms. The appropriate material stress-strain relationships are applied to different fibers, and their strains and stresses are monitored. The response of cross-sections is assembled from the response of different fibers. The reinforcing steel is represented by a bilinear elasto-plastic model, while the concrete response is represented by a uniaxial constant confinement concrete model [13]. The actual material strength values are used in the inelastic analysis [16]. Columns and shear walls are considered to be fixed at the foundation level [3].

A large set of natural earthquake ground motions are selected for calculating the seismic design response coefficients of the investigated reference structures. The selected natural input ground motions represent a severe distant earthquake scenario, as shown in Fig. 2. This critical seismic scenario was recommended in previous studies covering the seismic hazard and assessment of the study region [17]. Twenty natural ground motions are selected from the pacific earthquake engineering research center (PEER) and the European strong-motion databases to represent the above-mentioned seismic scenarios [18, 19]. The natural earthquake records are selected based on the following criteria: (i) Epicentral Distance, ranges from 91 to 161 km; (ii) Magnitude, ranges from 6.93 to 7.62; (iii) Site Class, stiff soil 'C' and very dense soil 'D'; and (iv) Spectral Amplification, to match the design spectrum of the studied area [3]. Fig. 2 shows the elastic response spectra of the selected real input ground motions. The selected earthquake records are initially scaled to a design peak ground acceleration (PGA) of 0.16g before applying to the reference structures. This PGA was recommended in previous seismic hazard studies for the reference area [17]. The number of input ground motions employed in the present study and their selection criteria ensure that the investigated buildings are assessed under diverse input ground motions and the most critical seismic scenario.

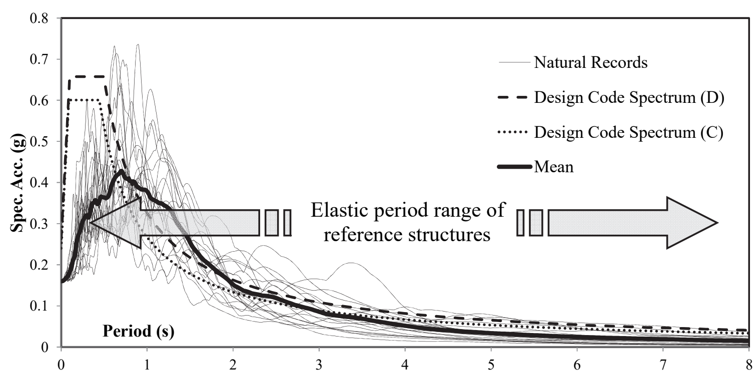


Fig. 2 – Response spectra of the twenty natural ground motions that represent the severe distant earthquake scenario along with the mean spectrum and the design code spectra for site classes "C" and "D"

2.3 Assessment approach of design coefficients

The overstrength (Ω), force reduction (R) and deflection amplification (C_d) factors are assessed in the present study using the fiber-based models of the ten reference structures and the wide range of input ground motions previously discussed. The overstrength factor is the ultimate-to-design strength (V_u/V_d), which is estimated from IPA [20, 21]. The R factor is estimated using the following expression: $R=R_\mu\,\Omega_{fy} = [(a_g)_c/(a_g)_y]\,\Omega_{fy}$, where R_μ is the ductility reduction factor, Ω_{fy} is the overstrength factor at first yield, $(a_g)_c$ and $(a_g)_y$ are the peak ground accelerations of the earthquake that causes collapse and the peak ground accelerations at the first indication of yield, respectively [5, 6]. The C_d factor is considered in the present study to be equal to the collapse-to-yield IDRs. The interstory drift is considered as the primary performance criterion to estimate the collapse prevention (CP) limit state of the reference buildings. The IDR corresponding to the CP limit state is 4.0%, 2.27% and 1.80% for the FSC, SW and TIT structural systems, respectively. These values are obtained from extensive THAs of twenty input ground motions and regression analyses of the THA results. The CP limit state is then verified with the values recommended in previous studies and by design provisions [22-25]. The life safety (LS) limit state, which is used to estimate the effective stiffness used in design, is considered to be equal to 50% of the CP limit state [25].

3. Verification of Seismic Design Coefficients

3.1 Overstrength factors

The seismic design modification factors are estimated in the present study through IPA and THA using the developed fiber-based analytical models of the reference structures. The Ω factor is the ultimate-to-design strength, which is calculated from IPA. Table 2 summarizes the calculated Ω factors of the ten reference buildings along with the code recommended values for different structural systems [3]. It is shown that the calculated Ω factors are higher than the code recommended values for all reference structures except for the 2-story building. This reflects that need to decrease the Ω factor of the low-rise flat slab-columns system by 10% as a result of its inefficient lateral force resisting system.

Table 2 – Comparison between the Ω factors of the reference buildings along with the code recommended values

System	Ref.	Design strength (kN)	Ultimate strength (kN)	Ω (Calculated)	Ω (Code)
FSC	2St	588	1697	2.9	3.0
	8St	1541	5725	3.7	
SW	18St	9015	29379	3.8	2.5
	26St	10042	28174	3.3	
	40St	20247	63994	3.7	
	50St	25725	63288	2.9	
	56St	35470	107359	3.6	
	66St	49025	119950	2.9	
TIT	80St	53396	177385	3.9	2.5
	100St	69157	177429	3.0	

3.2 Force reduction factors

Three main parameters are needed to calculate the R factor; Ω_{fy}, $(a_g)_c$ and $(a_g)_y$, as previously discussed. Fig. 3 shows a comparison between the first yield overstrength factor (Ω_{fy}) obtained from IPA and THA using twenty natural input ground motions representing the most critical seismic scenario expected for the region. Fig. 4 compares between the design strength of the reference structures and the strength at the first indication of yield obtained from IPA and THA. It is shown that the Ω_{fy} values obtained from IPA are lower than those from THA, particularly for mid- and high-rise buildings. This is due to the effect of higher modes of vibrations, which amplify the strength obtained from the dynamic time history analysis.

The peak ground accelerations (PGAs) and interstory drift ratios (IDRs) at the first indication of yield and collapse are summarized in Table 3. These results are obtained from THAs using the selected twenty real input ground motions. For the sake of brevity, only the minimum, maximum and median values obtained from the selected earthquake records are shown in Table 3. The results show a notable difference between the response of the reference buildings under the effect of different input ground motions which reflects the uncertainty due to seismic demand. The median values of the PGAs at the first indication of yield and collapse are presented in Fig. 5. The THA results at the first indication of yield and collapse obtained from the twenty natural earthquakes are illustrated in Fig. 6 for the ten reference structures along with the collapse-to-yield PGA and IDR ratios. The collapse-to-yield PGA ratios are utilized along with Ω_{fy} factors to estimate the R factors, as previously explained. The collapse-to-yield PGA and IDR ratios do not show a clear trend with increasing the building height or within the group of buildings with similar structural systems. This is attributable to the fact that a wide range of structural variables (i.e. layouts, systems and heights) are investigated in the present study. The PGAs at the first indication of yielding are marginally influenced by increasing the building height or changing structural system, as shown in Table 3 and Fig. 5.

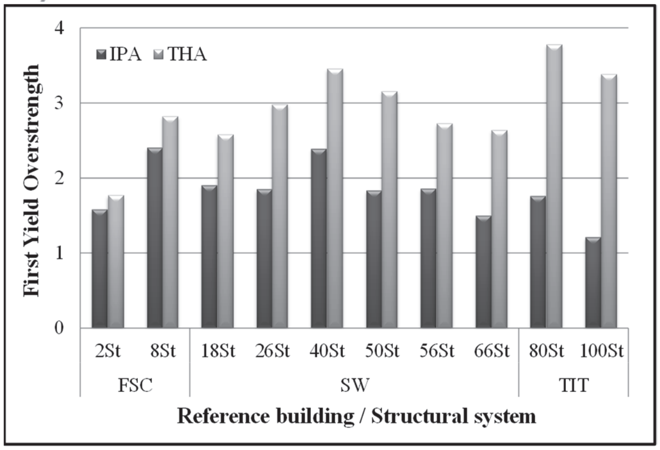


Fig. 3 – Comparison between first yield overstrength (Ω_{fy}) of the reference structures obtained from IPA and THA

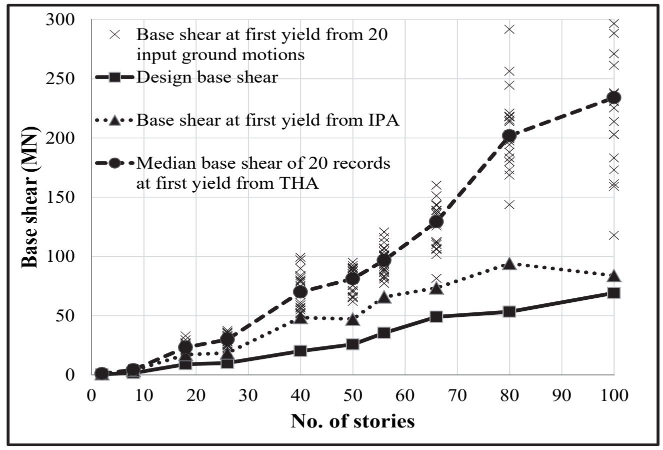


Fig. 4 – Comparison between the design strength of the reference structures and the strength at first yield obtained from IPA and THA

Table 3 – Summary of THA results at the first indication of yielding and collapse for ten buildings using twenty input ground motions

System	Ref.	First indication of yielding						First indication of collapse					
		PGA (g)			IDR (%)			PGA (g)			IDR (%)		
		Min	Max	Median	Min	Max	Median	Min	Max	Median	Min	Max	Median
FSC	2St	0.06	0.16	0.10	0.77	1.10	0.84	0.22	0.54	0.36	3.86	4.18	3.99
	8St	0.08	0.32	0.24	1.29	1.91	1.63	0.32	0.96	0.51	3.88	4.13	4.00
SW	18St	0.08	0.32	0.16	0.45	0.70	0.59	0.40	1.20	0.64	2.25	2.40	2.30
	26St	0.08	0.24	0.24	0.58	0.90	0.69	0.46	0.96	0.66	2.28	2.40	2.32
	40St	0.16	0.40	0.26	0.85	1.14	0.98	0.48	1.20	0.72	2.27	2.49	2.35
	50St	0.16	0.48	0.32	0.83	1.11	0.95	0.48	1.68	0.78	2.27	2.51	2.34
	56St	0.16	0.32	0.24	0.43	0.87	0.67	0.64	1.28	0.88	2.19	2.47	2.33
	66St	0.16	0.48	0.24	0.54	0.77	0.67	0.56	1.60	0.88	2.22	2.46	2.35
TIT	80St	0.08	0.32	0.16	0.35	0.62	0.50	0.44	1.04	0.64	1.69	1.96	1.83
	100St	0.08	0.40	0.16	0.48	0.72	0.59	0.44	1.28	0.74	1.70	1.98	1.83

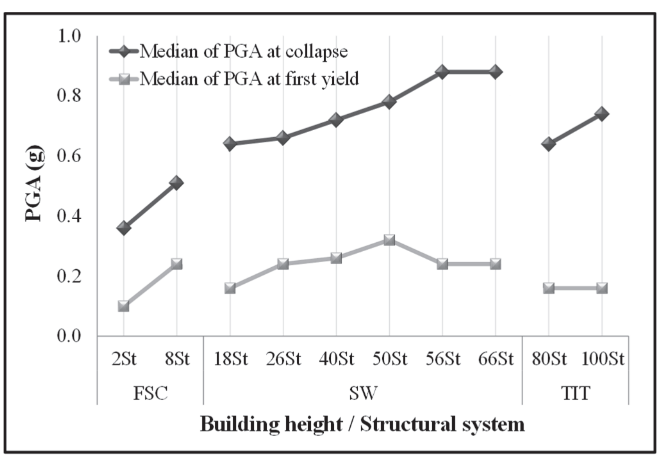


Fig. 5 – Comparison between the median of PGAs at first indication of yield and collapse obtained from THA using twenty input ground motions

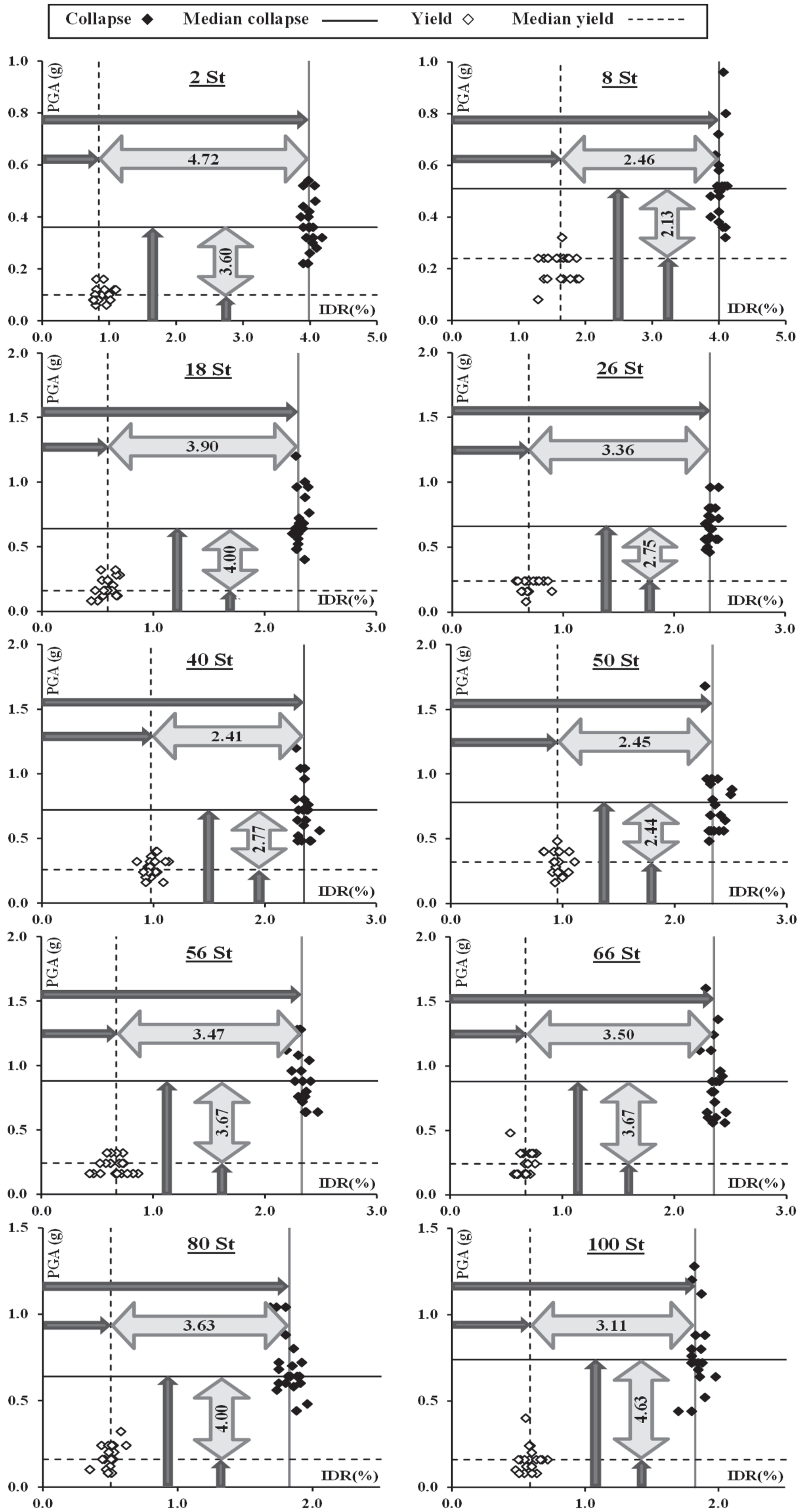


Fig. 6 – THA results at yield and collapse along with collapse-to-yield PGA and IDR ratios for the ten reference buildings using twenty long-period input ground motions

On the other hand, table 3 and Fig. 5 emphasize the direct relationship between the building height and the PGA at the first indication of collapse. For taller buildings, collapse is noticed at higher PGAs for each of the three considered structural systems. This implies that, within each structural system, the impact of earthquake loads decrease as the building height increases since higher PGA are required to cause collapse. The collapse-to-yield PGA ratios shown in Fig. 6 are employed to calculate the R factors of the reference structures, where the Ω_{fy} factors are calculated from both IPA and THA, as depicted in Figs. 7 and 8, respectively. The design R factors (R Code), which are 5.0, 4.0 and 5.0 for the FSC, SW and TIT systems, respectively, are also shown in Figs. 7 and 8. Both of the median R values obtained from individual input ground motions and those obtained from the median collapse-to-yield PGAs illustrated in Fig. 6 are presented in Figs. 7 and 8.

It is shown from Figs. 7 and 8 that the median R factors are generally higher than the values recommended by the design code, particularly for the SW and TIT systems [3]. Moreover, the median R factors for the mid- and high-rise buildings are significantly higher than the values recommended by the design code when considering the more reliable THA approach to evaluate the Ω_{fy} factors, as shown in Fig. 8. The results of this study show that the margin of safety of R factors for the SW and TIT systems is much higher than the FSC system. The results presented in Figs. 7 and 8 imply that the R factors of the SW and TIT systems can be increased by at least 10%. The R factors can be increased further if THA results are considered. However, it is recommended to carefully investigate the impact of the proposed increase of the R factors on the seismic response of the SW and TIT systems before implementing additional reductions in seismic forces.

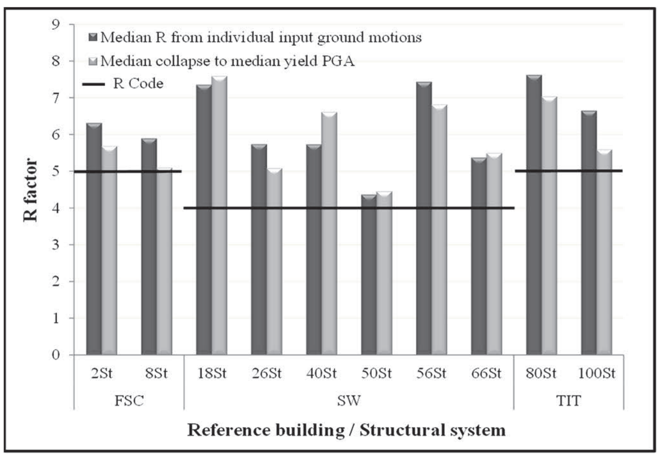


Fig. 7 – Comparison between the code values and the R factors of the reference buildings calculated using collapse-to-yield PGAs from THA and Ω_{fy} from IPA

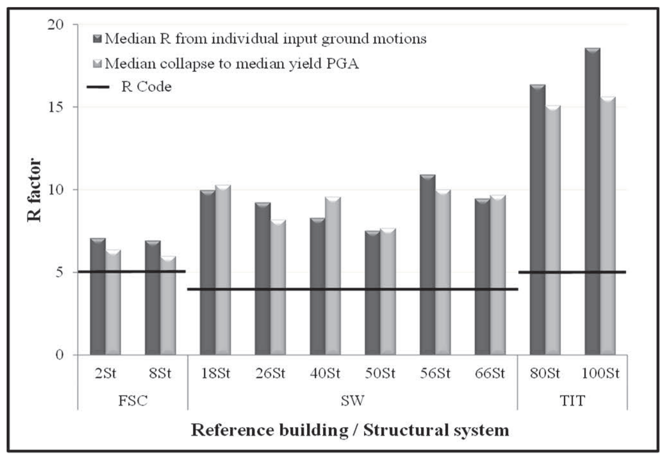


Fig. 8 – Comparison between the code values and the R factors of the reference buildings calculated using collapse-to-yield PGAs and Ω_{fy} from THA

3.3 Deflection amplification factors

The C_d factor is estimated from the collapse-to-yield IDRs shown in Fig. 6, as previously discussed. Fig. 9 summarizes the C_d factors of the reference structures, which are estimated using the median collapse-to-yield IDRs obtained from the twenty long-period ground motions. The design C_d factors (denoted C Code), which are 4.5, 4.0 and 4.5 for the FSC, SW and TIT systems, respectively, are also shown in Fig. 9. Both of the median C_d values obtained from individual earthquakes and those obtained from the median collapse-to-yield IDRs illustrated in Fig. 6 are presented in Fig. 9.

It is clearly shown that the median C_d factors of the SW and TIT systems are adequately conservative when compared with the code recommended values [3], while the code value is slightly non-conservative for the low-rise FSC system. The results suggest that the C_d factor of the low-rise FCS buildings, which lack an efficient lateral force resisting system, should be increased by 10%. Within the same structural system, the observed variations in the C_d and R factors are mainly due to the differences in the building heights and layouts, which result in different seismic demands. The results confirm the significance of studying a wide range of buildings with different geometric characteristics in order to arrive at a reliable assessment of the seismic design coefficients.

3.4 Effective stiffness

Effective stiffness values are typically used in elastic seismic analysis and design procedures. Different effective stiffness values are recommended by design provisions and previous studies, including ACI-318, ASCE-41 and Paulay & Priestley [9, 26]. In order to evaluate the effective stiffness values used in design, comparisons are made between the values recommended by the code provisions and previous studies with the results of the current work, as illustrated in Fig. 10. The results show that the actual elongated periods of the FSC system at the LS limit state are higher than those obtained using the effective stiffness values recommended by the code provisions and previous studies. On the contrary, the elongated periods of the TIT system are notably lower than those obtained using the effective stiffness values recommended by the code provisions and previous studies. The elongated periods of the SW system are higher than those obtained using the recommended effective stiffness for the 18- to 50-story buildings, while the 56- and 66-story buildings show comparable periods from inelastic analysis and the recommended effective stiffness.

Based on the present study results, it is proposed to reduce the effective stiffness of the FSC columns to 0.5EI. The suggested reduction in the column stiffness slightly increases the period of the 2- and 8-story buildings without exceeding the actual inelastic periods at the LS limit state. For the SW system, it is shown that the period elongation is related to the building height and layout. To avoid suggesting non-conservative stiffness values, it is recommended to use the values adopted by ASCE-41 and Paulay & Priestley for walls (i.e. 0.80EI). For the TIT system, the results clearly reflect that the effective stiffness values recommended by design provisions and previous studies are highly non-conservative since they result in much longer periods than the actual inelastic periods at the LS limit state, as shown in Fig. 10. The non-conservative periods may result in underestimating design forces. It is therefore recommended to use the full stiffness of structural members to arrive at conservative inelastic periods for the design of the TIT system.

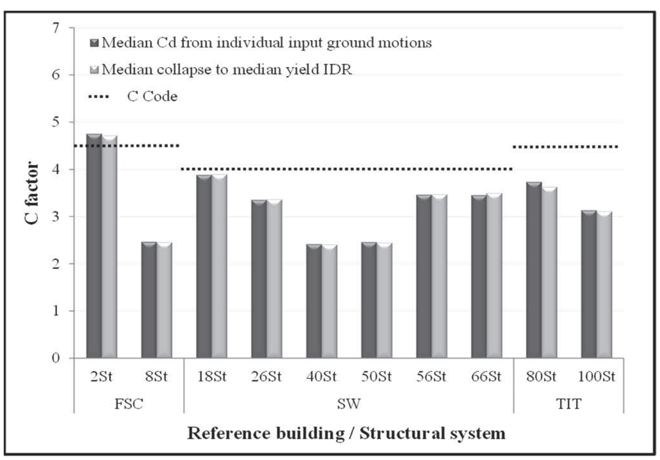


Fig. 9 – comparison of the C_d factors of ten reference structures with the code recommended values

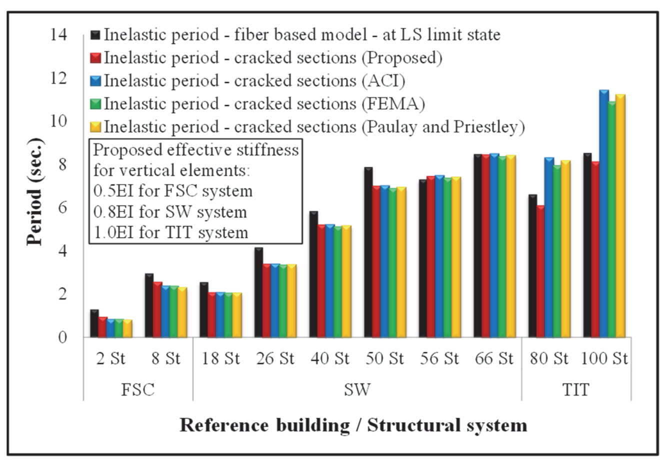


Fig. 10 – Comparison between the inelastic fundamental periods using different effective stiffness values along with those calculated from THA at the LS limit state

4. Conclusions

This study involved the selection, full structural design and development of fiber-based simulation models for ten reference structures representing the modern multi-story building inventory in a medium seismicity region. The benchmark structures are RC buildings of 2, 8, 18, 26, 40, 50, 56, 66, 80 and 100-story. The reference buildings have six various layouts and three different lateral force resisting systems, namely flat slab-columns (FSC), shear walls (SW) and tube in tube (TIT) structural systems. The uncertainty in seismic demands due to the variability in input ground motions was effectively accounted for by using twenty natural earthquake records representing a critical long-period seismic scenario. A large number of IPAs and THAs were performed using the selected earthquake ground motions to assess and fine-tune the overstrength, force reduction and deflection amplification factors.

This comprehensive study concluded that the overstrength factor of the low-rise FSC system should be reduced by 10% due to its inefficient lateral force resisting system. The results indicated that the force reduction factors of the SW and TIT systems could be conservatively increased by at least 10%. An additional increase in the force reduction factors of these systems is possible after a systematic assessment of the impact of the suggested reduction in seismic loads. The median of the deflection amplification factors for the SW and TIT systems were adequately conservative when compared with the code recommended values, while the code value was non-conservative for the low-rise FSC system due to its high lateral deformations. The results confirmed that the deflection amplification factor of the low-rise FSC system should be increased by 10%.

It was also recommended based on the large inelastic analysis results of the ten reference structures to employ an effective stiffness of 0.5% of the non-cracked stiffness for the columns of the FSC structural system. For the SW system, the code recommended effective stiffness for shear walls (0.8% of non-cracked stiffness) was adequately conservative. For the TIT system, the effective stiffness values recommended by design provisions and previous studies were non-conservative. Hence it was recommended to employ non-cracked sections to arrive at conservative inelastic periods of vibration for the design of this relatively stiff system. This comprehensive study enabled verifying the force-based design approach commonly used in the design of three different structural systems of RC buildings and fine-tuning important seismic design factors using reliable assessment techniques, which help to arrive at more consistent safety margins and cost-effectiveness for multi-story buildings in seismic regions.

5. Acknowledgements

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