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SIGNIFICANCE OF SEVERE DISTANT AND MODERATE CLOSE EARTHQUAKES ON DESIGN AND BEHAVIOR OF TALL BUILDINGS

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SUMMARY

Many sites around the world may be subjected to severe distant earthquakes alongside moderate-size, short source-to-site distance events. The two scenarios have different impacts on high-rise buildings and should be therefore investigated. Dubai, a region with an exceptionally high rate of development, is vulnerable to the aforementioned earthquake hazard scenarios. The region represents one of the most rapidly growing in construction of tall buildings worldwide. It was therefore selected for the investigation described in the paper. A hazard study for the construction site of a 187 m high reinforced concrete tower is conducted. Seismicity of the region is outlined and a hazard assessment is carried out to evaluate peak ground accelerations and uniform hazard spectra for different probabilities of exceedance. A number of natural and synthetic records are selected to represent different seismic assessment scenarios at the site. The RC tower is then modeled and analyzed using state-of-the art analytical platforms. Three-dimensional elastic, inelastic pushover and response history analyses are carried out to verify the dynamic characteristic and estimate the capacity to compare it with the predicted demand. The significance of including severe distant earthquakes in design and assessment of high-rise buildings is confirmed. Records representing the latter scenario amplify the fundamental mode that may be overlooked in design using short source-to-site earthquakes. A proposal is made for scaling the results from inelastic dynamic analysis to arrive at a safe and economical design level. The study not only presents comprehensive hazard and vulnerability study for the selected test case, but also gives conclusions that generically apply to the class of long-period buildings subjected to large-distant and small-close earthquakes. Copyright © 2006 John Wiley & Sons, Ltd.

1. INTRODUCTION

Tall buildings are on the increase around the world as a result of several economic, social and engineering considerations. There is a consequential increase in attention to understanding their complex behavior particularly under seismic loading. Regions subjected to low-to-moderate seismic hazard are not at risk from powerful earthquakes and hence, are more economical for constructing high-rise buildings. An example of such regions is Dubai, United Arab Emirates, where the escalating number of tall buildings currently under construction is most noticeable, including Borj Dubai, the publicized tallest building in the world. A vital criterion for ensuring structural safety is the reliable prediction of all anticipated seismic scenarios at the structure site. One of the important earthquake scenarios for such a region would be for severe earthquakes generated in neighboring regions. Such earthquakes would

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generate low levels of ground acceleration at the site, and are therefore often overlooked. A distant large-magnitude earthquake imposes, however, potentially very large displacement demands as well as excited long period (usually fundamental period) modes. It is therefore instructive, and potentially critical, to investigate the response of high-rise buildings, and long-period structures in general, to two assessment scenarios: (i) severe earthquakes with a relatively long epicentral distance; and (ii) moderate events with short source-to-site distance. The former scenario will typically have high amplification in the long-period range, leading to the development of fundamental mode response. The latter scenario will, on the other hand, amplify higher modes of vibration. The two scenarios are feasible for several regions around the world and should therefore be investigated for high-rise buildings, as demonstrated in the scope of investigation below.

Modern seismic codes (e.g. UBC, 1997; IBC, 2003; EC8, 2003) impose criteria on the analysis procedure used in design of tall buildings. Due to the significance of higher mode, dynamic analysis procedures should be used when the height or period of the structure exceeds a certain limit (e.g. $h > 73$ m in UBC or $T > \sim 2{\cdot}0$ s in EC8). The dynamic analysis may be either a modal response spectrum or a response history analysis. For irregular and long-period structures, response history analysis is recommended because the code response spectrum analysis does not account for important properties of the ground motion such as duration and site-specific characteristics.

Seismic design codes exploit the reserve strength and the ability of the structure to dissipate energy to reduce the elastic seismic forces to arrive at an economical design force level. Following this philosophy, UBC (1997) and IBC (2003) permit scaling the response parameters obtained from elastic response history analysis using the response modification factor R. This force reduction is allowed with the limitation that the resulting base shear is not less than that determined according to the equivalent static force procedure. UBC adopts the same procedure for modal response spectrum analysis, where the elastic spectrum is used and response parameters are reduced. IBC and EC8 (2003) adopt a more straightforward approach by employing the design (reduced) spectrum in modal response spectrum analysis. The design codes also use a factor, ranging from $0{\cdot}7R$ in UBC to $1{\cdot}0R$ in EC8, to amplify the drift demands obtained from elastic analysis procedures to obtain the maximum inelastic-equivalent displacement.

The inelastic response history analysis procedure is permitted for design of highly irregular and special structures. Seismic codes do not allow any reduction in the resulting inelastic displacement, which is a rational and justifiable approach. Other response parameters obtained from inelastic response history analysis are not mentioned, implying that they should be used in design without any reduction. This approach may be acceptable for structures designed to clearly exceed the yield limit state under the design ground motion and dissipate energy through ductile response. However, it may be an over-conservative approach for certain types of structure that are intended to behave within or slightly beyond the elastic range under the design earthquake. The response parameters obtained from inelastic analysis of these structures will be close to those from elastic procedures. The latter feature of design codes causes confusion in design since response parameters from elastic procedures are permitted to be significantly reduced using R, unlike those from inelastic analysis. A new measure to characterize the response of structures and their behavior in relation to a well-defined yield limit state has been recently suggested (Elnashai and Mwafy, 2002). This measure, which is termed 'inherent overstrength factor', may be used to treat the above-described inconsistencies. Little attention has been paid to the investigation of the above mentioned issues. The scope of the current study is therefore as follows:

- Select a typical high-rise RC structure in an appropriate region for this investigation. An actual 54-story high-strength RC building from Dubai was selected.

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- Review the seismicity of the site and conduct a comprehensive hazard study to assess the exposure to near and distant earthquakes, leading to reliable recommendations and criteria for design in the selected region.
- Meticulously model and analyze the selected tall building using state-of-the-art analytical tools, making use of the hazard study design criteria and a representative ensemble of natural and synthetic records.
- Conduct both 2D and 3D analyses to verify the developed analytical models and estimate the dynamic characteristics of the structure.
- Accurately quantify the capacity and demand by means of inelastic pushover and response history analysis.
- Compare the capacity–demand balance for the two seismic scenarios to highlight their significance on design of high-rise buildings.
- Discuss implications of this study on analysis procedures used in design and adopted by seismic codes to arrive at more practical suggestions for the design office environment.

2. SEISMICITY OF THE SITE

A site-specific hazard study is undertaken for the city of Dubai, UAE. An earthquake catalogue for the study area was extracted from three different sources, namely Ambraseys and Melville (1982); Ambraseys *et al.*, (1994); and GSHAP (2004). The geographical distribution of the earthquakes for the period 734–1996 is shown in Figure 1. The dots shown in the figure denote earthquake epicenters and their size indicates the earthquake magnitude. It is shown that none of the moderate-to-big earthquakes (magnitude > 6) are closer to Dubai than about 150 km. It is also seen that the epicentral distance of small earthquakes (magnitude > 4·5 to 5·5) closest to the city is roughly 50 km. Almost all earthquakes are located on the northern side of the Persian Gulf, Strait of Hormuz and Gulf of Oman. It is worth noting that the Arabian Peninsula is one of the most seismically quiet and geologically stable regions in the world (Ambraseys *et al.*, 1994). This shows that the earthquake hazard in Dubai is dominated by the seismicity of southern Iran, especially the region around Bandar Abbas. On the other hand, it is possible that local seismicity related to faults south of the Strait of Hormuz may be of importance. Therefore, additional earthquake sources covering the last 10 years were obtained (NEIC, 2004). Only earthquakes with an epicenter less than 1000 km from Dubai are shown in Figure 2. A scatter diagram of magnitude and epicentral distance is shown in Figure 3. It is confirmed that big earthquakes (magnitude > 7) are associated with large epicentral distance (>200 km), and the earthquakes closest to Dubai are relatively small. It is noteworthy that despite the limited information currently available on the seismic activity of the Dibba fault and the fault along the west coast of the United Arab Emirates, both faults have been included in the current study.

3. HAZARD ASSESSMENT OF STUDY SITE

Clearly, it is desirable to use a ground motion attenuation model based on strong-motion recordings from the study area. Since the data available for this region is too limited for derivation of a site-specific equation, it was decided to use a model containing data from the study area augmented by data from other regions. The selected model is that given by Ambraseys *et al.* (1996), which was derived using data from Europe and the Middle East. On the other hand, the peak ground displacement estimation model employed here is that suggested by Ambraseys and Srbulov (1994). The displacement model is selected since it uses a dataset comparable to the dataset employed in derivation the attenuation model. With regard to the response spectrum estimation model, the approach outlined

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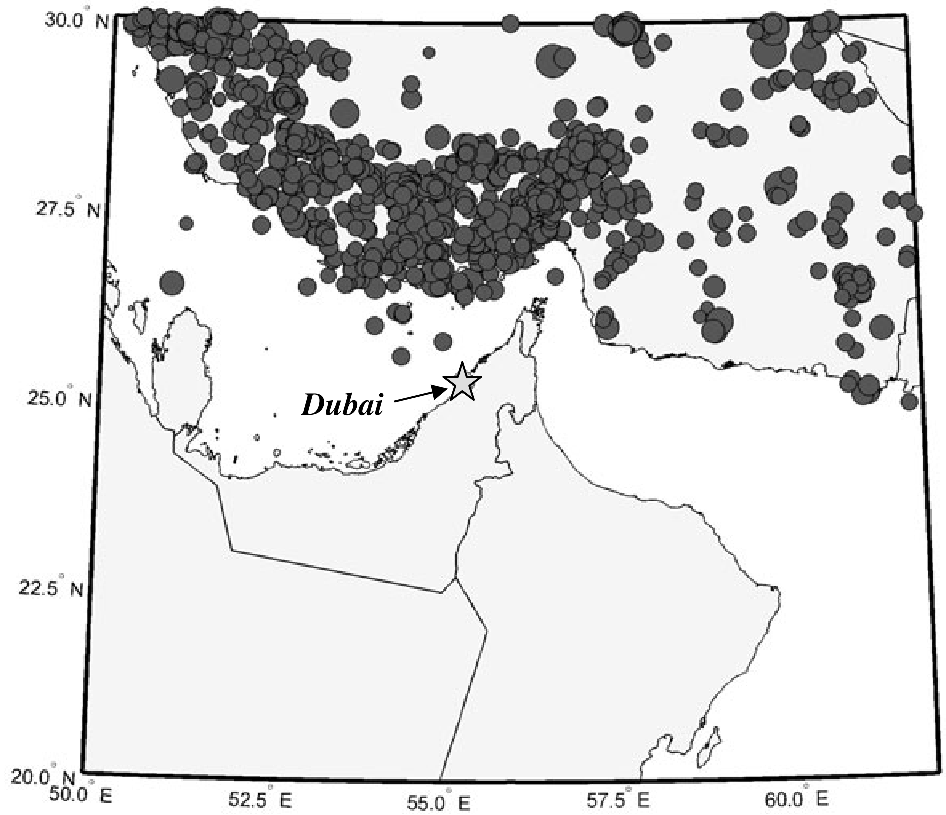

Figure 1. Distribution of earthquake data for the period 734–1996 based on Ambraseys and Melville (1982), Ambraseys *et al.* (1994) and GSHAP (2004). The dots denote earthquake epicenter and their size indicates the earthquake magnitude

in the study of Ambraseys *et al.* (1996) was followed. This approach leads to obtaining consistency in peak ground acceleration and spectral ordinates. The characteristic difference in earthquake-induced response due to a big event (magnitude 7) with relatively large source-to-site distance (100 km) and a moderate-sized event (magnitude 6) with short source-to-site distance (20 km) is exemplified in Figure 4. It is clear that the big distant events have the greatest effects on long period structures while the small to moderate sized event is more likely to have the most severe effect on short period structures, or structures responding in their higher modes.

Figure 5 shows a hazard curve for peak ground acceleration and Table 1 gives the corresponding values, with results for 10%, 5% and 2% probability of exceedance in 50 years. In seismic design codes, the design earthquake load is usually defined as a 10% probability of exceedance in 50 years, which represents a mean return period of 475 years (with the exception of some guidance notes that recommend using a 2500-year return period earthquake). Hence, a PGA of $0{\cdot}16\,g$ corresponding to a 10% probability of exceedance is employed in the inelastic assessment of the tower investigated in the current study. It should be stressed that, as a conservative assumption, the size of the causative faults was taken into consideration and the distance measure in the ground motion estimation model was taken as the shortest distance to the fault rupture. The error term in the ground motion estimation model was also accounted for and no truncation of distributions was introduced.

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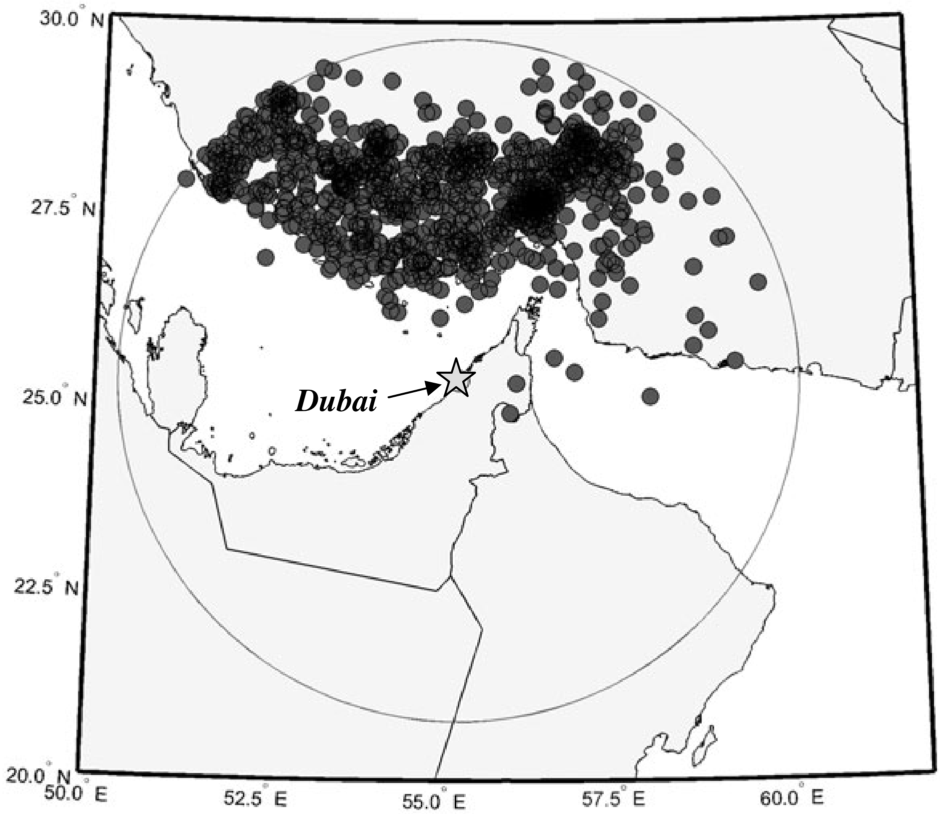

Figure 2. Distribution of earthquake data for the period 1973–2004 based on NEIC (2004). Dots are of the same size and denote earthquake epicenter

It is observed that the values shown in Table 1 are lower than the finding of GSHAP (2004). The latter maps indicate values for Dubai in the range of 0·24–0·33 g. However, it has been pointed out (GSHAP, 2004) that some of the assumptions made in the latter study might be conservative. Also, it should be pointed out that faults, like the Dibba fault and the fault on the west coast of the UAE, were not included in the GSHAP study. A recent seismic hazard assessment study for UAE (Abdallah and Al-Homoud, 2004) showed that the design horizontal peak ground acceleration in Greater Dubai, Sharjah and Ajman area is between 0·10 g and 0·20 g, with an average of 0·15 g. The highest PGA in the latter study (0·2 g) is assigned to Fujaira. These values are consistent with the finding of the current study and confirm the over-conservatism of the values suggested by GSHAP.

The uniform hazard spectra for the site for 10%, 5% and 2% probability of exceedance in 50 years are shown on Figure 6. No smoothing has been performed on the response spectra. Hence, their shape shows the same type of variability, especially for short period structures, as seen in the deterministic spectra given in Figure 4. Moreover, Figure 7 shows the relationship between absolute acceleration response and relative displacement. For a structure with 3·0 s natural period, the relative displacements for 5% and 2% probability of exceedance are 30 cm and 35 cm, respectively.

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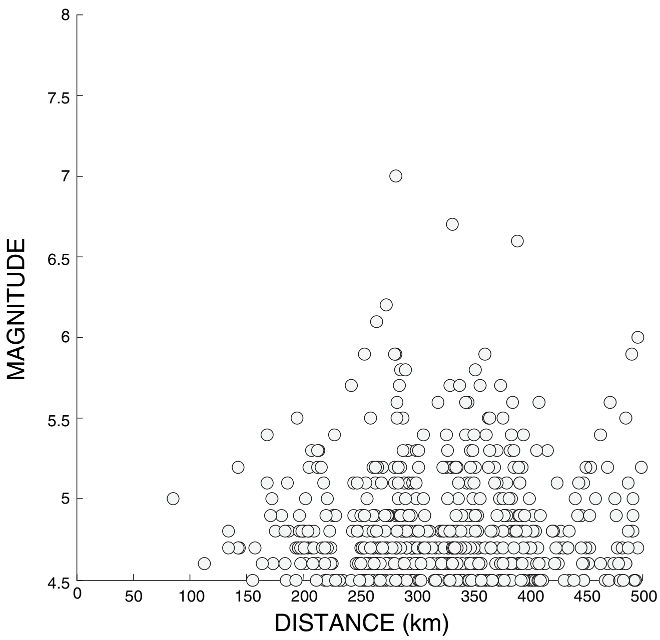

Figure 3. Scatter diagram of earthquake magnitude and epicentral distance for the period 1973–2004, based on NEIC (2004)

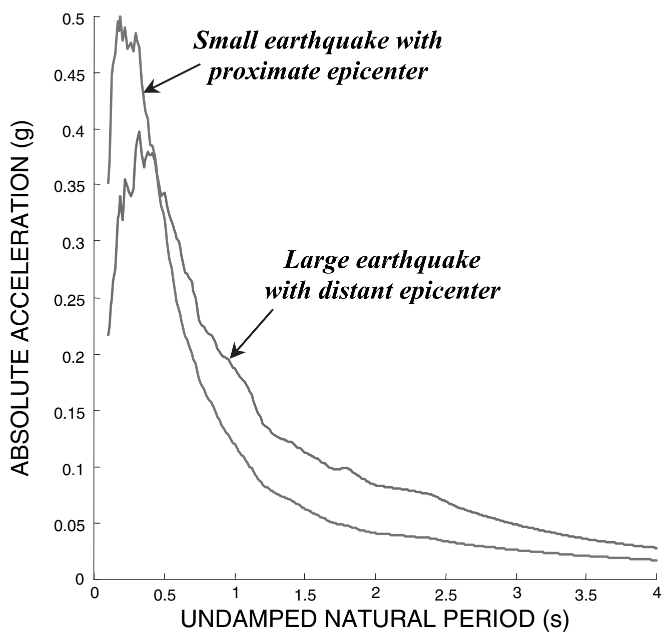

Figure 4. Response spectra due to a large earthquake of magnitude 7·0 with epicentral distance equal to 100 km and a moderate event of magnitude 6·0 with epicentral distance equal to 20 km

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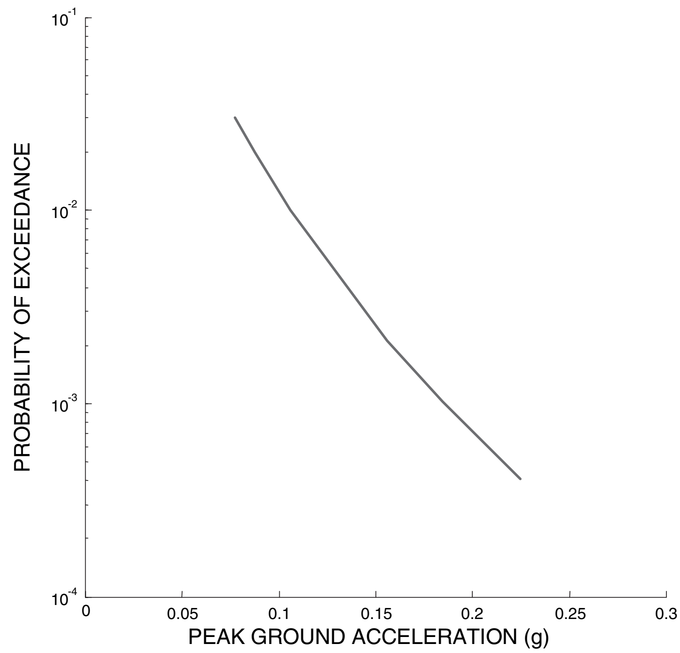

Figure 5. Site-specific earthquake hazard for Dubai

Table 1. Horizontal peak ground acceleration for Dubai

Probability of exceedance	Mean return period (years)	Probability of exceedance (1/year)	Peak ground acceleration (g)
—	—	0·0100	0·11
10% in 50 years	475	0·0021	0·16
5% in 50 years	975	0·0010	0·18
2% in 50 years	2475	0·0004	0·22

4. GROUND MOTIONS FOR DESIGN AND ASSESSMENT

The uniform hazard spectrum is a 'compendium' of all earthquake-induced effects (responses) from events that cause the dominant hazard with a prescribed probability of exceedance. As seen from Figure 4, the big distant events contribute most to the long-period part of the uniform hazard spectrum while the short-period part is due to the moderate-sized events with short source distance. From the design point of view, structures will experience one earthquake at a time. Hence, it is necessary to have, in addition to the individual (deterministic) response spectra, a set of realistic time series representing single events. The importance of time series is particularly true for inelastic response history analysis required for final design and structural assessment. Such time series can either be chosen from a suit of recordings of real events or they can be simulated using available geophysical information along with engineering models. Real recordings suitable for the site concerned in this study are available from several sources such as the ISESD databank (Ambraseys *et al.*, 2002). Two natural records (Loma

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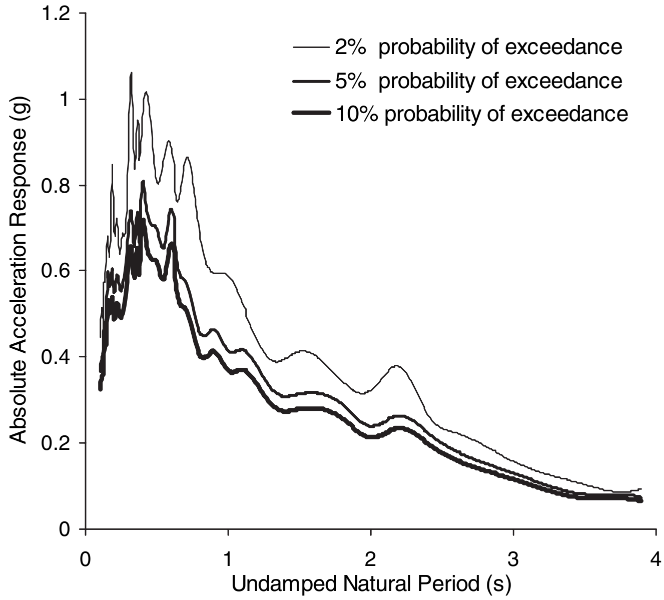

Figure 6. Uniform hazard spectra for Dubai for a 5% critical damping

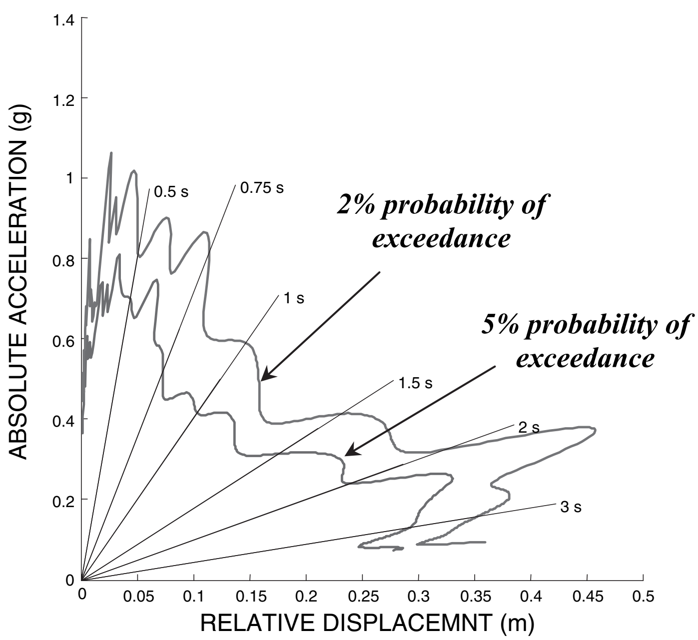

Figure 7. Uniform hazard spectra for Dubai for 5% critical damping showing the acceleration response as a function of relative displacement response

Prieta earthquake at Emeryville, USA, 1989, and Northridge earthquake at Hollister City Hall, USA, 1974) were therefore selected and scaled to a PGA of $0{\cdot}16\,g$. One of these records (Emeryville) is a representative of severe distant events, while moderate close earthquakes are represented by Hollister.

Alternatively, the simulation can be performed in two different ways. First, the simulation can be based on available geophysical and seismological information using a proper mechanical model to represent the source, path and site effects. A simplified approach may be obtained using a point source and Fourier spectrum to represent the source, path and site. This model gives a fair approximation for engineering purposes and is easily used to generate many records with the same average statistical properties. Secondly, the simulation can produce a series that is formally consistent with any response spectrum such as the uniform hazard spectrum. The result will be an acceleration record that does not represent real accelerograms in terms of sequence and ratio of peaks, duration, frequency and energy contents. Such an approach is generally not recommended, particularly for inelastic structural or soil analysis. Accordingly, a number of time series are simulated independently using the simplified Fourier approach. These time series should therefore be regarded as samples of many possible artificially generated records.

According to IBC (2003) and UBC (1997), at least three data sets of ground motions should be employed in response history analysis and the maximum value of each response parameter should be used to determine design acceptability. If seven or more records are employed, use of the average values response parameters is permitted. Hence, five synthetically generated accelerograms, in addition to Emeryville and Hollister, are employed in the current study and the average response is obtained to verify the design acceptability. Three of the simulated records (BEQ1, BEQ2 and BEQ3) are generated by a magnitude 7·4 earthquake with 100 km epicentral distance, representing a large distant earthquake. The other two artificial records (SEQ1 and SEQ2) are generated to simulate a moderate close event of magnitude 6·0 and a short distance to the causative fault of 10 km.

The 5% damped elastic response spectra of big and small earthquakes are shown in Figures 8 and 9. It is clear that characteristics of the selected natural record representing a severe distant earthquake

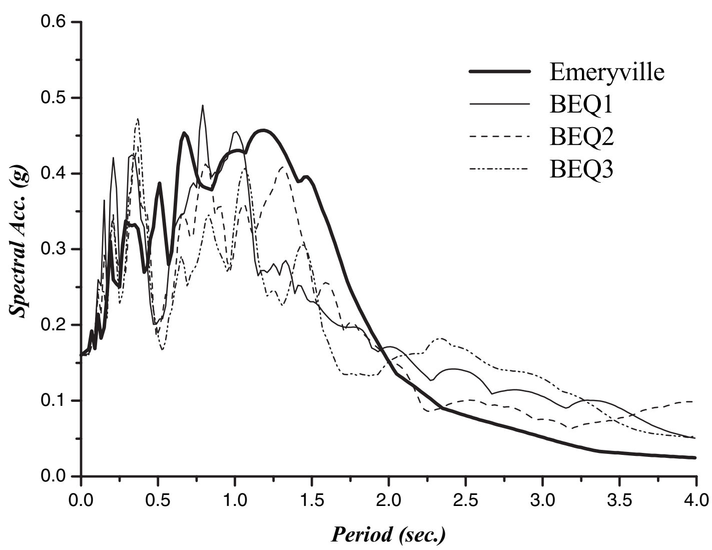

Figure 8. 5% damping response spectra of natural and artificial records representing a large distant earthquake of magnitude 7·0 with epicentral distance of 100 km

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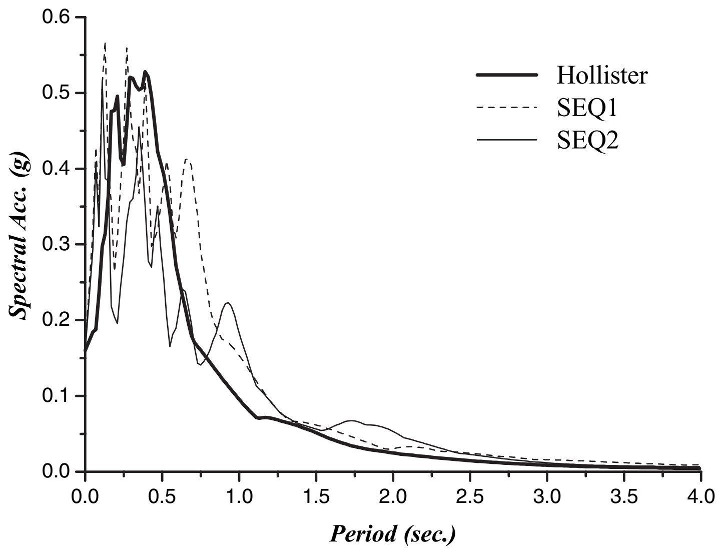

Figure 9. 5% damping response spectra of natural and artificial records representing a moderate close event of magnitude 6·0 with epicentral distance of 20 km

resemble those of the simulated accelerograms, namely the relatively high amplification for long periods. The similarity is also shown between the natural record and the simulated accelerograms representing moderate close earthquakes. It is clear that the selected set of records have high amplification at different frequencies, covering the whole range of structural periods. This ensures that the investigated structure is assessed under input ground motions representing all possible seismic scenarios at the site.

5. STRUCTURAL MODEL OF THE TEST CASE TOWER

To investigate the effect of severe distant and moderate close earthquakes on tall buildings, a 54-story, 187 m height reinforced concrete tower was selected. Figure 10 depicts the typical plan of the tower, showing the main lateral force-resisting systems in the two orthogonal directions. The lateral force resisting system comprises a number of RC internal cores, shear walls and columns connected with floor slabs and a system of beams at the perimeter. The central cores extend the full height of the building, while the two shaded wings shown in Figure 10 are terminated at mid-height. The structure sits on a thick concrete mat supported on a system of piles. The whole structure is symmetrical about the x-axis and slightly unsymmetrical about the y-axis due to the shift in the location of wall W1 in Figure 10. The lateral force design of the structure was conducted according to the 1997 Uniform Building Code (UBC, 1997). The structural members are sized and detailed on the basis of the ACI guidelines (ACI, 2002). The compressive strength of concrete is 40 and 60 N/mm^2 for horizontal and vertical structural members, respectively. The yield strength of steel is 460 N/mm^2. The strength of the vertical members is therefore in the region between normal and high-strength concrete.

Detailed 2D and 3D FE analytical idealizations are constructed for the structure for the purposes of inelastic and elastic analysis, respectively. The platforms used for 3D elastic analysis are the structural analysis and design programs SAP2000 and ETABS (CSI, 2003a, 2003b). In SAP2000, shear walls are modeled using the column analogy approach, whereby frame elements are located at the

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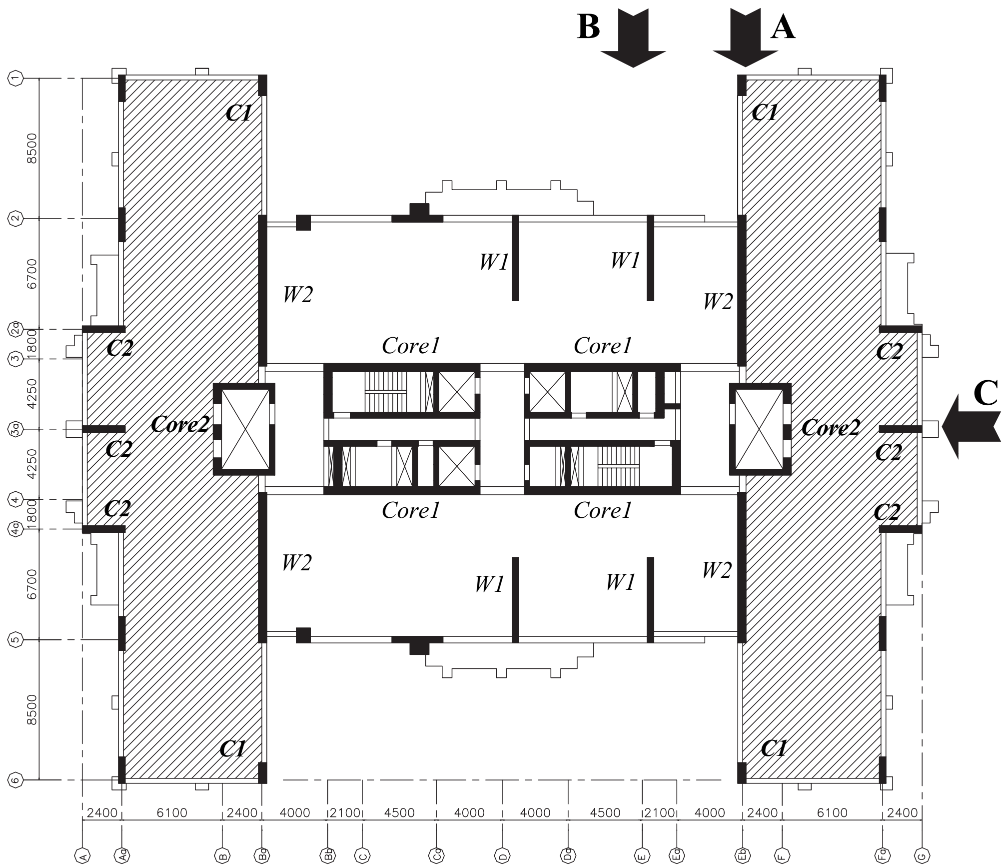

Figure 10. Plan of the 54-story tower showing main lateral force resisting systems (the two shaded wings are terminated at mid-height)

centroid of the walls' cross-section and rigid arms are employed to connect the wall to adjacent beams. RC frame elements are used for modeling of beams and columns. Slabs are also modeled using equivalent frame cross-sections. In ETABS, a more refined analytical model is used, whereby shell elements are used to model cores, shear walls and floor slabs. Figures 11 and 12 show the 3D view of the first two mode shapes in the transverse and longitudinal directions obtained from SAP2000 and ETABS FE models, respectively.

The Mid-America Earthquake Center program ZEUS-NL (Elnashai *et al.*, 2004), which is a state-of-the-art inelastic analytical platform utilizing the fiber approach, is used for inelastic static and dynamic analysis of the tower. The program has been extensively verified against test data from Imperial College, UK, University of Illinois at Urbana-Champaign, USA, and elsewhere (e.g. Elnashai and Elghazouli, 1993; Elnashai and Izzuddin, 1993). It has been also employed in a large number of research projects in the USA and Europe (e.g. Mwafy and Elnashai, 2002; Elnashai and Mwafy, 2004). The refined idealization adopted in the current study effectively models reinforcement steel, unconfined and confined concrete. This approach allows monitoring the stress–strain response at each fiber during the multi-step analysis. A number of cubic elasto-plastic elements capable of representing the spread of yielding and cracking are used to model each structural member. Although ZEUS-NL is effectively

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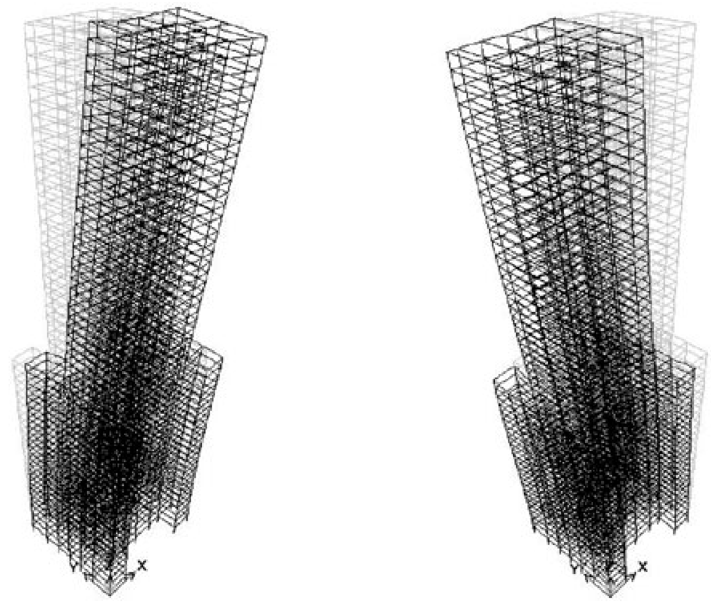

Figure 11. 1st and 2nd mode shapes in y and x direction obtained from the SAP2000 3D model (T_{1y} = 3·8 and T_{1x} = 3·4 s)

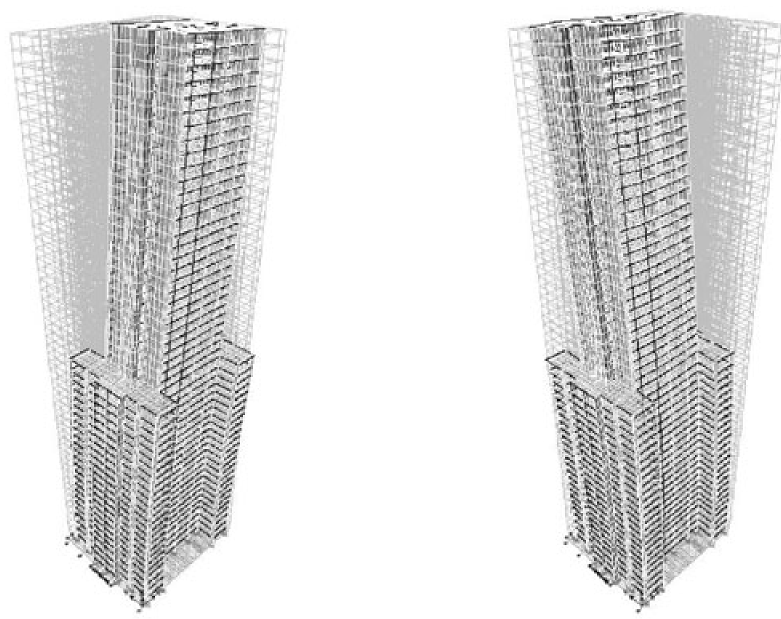

Figure 12. 1st and 2nd mode shapes in y and x direction obtained from the ETABS 3D model (T_{1y} = 4·05 and T_{1x} = 3·47 s)

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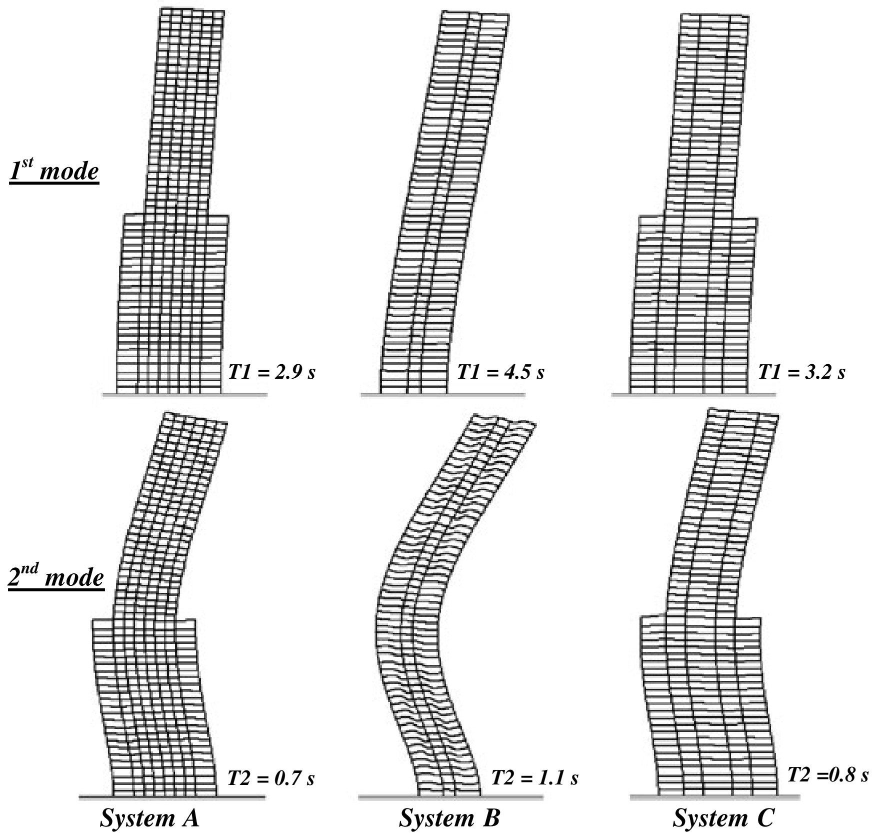

Figure 13. 1st and 2nd mode shapes and period of vibrations obtained from ZEUS-NL

capable of performing 3D inelastic response history analysis of multistory structures, such modeling and analysis are hugely demanding from a computational view point for a 54-story RC tower. Therefore, a 2D idealization is utilized for inelastic pushover and response history analysis carried out. It is assumed here that four framing systems resist the lateral forces in the transverse directions: two of type A and another two of type B (refer to Figure 10). Since the left and right wings are terminated at mid-height, the two framing systems at the left and right margins are assumed to resist gravity loads only. It is also clear that the main lateral force-resisting system in the longitudinal direction is the framing system C, which consists of the internal cores and perimeter columns C2. Walls and other peripheral columns are conservatively assumed not to participate in resisting the lateral forces in the longitudinal direction. Figure 13 shows the first and second mode shapes of the three framing systems A, B and C obtained from ZEUS-NL.

6. DESIGN LATERAL LOAD

The seismic design force estimated from the equivalent static lateral loads is based on UBC (1997) provisions. The base shear is calculated as a ratio of the total (effective) seismic dead load. The fundamental period of vibration according to UBC (1997) and IBC (2000) is calculated as $T = C_t\,(h_n)^{3/4}$ with $C_t = 0{\cdot}0488$. For a height of 187 m, this produces a period of 2·47 s. The average eigenvalue

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analysis results obtained from the SAP2000 and ETABS 3D models indicate that the uncracked period is approximately 3·4 and 3·9 s for the longitudinal and the transverse directions, respectively. Also the ZEUS-NL model results in a fundamental period of 3·2–3·7 s in the two orthogonal directions. The structural system under consideration is stiffer than conventional buildings owing to its stiff coupled walls and cores and also as a result of employing relatively high-strength materials in design. However, the periods calculated from analysis are higher by more than 35% than those estimated from the code expression. It is worth noting that EC8 (2003) adopts the same expression mentioned above but with a note that it is applicable for buildings with a height up to 40 m, which seems more rational. UBC (1997) and EC8 (2003) also suggest estimating C_t for shear wall systems using the area and length of walls. The period calculated using the UBC approach is even more conservative. The latter discussion indicates that the code approximate expressions are over-conservative in estimating the elastic period of high-rise buildings. It is not clear at this stage what effect such an over-conservatism is on design actions and deformations, due to the effect of the input motion characteristics.

For seismic zone 2A, a soil profile type Sc, an importance factor of 1·0 and a response modification factor of 5·5, the design base shear is 0·020 W. This is the lower bound imposed by UBC. Considering a 5% eccentricity, the total design shear at the base of the tower is 22400 kN.

Owing to the height of the tower, dynamic analysis is required in the design. Modal response spectrum analysis is conducted using the 3D ETABS model and the UBC design spectrum. Twenty modes of vibration, which represent ~95% of the participating mass in the two orthogonal directions, are considered. The damping in each mode is taken as 5% of critical, and modal responses are combined using the CQC procedure to account for closely spaced modes. Comparison of the equivalent static force and the response spectrum analyses employing the design (scaled) spectrum indicates that the latter analysis produces lower base shear (34–40%). However, this significant difference is mainly due to the conservative period estimated from the code expression ($T_1 = 2{\cdot}47$ s), which leads to increasing the base shear calculated using the equivalent static force method. The results imply that the static load procedure is more conservative and generates higher story shears, and, hence, controls the design for this 54-story shear wall structure. This is in agreement with the conclusion of previous studies carried out on high-rise RC buildings (e.g. Anderson *et al.*, 1997).

7. VERIFICATIONS OF ANALYSIS MODELS

The effects of rigid arm stiffness employed for modeling of structural walls, damping and material modeling are all critical components in the prediction of seismic structural response. Hence, they were comprehensively investigated before executing the ZEUS-NL inelastic analysis for final performance assessment. It was shown that stiffness of rigid arms plays a significant role in the lateral resistance of the structure and hence its period of vibration. The period decreases and the lateral strength increases with increasing the stiffness of the rigid arms up to an upper bound limit. This effect was also observable in the SAP2000 3D model since walls and cores are modeled using beam elements and rigid arms. This is not the case for ETABS, which utilizes shell elements. Ignoring the rigid arms in the SAP2000 model leads to an increase in the period by 100%. The stiffness of the rigid arms was therefore taken as that at the saturation level.

The influence of damping was also observed in the inelastic response obtained from ZEUS-NL. In addition to the hysteretic damping due to inelastic energy absorption it was decided to use a 1% constant Rayleigh damping based on careful comparisons of the response for different damping levels. This leads to reduction of the average base shear demand in the two directions by about 17% and the top displacement by about 5% compared with the case without Ralyeigh damping. This is shown from the response history analysis results summarized in Tables 2 and 3. Changes in response at the design earthquake were not significant when the Ralyeigh damping was increased further. The effect of

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Table 2. Maximum inelastic global demand of framing system A from seven accelerograms

	Earthquake record	No damping: Top drift (mm)	No damping: Base shear (kN)	1% Rayleigh damping: Top drift (mm)	1% Rayleigh damping: Base shear (kN)	Diff. %: Top drift	Diff. %: Base shear
Small earthquakes	SEQ1	138	30 230	135	23 300	2·3	23
	SEQ2	171	24 780	167	19 115	2·3	23
	Hollister	70	23 650	47	16 352	33	30
Big earthquakes	BEQ1	604	51 815	566	39 800	6·3	23
	BEQ2	391	36 500	390	35 200	0·3	3·6
	BEQ3	607	51 275	569	39 500	6·3	23
	Emeryville	281	29 800	275	27 995	2	6
	Average	323	35 436	307	28 752	7·5	18·8

Table 3. Maximum inelastic demand in longitudinal and transverse directions from ZEUS-NL response-history analyses (average of seven records)

	No damping: Top drift (mm)	No damping: Base shear (kN)	1% Rayleigh damping: Top drift (mm)	1% Rayleigh damping: Base shear (kN)	Diff. %: Top drift	Diff. %: Base shear
Average demand in transverse direction	328	108 702	313	88 749	4·6	18·4
Average demand in longitudinal direction	254	77 233	242	64 823	4·7	16·1

material strength (characteristic and mean) on the dynamic properties and seismic response was marginal. For the sake of brevity, only the above-mentioned results of the parametric study are presented.

The 3D models and the elastic analyses conducted using the SAP2000 and ETABS platforms are employed to verify the ZEUS-NL 2D model. The latter is exclusively used for estimation of capacity in the post-elastic range and prediction of inelastic seismic demand. Free vibration analyses are conducted using different programs to determine the dynamic characteristics of the tower. The first and second mode shapes of SAP2000 depicted in Figure 11 are in the transverse ($T = 3{\cdot}8$ s) and longitudinal directions ($T = 3{\cdot}4$ s), respectively. The first and second mode shapes produced from the ETABS model are also in the transverse ($T = 4{\cdot}05$ s) and longitudinal directions ($T = 3{\cdot}47$ s), respectively. It is clear that the longitudinal direction is stiffer compared to the transverse direction as a result of the orientation and dimensions of the central cores. It is shown that the two 3D idealizations validate one another despite the significant differences in modeling approaches and solution procedures.

In the transverse direction, ZEUS-NL eigenvalue analysis results illustrated in Figure 13 show that the elastic period of vibrations of frame B, which has lower stiffness, are much longer when compared to frame A. It is also worth noting that both frames A and B are loaded with 25% of the total mass of the structure. The average fundamental period of vibration in this direction is 3·7 s. In the longitudinal direction, the analysis shows a fundamental period of 3·2 s for frame C, which is loaded with 100% of the mass. Clearly, the period of the structure in the longitudinal and transverse directions is compatible with those obtained from the SAP2000 and ETABS 3D models. The difference between the 2D and the 3D models is less than 10%. This result validates the ZEUS-NL 2D analytical models

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developed in the current study and lends weight to the extensive inelastic results presented in subsequent sections.

8. SEISMIC DEMAND FROM DISTANT AND PROXIMATE EARTHQUAKES

Tables 2 and 3 summarize the global inelastic seismic demands estimated from ZEUS-NL response history analyses. Table 2 shows detailed results of one of the three framing systems analyzed in this study from seven records. The average demand in the longitudinal and transverse directions from all input ground motions is presented in Table 3. The results are given with and without Rayleigh damping. Based on the parametric study discussed above, the demand resulting from the 1% Rayleigh damping is employed to compare with the capacity of the structure, as shown.

Sample roof displacement and base shear time histories obtained from the ZEUS-NL inelastic dynamic analyses are presented in Figures 14 and 15. These are given for three records from the seven accelerograms employed in the current study. One artificial record from the small earthquakes set and two accelerograms from the large earthquakes set are shown. In general, the response obtained from the synthetic accelerograms is higher than that from the natural records. This observation applies to the two sets of earthquakes. The variability of the inelastic response is quite significant; whereas the large artificially generated records produce the maximum drift and base shear demands, the small accelerograms produce the lowest deformation and base shears, particularly in the Hollister record.

The second mode periods obtained from ZEUS-NL are 0·9 and 0·8 s for the transverse and longitudinal directions, respectively. It is clear from the response spectra of the ground motions selected to represent the site (Figures 8 and 9) that participation of this mode to the building response is significant compared with the fundamental mode. For the large earthquakes set, the spectral amplification corresponds to the fundamental mode period is about 0·1 *g* and 0·05 *g* for the artificial and Emeryville records, respectively. Amplification is about 0·35 *g* for the second mode period. Hence, the contribution of the second mode, despite its lower mass participation (25% as opposed to 50% for the first mode), is more pronounced. The spectra of the artificial records are also higher at the fundamental mode period, compared with Emeryville. On the other hand, it was expected that the small earthquakes will not amplify the fundamental mode since the ordinates of their spectra is minimal beyond 3·0 s. The amplification corresponds to the second mode period (~0·15 *g*), which is significantly lower than that for the large earthquakes set (~0·35 *g*). These observations become even more prominent after period elongation and stiffness degradation owing to the accumulation of damage.

The Fourier spectra of the top acceleration response depicted in Figure 16 also provide an insight into the inelastic response obtained from the two sets of records investigated in the current study. It is observed that the response under moderate close events is dominated by the contribution of higher modes since no peaks are shown for the fundamental mode. For the severe distant earthquakes, the participation of the fundamental mode is shown in the spectra of frames A and C, which have fundamental periods of 2·9 and 3·2 s, respectively. This is not observed in the spectra of frame B since its first mode period is quite long (4·5 s). The higher participation of the fundamental mode for the large artificial records compared with the natural one is also confirmed from their Fourier spectra. The aforementioned discussion justifies the response obtained from inelastic analysis and reflects the significant contribution of higher modes to the response. It also confirms the vulnerability of high-rise buildings to severe events generated at long epicentral distance.

9. CAPACITY VERSUS DEMAND

Inelastic pushover analysis is conducted using ZEUS-NL to estimate the ultimate capacity of the structure in two orthogonal directions. Owing to the significance of higher modes, a number of lateral load

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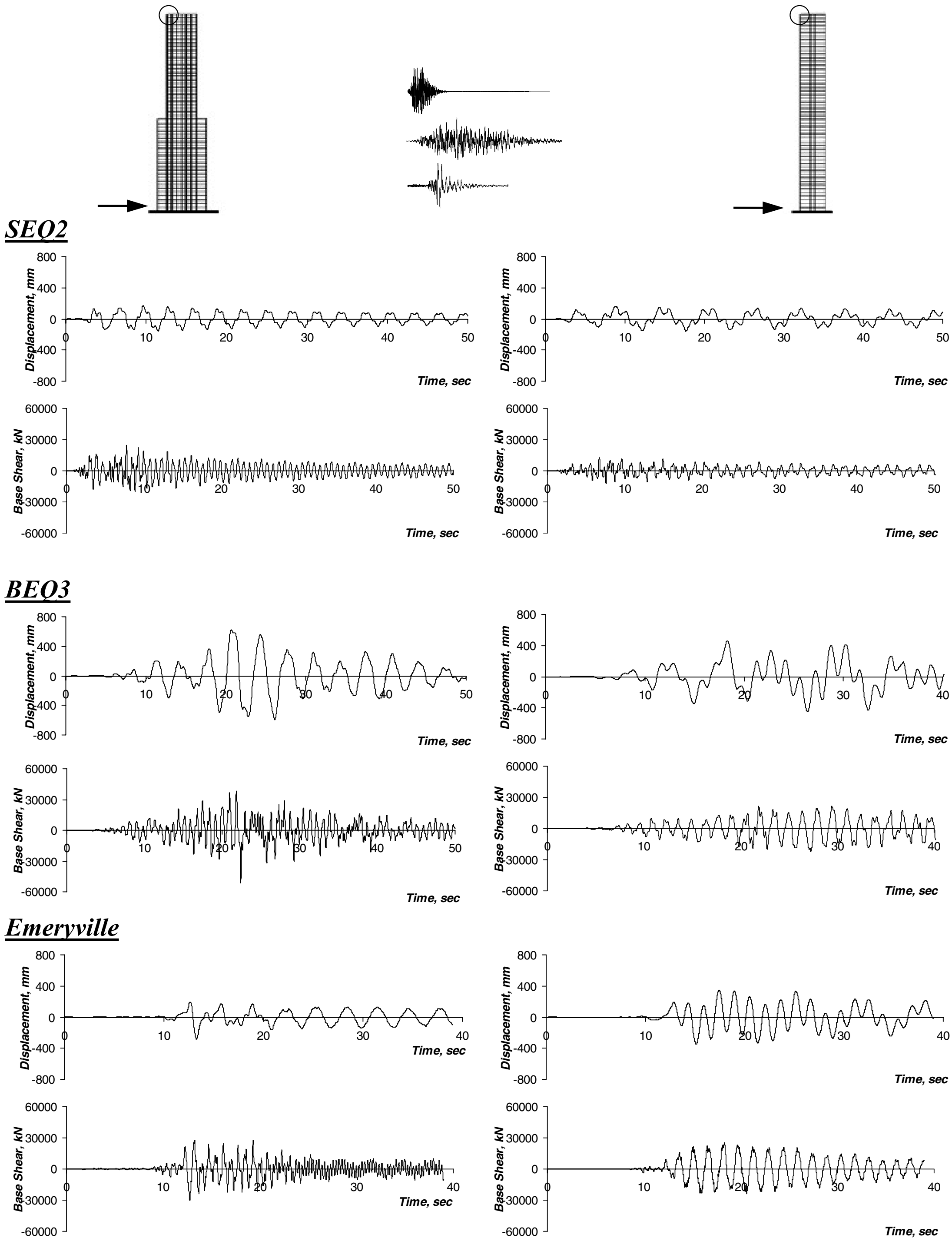

Figure 14. Sample displacement and base shear time histories of frames A and B. Both resist lateral forces in the transverse direction and each is loaded with 25% of the total mass

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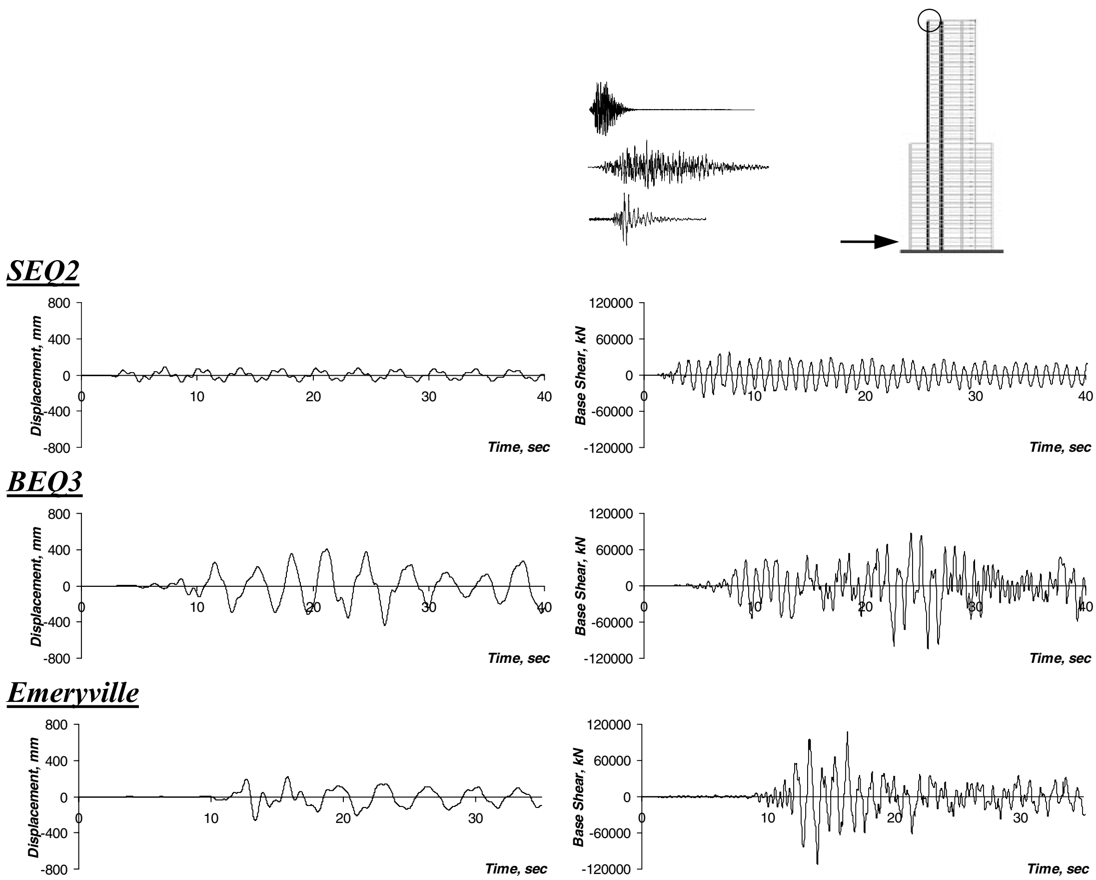

Figure 15. Sample displacement and base shear time histories of frame C, which is loaded with 100% of the total mass and resists solely the seismic load in the longitudinal direction

distributions are employed in the analysis. The predefined lateral load patterns are applied incrementally in a step-wise manner on the three framing systems until a specified target roof displacement of 3 m is reached (~1·6% total drift), taken as the ultimate limit state. The analysis was stopped at this target displacement as a result of the significant spread of yielding and cracking in all structural members including walls and cores; hence the ultimate limit state is considered to have been reached. Following the recommendations of modern design codes and guidelines (EC8, 2003; FEMA, 2000; ATC 40, 1996), the first load shape is a uniform pattern, representing lateral forces that are proportional to mass regardless of elevation. The second lateral load distribution is an inverted triangular load, which resembles the first mode shape. The predominant effect of the second mode of vibration is confirmed from the inelastic response history analysis. Hence, a third lateral load pattern representative of the second mode shape is also used. It was difficult to continue the analysis up to the target displacement when the lateral load from the second mode was used since the load is applied in two opposite directions, leading to early localized damage. However, this load is useful for estimating the elastic stiffness of the building when vibrating in the second mode.

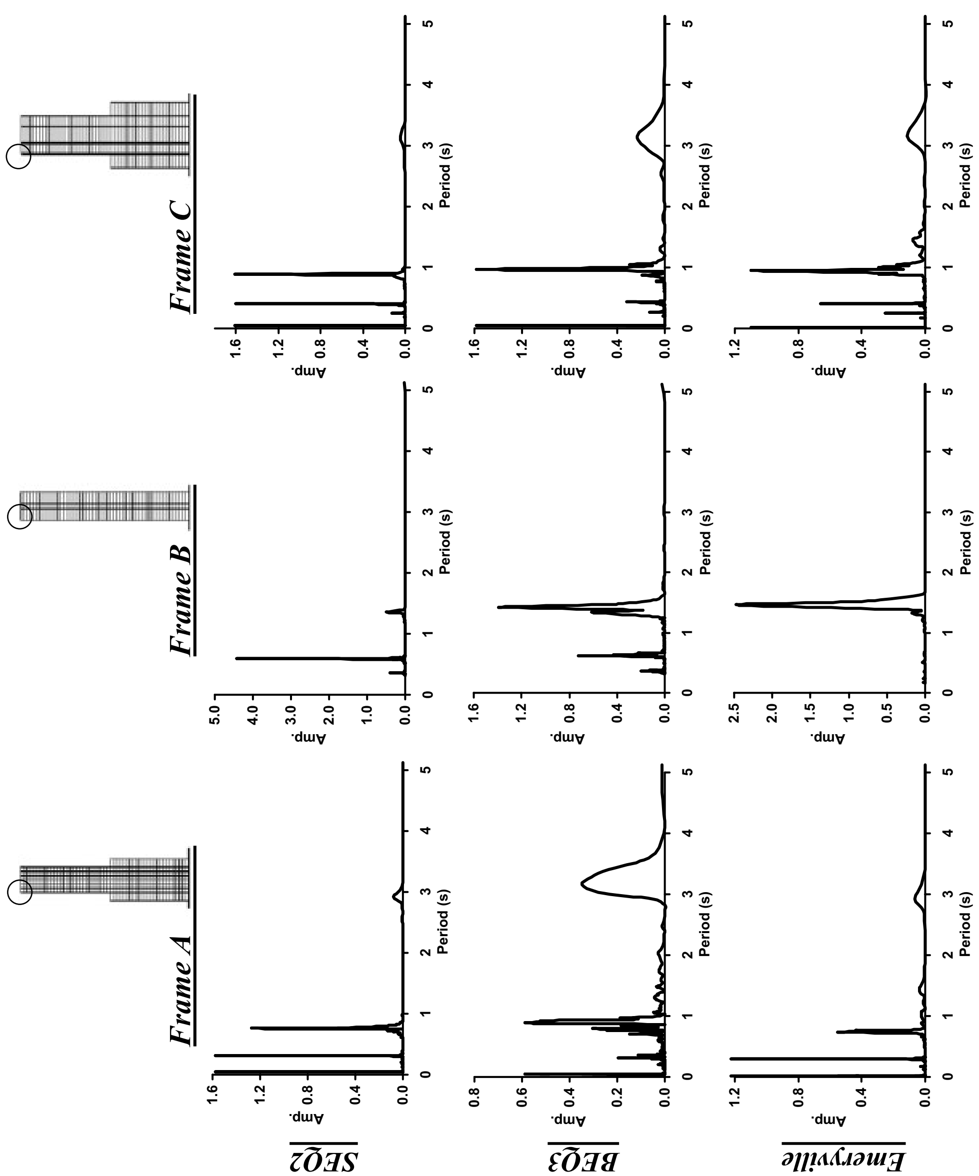

Figure 16. Fourier spectra of top response confirming contribution of different modes

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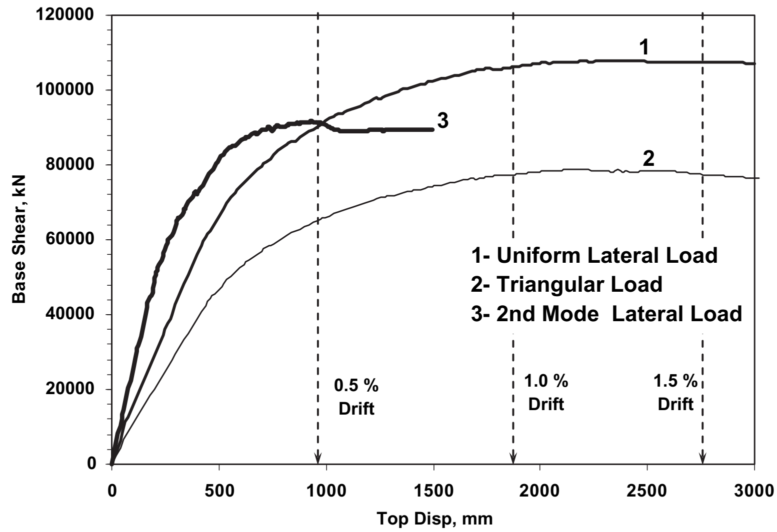

Figure 17. Comparison between the lateral capacities in the longitudinal direction obtained from inelastic pushover analysis using different lateral load patterns

The comparison depicted in Figure 17 shows that the uniform lateral load pattern reasonably estimates the initial stiffness and the ultimate capacity of the structure. The elastic stiffness of the tower under the uniform load is between that obtained from the first and second mode shapes. It was concluded by Mwafy and Elnashai (2001) that the inverted triangular load underestimated the capacity, particularly for structures significantly influenced by higher modes, while the uniformly distributed load gives better prediction of the ultimate strength. This conclusion is also in line with the conclusion of other studies carried out on high-rise buildings (e.g. Ventura and Ding, 2000). Based on this conclusion and the observation shown in Figure 17, it was decided to use the uniform lateral load to estimate the capacity of the three framing systems investigated. The capacity of frames A and B were combined to obtain the overall capacity envelope in the transverse direction.

Figures 18 and 19 show comparison of the lateral capacity in the two orthogonal directions versus the demand predicted from inelastic response history analysis. The average base shear demand is estimated from results of the seven accelerograms utilized in this study. Also, average results are depicted on the graphs for each of the two sets of records investigated here; three moderate earthquakes and the four severe records. It is shown that the initial stiffness of the structure in the longitudinal direction is higher than that in the transverse direction. This justifies the longer elastic periods of the latter direction compared to the former. It is also clear that the transverse direction is more vulnerable when compared to the longitudinal direction. The margin of safety, estimated as the ratio of ultimate capacity to average demand, is higher for the longitudinal direction. Generally, the average demand of the seven records is well below the lateral capacity in both the longitudinal and transverse directions.

Comparisons of the capacity and the average demand predicted from each of the two seismic scenarios investigated in the current study confirm the severity of the large distant earthquakes compared to the moderate proximate earthquakes. The inelastic behavior of all structural members is monitored during the response history analysis and cracking in confined concrete or yielding in main tensile reinforcement is recorded. In the transverse direction, local assessment indicates that yielding spreads in several coupling beams under the records representing far distant earthquakes, whereas only few

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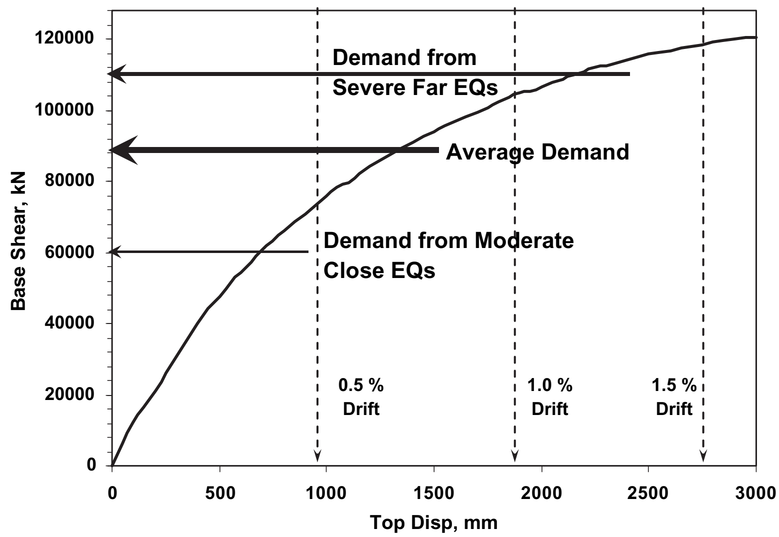

Figure 18. Capacity versus demand in the transverse direction

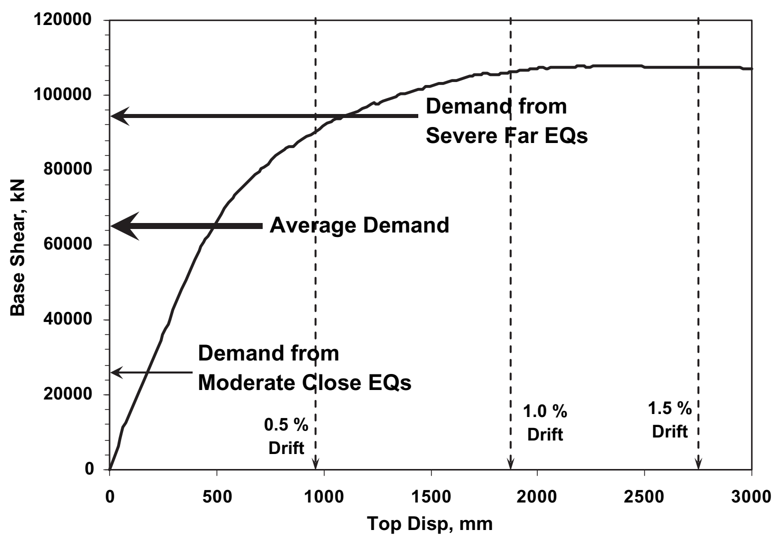

Figure 19. Capacity versus demand in the longitudinal direction

plastic hinges in these critical beams are observed from the moderate events. In the other direction, no inelasticity is observed from the latter records, while yielding spreads under the large earthquakes in coupling beams and slabs connecting columns to central cores. A plastic hinge is also observed at the base of an external column from one of the large earthquakes.

Monitoring the local response confirms the observations from global performance and indicates that the behavior of the structure under moderate records is mainly elastic. Severe earthquakes generate high ductility demands only in short coupling beams. For coupled walls, relative flexural stiffness of

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the coupling beams compared to adjacent walls generally results in early yielding and cracking in connecting beams. It was concluded (Mwafy, 2001) that after yielding top and bottom reinforcement and even crushing of concrete at the extreme fiber, the bi-diagonal reinforcement typically used in coupling beams will continue to provide an efficient load transfer mechanism. Therefore, the steel yielding observed in these beams, which is based on the strain of top or bottom reinforcement, is not considered an indication of significant damage. In considering that inelasticity only occurs in coupling beams, the local response under the severe distant earthquakes suggests that the response is just at or slightly beyond the yield limit state. This is in agreement with the global response shown in Figures 18 and 19.

Although it is unrealistic to presume that the structure will be subjected to a single seismic scenario, the response of the structure to large distant earthquakes, as opposed to moderate events, is alarming and reflects the need to consistently represent this scenario in the design procedure for similar sites. It also emphasizes the pressing need to reliably asses the inelastic response of high-rise structures in the final design phase under a diverse ensemble of accelerograms representing all anticipated earthquake input motions.

10. DESIGN VERSUS DEMAND AND CAPACITY

It is shown from comparison of the design base shear (2240 kN) calculated based on the UBC (1997) provisions and the capacity–demand graphs shown in Figures 18 and 19 that the average inelastic demand of the seven records is significantly higher than the design value. The observed demand-to-design base shear is 4·0 and 2·9 for the transverse and longitudinal directions, respectively. However, the overstrength exhibited by the structure is also significant. The observed overstrength factor (Ω_d) may be defined as the ratio of the ultimate (V_y) to the design (V_d) lateral strength (Elnashai and Mwafy, 2002; Mwafy, 2001). This yields an average overstrength factor of 5·2.

Elnashai and Mwafy (2002) recently suggested a measure of response termed 'inherent overstrength factor (Ω_i)'. The proposed measure connects the ultimate strength (V_y) to the elastic force (V_e) estimated by scaling up the design strength (V_d) using the response modification factor (R). Hence, $\Omega_i = V_y/(V_d \cdot R)$. In the case of $\Omega_i \geq 1{\cdot}0$ the response will be almost elastic under the design earthquake, reflecting the high reserve strength of the structure. Below this limit, the difference between the value of Ω_i and unity is an indication of the ratio of the forces that are imposed on the structure in the post-elastic range. Therefore, Ω_i, which is the inverse of the ductility component of the force reduction factor (R_μ), better reflects the anticipated behavior of the structure under the design earthquake owing to its explicit elastic response limit. The average inherent overstrength factor for the structure under consideration is 0·93. This implies that the inelasticity generated under the design earthquake is limited and the response of the structure is almost at the yield limit state. This observation is also confirmed from the inelastic member response discussed above.

11. RECOMMENDATION FOR DESIGN

It is clear from the above-mentioned discussion that if inelastic response history analysis is used in design without any reduction, it will yield an over-conservative and uneconomical design for the tower under consideration. The response parameters obtained from elastic analysis procedures employing the code recommended reduction factor (R) is significantly lower than those from inelastic analysis. Mwafy (2001) extensively investigated the inelastic response of 12 RC multistory buildings of different characteristics designed to EC8. The reliability of all these buildings was confirmed under the design PGA. Also, an acceptable response was observed under higher ground motions. Summary of average base shear demand obtained from inelastic response history analysis employing eight natural

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and artificial records scaled to the design PGA is given in Table 4, after Mwafy (2001). It is clear that for these medium-height buildings (8–12 stories) the observed *V*/*W* is higher than the design by a factor ranging from 1·24 to 2·37. This observation confirms that response parameters obtained from inelastic response history analysis should be reduced for design since the inherent ductility and reserve strength have not been entirely exploited at the design ground motion.

It is suggested in the current study to use a scaling factor equal to the response modification factor (R) revised by the inherent overstrength (Ω_i). It is also suggested to adjust the latter expression by a factor of 0·8 to account for the uncertainties in estimating the ultimate capacity from pushover analysis. Hence, to reduce the response parameters obtained from inelastic analysis, the scaling factor becomes ($0{\cdot}8R \cdot \Omega_i$). A limitation may be imposed to ensure that the resulting base shear is not less than that determined according to the equivalent static force procedure. Clearly, this reduction is not applicable to the inelastic displacement demand. This proposal rationale follows the philosophy adopted by modern seismic codes and overcomes the discrepancies in the provisions of the analysis procedures used in design. The proposed approach yields more economical design and clears the inconsistencies in the code provisions.

12. CONCLUSIONS

A moderate seismicity region, represented by Dubai, UAE, was selected to investigate the vulnerability of tall buildings to two different seismic scenarios: (i) severe earthquakes with a relatively long epicentral distance; and (ii) moderate events with short source-to-site distance. The two scenarios are feasible for several regions around the world and therefore investigated in this study. A hazard study for a construction site of a 54-story, 187 m height RC tower was carried out to define reliable criteria and input ground motions for design. The tower is meticulously modeled and analyzed under the two above-mentioned scenarios using state-of-the-art analytical tools. This comprehensive study leads to the following conclusions:

- The earthquake catalogue of the study area was extracted from different sources and covers the period from 734 to 2004. It indicates that the large earthquakes are with long epicentral distances, while those closest to the site are apparently small. Based on the derived hazard curve, a PGA of 0·16 g is assigned to the construction site for a 10% probability of exceedance in 50 years. This PGA is lower than the finding of the study of GSHAP (2004). However, the comprehensive earthquake data used herein and the consistent models employed render the above results more reliable.
- The characteristic difference in earthquake-induced response due to large distant events and moderate close earthquakes is predicted from their response spectra and the uncracked periods of vibration. Contrary to the latter, the former scenario has higher amplifications matching the second mode that dictates the response. It also amplifies vibrations in the fundamental mode that may be overlooked in design. The Fourier spectra of the top response also confirm that, despite the notable contribution and mass participation from the fundamental mode, the second mode is dominant under the effect of severe distant earthquakes.
- The local and global response from the severe distant earthquakes indicates that the response is slightly beyond the yield limit state, while it is entirely elastic under the moderate close records. Comparison of the global capacity and inelastic demand confirms that distant earthquakes produce the maximum global demand. A notable spread of yielding is also observed in the short coupling beams only under the large distant seismic scenario.
- Although high average demand-to-design base shear is observed, the overstrength exhibited by the structure (Ω_d = 5·2) is also significant. The inherent overstrength (Ω_i = 0·93) better articulates the behavior of the structure under the design earthquake due to its explicit elastic response limit.

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Table 4. Average base shear demand of eight ground motions observed from inelastic response-history analysis for 12 RC buildings analyzed by Mwafy (2001)

Ref.	Structural system and height	Design PGA (g)	Design ductility	Max. observed base shear (KN)	Max. (V/W) Observed	Max. (V/W) Observed/Design	R	Overstrength (Ω_d)	Overstrength (Ω_i)
1	8-story frames	0·30	High	9 007	0·39	2·24	4	2·35	0·59
2			Medium	9 168	0·40	1·71	3	2·2	0·73
3		0·15		4 908	0·22	1·93	3	2·51	0·84
4			Low	4 722	0·21	1·24	2	2·1	1·05
5	12-story frames	0·30	High	11 105	0·29	2·37	5	2·42	0·48
6			Medium	11 316	0·30	1·89	3·75	2·06	0·55
7		0·15		5 116	0·14	1·78	3·75	2·4	0·64
8			Low	5 661	0·16	1·30	2·5	2·14	0·85
9	8-story dual	0·30	High	9 446	0·43	2·13	3·5	2·81	0·8
10			Medium	10 643	0·48	1·85	2·625	2·65	1·01
11		0·15		4 444	0·22	1·69	2·625	2·74	1·04
12			Low	4 966	0·24	1·27	1·75	2·3	1·31

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The inherent overstrength parameter indicates that the inelasticity generated under the design earthquake is limited and confirms that the response slightly exceeds the yield limit state.

- The uncracked periods of vibration calculated from the verified 2D and 3D analyses are significantly higher than those estimated from the code recommended expressions, which appear to be over-conservative for high-rise high-strength RC buildings. The equivalent static force analysis is more conservative and produces higher story shears than that from the modal response spectrum analysis employing the UBC design (scaled) spectrum.
- Response parameters obtained from inelastic dynamic analysis may be reduced for design since the inherent ductility and reserve strength have not been entirely exploited at the design ground motion. It is suggested in the current study to use a scaling factor, equal to the response modification factor adjusted by the inherent overstrength and a factor to account for uncertainties in estimating the ultimate capacity ($= 0{\cdot}8R \cdot \Omega_i$). A limitation may be imposed to ensure that the resulting base shear is not less than that determined from the equivalent static force procedure.

It is important to stress that the response of the investigated structure, which has observable reserve strength, is alarming under the severe distant records and reflects the need for considering this seismic scenario for similar structures and sites. Whereas significant reduction in design forces obtained from inelastic analysis is recommended, the margin of safety remains adequately conservative. The force reduction proposal given above follows the philosophy adopted by modern seismic codes and overcomes any inconsistencies in the code provisions.

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