The response of tall buildings to far-field earthquakes and the case of a 49-storey steel building

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Abstract: This paper investigates the seismic response of an instrumented 49-storey steel structure in San Francisco to weak, far-field, and strong, near-field ground motions. The instrumentation records obtained during the 1989 Loma Prieta earthquake are used to verify the accuracy of the predictions of the time-history analysis of the model. The ChiChi-002 ground motion record from the 1992 Chi-Chi earthquake in Taiwan (PGA = 0.08 g), representing a 'weak, far-field' earthquake and the record from the 1994 Northridge-Newhall earthquake (PGA = 0.60 g) representing a 'strong, near-field' earthquake were used in the study. The results showed that the force, acceleration, and displacement responses of this long-period structure to the 'weak far-field' ground motion are much larger than its response to the 'strong, near-field' ground motion. Also, the response attenuates at a slower rate for the weak, far-field earthquake, indicating the possibility of greater damage, both to structural and non-structural elements, during the earthquake. Interim seismic design recommendations are formulated to address this issue in the design of tall buildings with long periods.

Keywords: structural engineering; earthquake engineering; nonlinear time-history analysis; instrumented buildings; far-field earthquakes; seismic design; tall buildings; near-field earthquakes; dynamic resonance; long-period motion; seismic performance; drift control; seismic response.

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1 Introduction

Many large, urban cities around the world are responding to a dearth of buildable space by approving the construction of very tall structures. According to the Council on Tall Buildings and Urban Habitat, there were only 263 buildings higher than 200 m in the year 2000 (Brass et al., 2012), but by the year 2015, 1,055 buildings were taller than 200 m (Gabel et al., 2016). In 2015, construction of tall buildings reached an all-time high when 106 buildings taller than 200 m were constructed in a single year. Due to their height and dynamic properties, these tall structures exhibit complex seismic behaviour. Even the slightest modification in design parameters of these structures may initiate significant change in their seismic response. Therefore, extensive studies and careful monitoring can help in understanding the seismic behaviour of such structures and aid in refining the existing seismic design practice of tall buildings.

In seismic design, site-to-source distance is a major factor influencing the entire design process. The first step in seismic hazard analysis is to identify possible sources of the earthquake. In a deterministic approach, the source-producing maximum intensity measure at the site is selected. The intensity measures used are peak ground acceleration (PGA), peak ground velocity (PGV), peak ground displacement (PGD), 5% damped spectral ordinates at selected periods, etc. The maximum of these peak values is usually determined by considering the active fault nearest to the site (Reiter, 1991). In the probabilistic seismic hazard analysis (PSHA), ground motion prediction equations (GMPEs) are used to estimate the distribution of the above-mentioned intensity measures. However, the GMPEs that estimate peaks values (i.e., PGA, PGV and PGD), have a positive correlation to distance, i.e., the nearer the source, the higher the estimates. Therefore, the impact of long-distance earthquakes often remains unaccounted for when the peak values are used as intensity measures.

Long-distance or far-field earthquakes usually are low-amplitude events. They may occur due to faulting or result from human-induced activity (e.g., fracking). When a wave travels a relatively long distance from its initiation point, peak acceleration intensity diminishes, but the long-period surface waves elongate. This elongated long-period surface wave of far-field motions can produce a strong response in structures that have a long fundamental period of vibration due to resonance. The effect of such low-amplitude, long-period earthquakes on the tall structures needs to be investigated to understand what aspects of the behaviour are sensitive to design issues and whether they need to be incorporated into the seismic design codes.

The main aim of this study is to investigate the effects of long-distance, long-period ground motions on seismic response of tall buildings, which usually have relatively long-periods of vibration. We studied the effect of a far-field, low-acceleration amplitude, long-period earthquake, and a near-field, high-acceleration amplitude, with a relatively short dominant period on the simulated model of an existing 49-storey steel structure. The methodology, details of the model, and the results obtained from this study are presented below.

2 Background

Several studies have been carried out over the last two decades to understand the response of tall structures when subjected to seismic forces (Astaneh-Asl et al., 1991a, 1991b, 1991c; Chen et al., 1992; Çelebi, 1993; Bonowitz and Astaneh-Asl, 1994; Fan et al., 2009; Over et al., 2010; Astaneh-Asl, 2013; Muin and Astaneh-Asl, 2013; Christopoulos and Montgomery, 2013; Leung et al., 2016; Muin and Mosalam, 2017). One common practice is to assess a structure's performance under near-field ground motions. Near-source or near-field ground motions exhibit a pulse-like motion with aggregated energy, particularly in the direction of the fault-plane rupture (Bozorgnia and Bertero, 2004). For detailed information on the response of structures subjected to a near-field earthquake, see Anderson and Bertero (1987), Hall et al. (1995) and Zhang and Iwan (2002). The relatively large deformation demands on the fixed-base buildings and base-isolated buildings have been of particular concern (Bertero et al., 1978; Taniguchi et al., 2016).

The response of structures to a far-field earthquake was first reported in 1952 when tall buildings 100–150 km away from the epicentres of the Kern County earthquake experienced significant shaking (Hodgson, 1964). In the devastating 1967 Caracas earthquake, towns as far as 80–100 km away from epicentre reported structural damage and casualties, bringing up the total number of deaths to 236 (Papageorgiou and Kim, 1991). The 1985 Mexico City earthquake is the most noteworthy example of the impact of a far-field earthquake where Mexico City suffered extensive damage, even though it was situated almost 350 km away from the coastal epicentre (Anderson et al., 1986). For the first time, a mid-rise, 23-storey steel structure collapsed, and two other identical towers were on the verge of collapse in response to this earthquake (Astaneh-Asl, 1986). In this case, the damage originated from ground motion amplification and resonance due to soil conditions (Çelebi et al., 1987). Another interesting case is that of the 2011 Tohoku earthquake where a far-field earthquake induced resonance because the fundamental period of vibration of structures fell close to the predominant period of the ground motion. Long-span, long-period bridges, and cable-stayed bridges located hundreds of kilometres away from the epicentre suffered damage (Çelebi et al., 2014).

To date, there has been little research on the response of tall buildings to the far-field earthquakes. Most investigations in the past have focused on the resonance effect around the 2 sec period. Tall structures in deep sediments have fundamental periods of vibration in the range of five to eight seconds, and the response of the structure in this range of period is yet to be well understood. After the 2011 Tohoku earthquake, more attention has been devoted to this issue (Takewaki et al., 2011; Minami et al., 2013), including research into the effect of far-field earthquakes on base-isolated structures (Ariga et al., 2006) and uncertainty in predicting long-period ground motion (Takewaki et al., 2013). Most of these studies have focused on instrumented buildings in Japan, and the effect of far-field earthquakes on new buildings with improved seismic technology (such as dampers or base isolation) (Çelebi et al., 2014). Several recent projects have focused on studying the seismic response of tall buildings and long-span bridges (Astaneh-Asl, 2013). This paper summarises one of the projects (Muin and Astaneh-Asl, 2013).

As mentioned earlier, the primary goal of this study was to investigate the response of an existing 1977-era 49-storey tall steel structure located in San Francisco, California, to strong, near-field, and relatively much weaker far-field earthquakes, and comparing its response. Current structural engineering practice and seismic codes demand that these structures be designed for strong near-field earthquakes, with the assumption that such weak and long distance earthquakes will do little to no damage to structures. This study demonstrates that relatively low damping (~2%) combined with a long fundamental period of vibration of tall buildings can make these structures susceptible to seismic damage from far-field, weak, but long-period earthquakes.

3 Research methodology

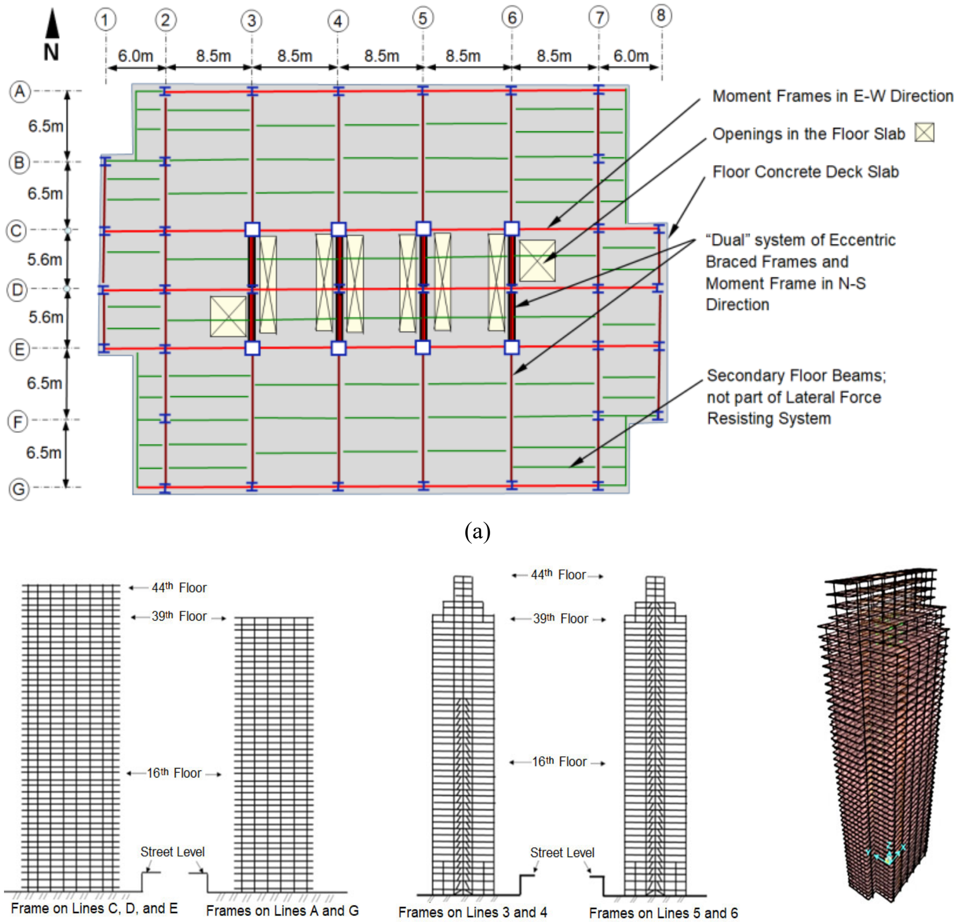
The study was carried out by developing a three-dimensional structural model of an existing 49-storey steel structure. Two representative ground motions were used herein: the first was a far-field, low-amplitude earthquake, and the second was a near-field, relatively strong earthquake. Comparisons of response parameters such as drift and base shear are presented to understand the behaviour of the structure under these two different excitations.

4 The structure

4.1 Description of the existing structure

A 49-storey building located in downtown San Francisco was used in this investigation. It was designed and constructed between 1977 and 1979 following the 1976 Uniform Building Code (ICBO, 1976). Due to the importance of the building, design response spectra were also used, and the structural design exceeded code requirements. This was the first tall building with eccentrically braced frames built in the USA. The building is a high-rise tower with 47 floors above grade and two below-grade basement levels. The floor-to-floor height of this tall structure is 3.65 m, with a total height of 171 m above grade. The building is primarily used for office occupancy with 41 stories of office floors. The top six stories of the building have a reduced floor area, with the top two floors housing mechanical equipment. There is another mechanical floor just below the first office floor (or actual third floor), and below that, there are three floors for commercial use. Figures 1(a), 1(b) and 1(c) show typical floor framing plan and frame elevations of the building.

Figure 1 Details of the structure under consideration, (a) typical floor framing plan (b) moment frames (east-west direction) (c) eccentrically braced frames (north-south direction) (d) 3D view of the model (see online version for colours)



(a)

(b) (c) (d)

The lateral framing system consists of five steel moment-resisting frames (SMRF) in an E-W direction. In N-S direction, four eccentric braced frames (EBF) were used with bracing in the middle two bays along with four-moment frames. Though moment frames are continuous to the roof level, the braced frames terminate at the floors below the roof, see Figure 1. There are no braces above 41st storey for the two frames and above the 29th storey for the remaining two. The eccentric brace frames have two diagonals in each bay, with 1.37 m-long shear links at the top and bottom of each diagonal. The floor system consists of a composite deck with 6.35 cm of concrete over a 7.6 cm metal deck connected by studs. The foundations are composed of 1.52 m-thick reinforced concrete mat foundation supported on the reinforced concrete piles.

The building was instrumented by the California Strong Motion Instrumentation Program (CSMIP). During the 1989 Loma Prieta earthquake, valuable records were collected by the instrumentation at the foundation level and some floors (Shakal, 1989).

4.2 Model development by SAP2000

The computer program SAP2000 (v. 15) was used to develop and analyse the three-dimensional model of the tower. The model was developed based on the available drawings and detailing of the structure, see Figure 1(d). Note: some elements of the structure are modelled in a simplified way with the assumption that these modifications will not change the global response of the structure. All frame sections are assigned according to the structural drawings. All beam-to-column connections are moment connections. The end connections of the bracings are pin connections. The composite floor slab of 6.35 cm concrete and 7.6 cm corrugated steel deck is modelled as a 10 cm concrete deck slab by averaging the corrugated part and adding the average thickness to the solid part. El-Dardiry and Ji (2006) have shown that this simplification results in a reasonably accurate representation of the corrugated deck slab with a simple solid slab. The floor slab is not considered at the staircases and elevator shafts. Floor slabs are modelled as rigid diaphragms with storey mass at the centre. It is assumed that the model cannot displace horizontally up to level 3. This is a reasonable assumption based on the fact that there is a three-storey shopping mall surrounding the tower up to that level. The base of the model is considered at the second basement (B level).

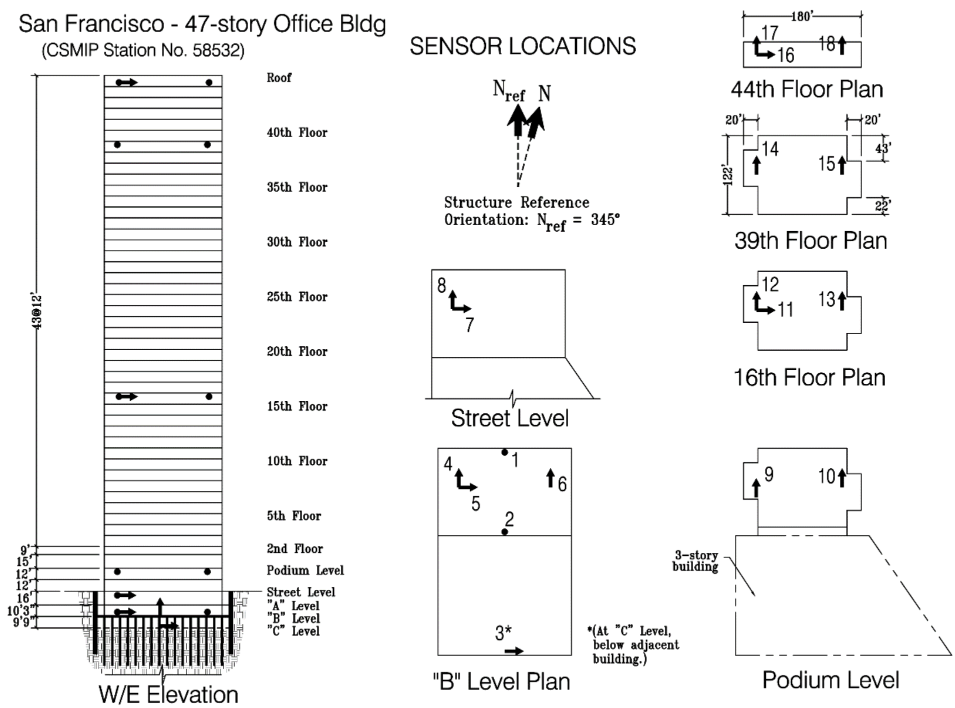
The model considered both the dead load and live load. For simplicity, the dead load on all floors was assumed to be the same with a value of 4.98 kPa (Chen et al., 1992). A distributed live load of 2.4 kPa was applied, which is standard for office occupancy. The mass source is defined by the dead load and 25% of the live load (ASCE, 2016); these loads were not combined during the analysis. The number of modes used in the analysis was 23, which accounted for 99% of the effective mass. The damping ratio in the analysis was 2%. Previous studies (Chen et al., 1992) indicate that the damping ratio for this structure lies between 1.7% and 3%. The dynamic equilibrium equation was solved by using the direct integration method. The Newmark method with $\beta = \frac{1}{4}$ and $\Delta t = 0.001$ was used in order to ensure stability.

4.3 The strong motion instrumentation of the building

The CSMIP was established in 1972 as a California State Agency to obtain vital earthquake data for the engineering and scientific communities through a statewide network of strong-motion instruments. It is operated by the California Department of Conservation in cooperation with USGS/National Strong Motion Program. The building under consideration was instrumented by CSMIP with 18 accelerometers at seven different floors and in different directions (Figure 2).

Placement of the accelerometers in the CSMIP depended on various factors, including building configuration, floor planning and structural framing (Rojahn and Matthiesen, 1977). The goal was to capture the response of the structure as completely as possible with limited sensors. Figure 2 also shows accelerometers at level B, street level, podium level, 16th floor, 39th floor and 44th floor. Level B is the basement level; therefore, acceleration at this level is considered as the ground motion for the structure. This level has five accelerometers, three of which capture acceleration in two horizontal orthogonal directions and two sensors record acceleration in the vertical direction. All other floors record acceleration in two horizontal orthogonal directions along the longitudinal and transverse direction of the building.

Figure 2 Strong motion instrumentation scheme



Source: Shakal et al. (1989)

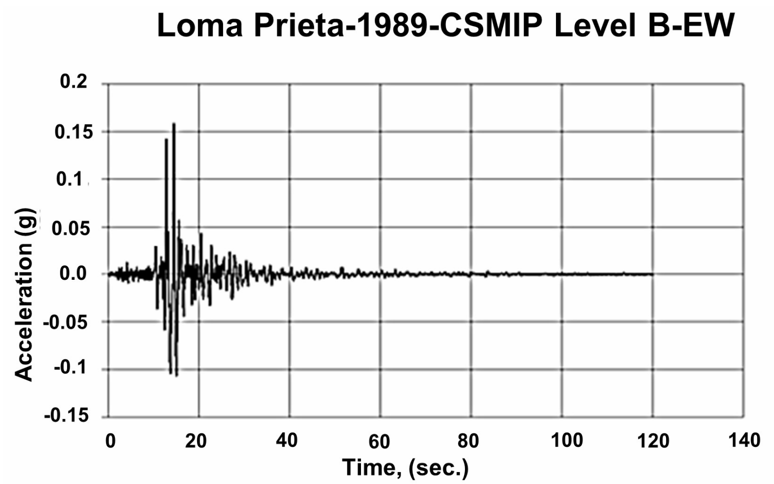
5 Ground motions

This study utilised the ground motion input in two different ways. First, it was used to validate the finite-element model developed and described within. Second, it was used to compare the response of the structure to the near-field, strong and far-field, weak ground motions. Three different earthquake records were used in this study.

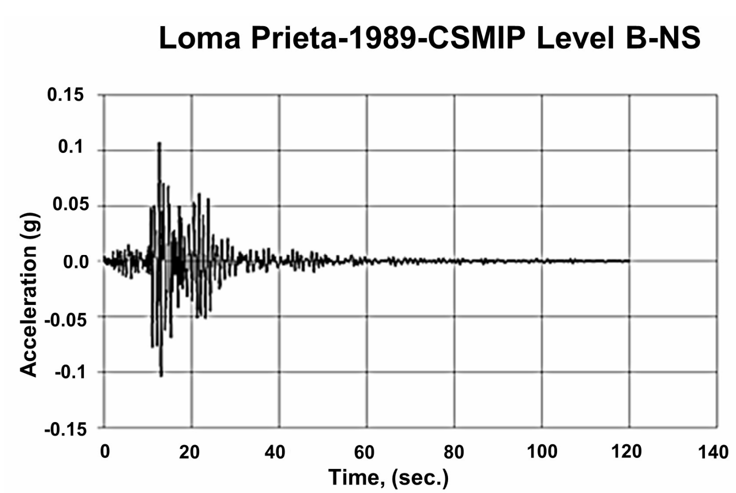
Data from the 1989 Loma Prieta earthquake was used to verify the accuracy of predictions of the three-dimensional finite-element model of the structure. As mentioned earlier, the building was instrumented by CSMIP and recorded the response of the structure during this earthquake. The recorded data has been since used to evaluate the dynamic properties of the building (Çelebi, 1993; Astaneh-Asl et al., 1991a, 1991c). The earthquake originated in the Santa Cruz Mountains, which 95.6 km from the building. It had a horizontal PGA of 0.15 g, peak structural response of 0.48 g, and a predominant period of 1 sec, see Figure 3.

For the second objective of evaluating the response of the structure to the far-field earthquake and to near-field ground motions, near-field and far-field ground motions needed to be selected. Although there is a dispute among researchers about the distance range of near-field and far-field ground motions, ground motion is considered near-field if the PGA is above 0.2 g and acceleration or velocity time history exhibits a distinct pulse. Low amplitude, low frequency, and longer duration are the signature of the far-field ground motions. The attenuation relationships shown in Figure 4 (Graizer and Kalkan, 2007), demonstrate that the PGA is below 0.2 g after about 20 km fault distance (without the basin effect).

Figure 3 Ground motion used to validate the model, (a) acceleration time history E-W (b) acceleration time history N-S (c) frequency spectrum

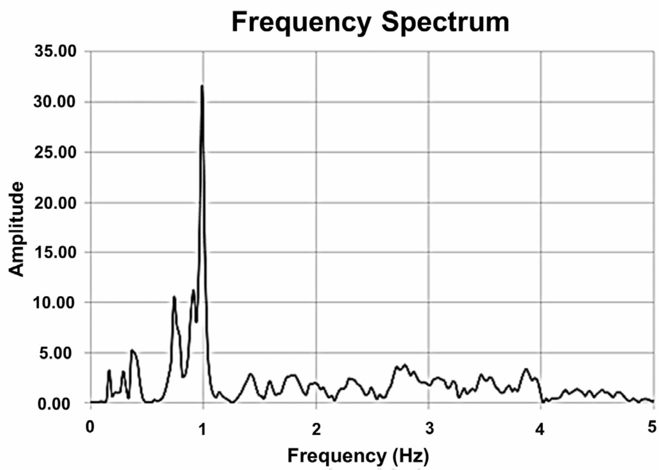


(a)



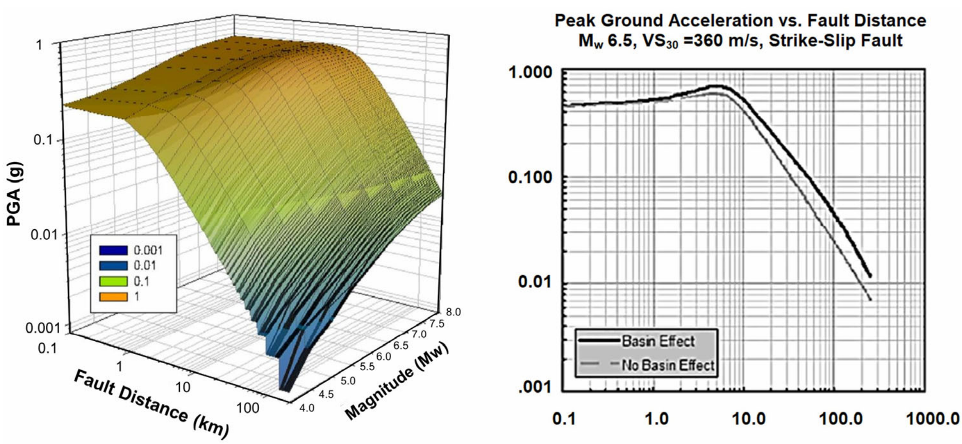
(b)

Figure 3 Ground motion used to validate the model, (a) acceleration time history E-W (b) acceleration time history N-S (c) frequency spectrum (continued)



(c)

Figure 4 Attenuation relationship (see online version for colours)



Source: Graizer and Kalkan (2007)

This study considered ground motion records obtained at stations with a fault distance more than 20 km, a PGA less than 0.2 g, and of the long duration of vibration as far-field ground motions. Another condition considered while selecting representative far-field ground motion was the predominant period of the ground motion. A ground motion record with a predominant period close to the fundamental period of vibration of the structure being studied is more significant than others due to the likelihood of the resonance effect (Chopra, 2001). Therefore, ground motions having a predominant period around 4 sec to 6 sec were considered more suitable for this study where the fundamental period of vibration of the structure was 5.2 sec.

In this study, the three components of the ground motion recorded for the 1999 Chi-Chi, Taiwan, earthquake at the CHY002 station (ChiChi-002) were used as the representative ground motion for far-field, low-intensity, long-period earthquake. Figure 5 shows the two horizontal acceleration components of this ground motion record. This is the same ground motion that was used for the first time in the studies of a 54-storey building in Los Angeles with steel moment frames in both directions (Over et al., 2010). The closest distance of the CHY002 station to the ruptured fault is 28 km. The PGA recorded at this station is 0.117 g in E-W direction and 0.14 g in N-S direction, making it an ideal candidate for a far-field, low-intensity earthquake.

Figure 5 Horizontal acceleration records of the far-field (top) and near-field ground motions used in the study (see online version for colours)

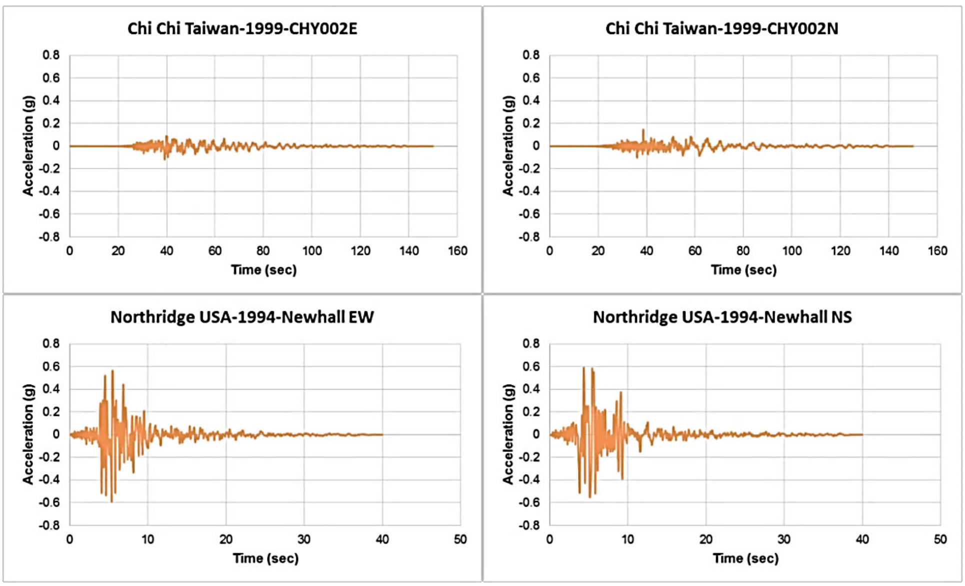


Figure 6 Pseudo-acceleration spectra of the far-field and near-field (dashed line) ground motions used in the study

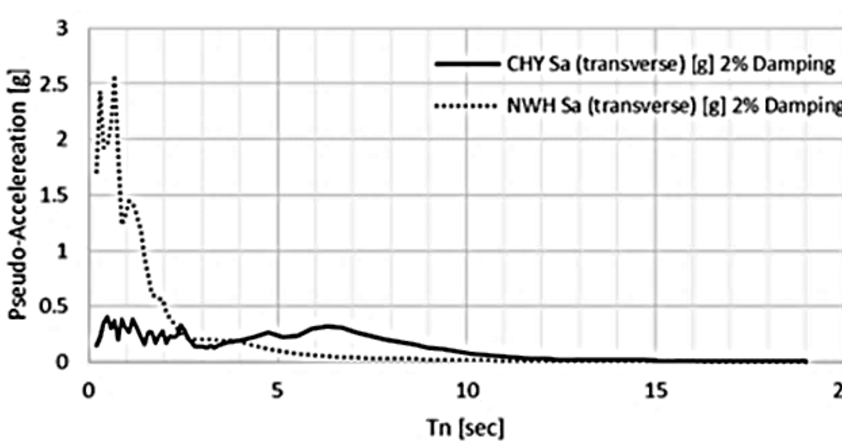
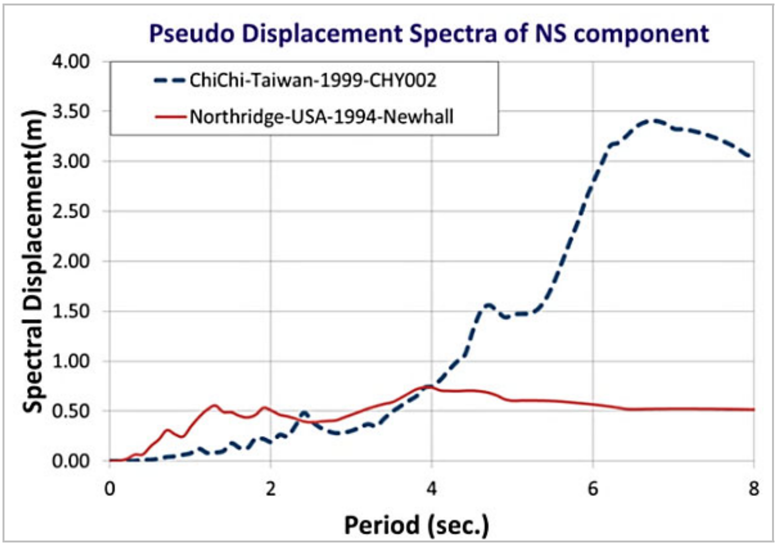
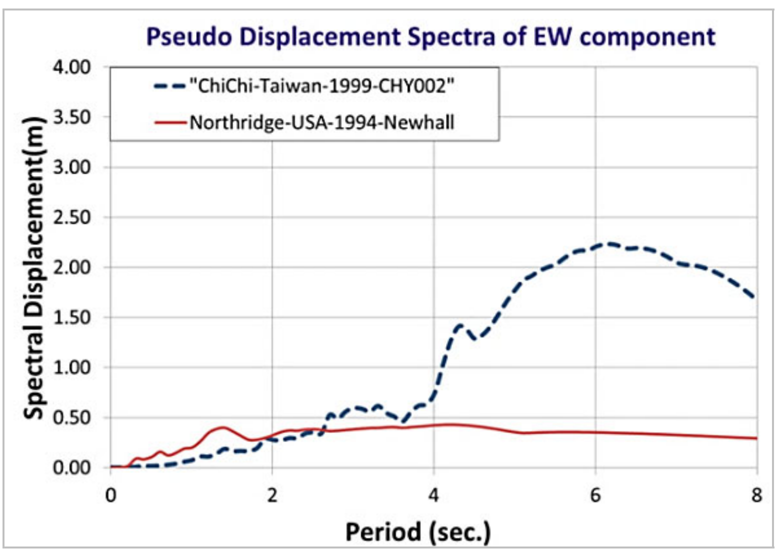


Figure 7 Pseudo-displacement spectra of the far-field and near-field (dashed line) ground motions used in the study (see online version for colours)



(a)



(b)

The 1994 Northridge ground motion recorded at the Newhall fire station was used in this study to represent typical near-field, relatively strong and short-period ground motions. This station is located 3.2 km from the fault with PGA of 0.60 g in both E-W and N-S direction. Response spectra for the 2% damping for both ground motions are shown in Figure 6. Note: the Northridge-Newhall record has a maximum spectral acceleration of 2.5 g, which is six times more than the maximum spectral acceleration of the ChiChi-002 ground motion records at 0.40 g. The far-field ChiChi-002 spectrum has a relatively high value for periods greater than 5 sec, while the Northridge-Newhall record has very small spectral acceleration value beyond 5 sec. The displacement spectra in Figure 7 clearly show the long-period effect of ChiChi-002 record for both components.

6 The response of structure and model to Loma Prieta earthquake

Model validation is done by comparing the data recorded during the 1989 Loma Prieta earthquake in the building with the response obtained from a time history analysis using the model presented herein. Accelerations recorded at Channel 4 and Channel 5 were used as input ground motion for the time history analysis of the model, see Figure 2. Displacement records show that all maximum and minimum values occur within the first 50 sec of the shaking; therefore, for this study, all analysis was done considering the first 60 sec of the record.

The periods of vibration of the structure based on using the recorded data agree well with the periods estimated from the results of the time history analysis using the model. The FFT of the transfer function shows the fundamental period to be 6.35 sec in E-W direction and 5.27 sec in N-S direction, see Figure 8. Modal analysis of the structural model established first-mode periods of 6.53 sec in E-W direction and 5.14 sec in N-S direction. The second-mode periods found from the measured data were 2.08 sec and 1.92 sec in E-W and N-S direction, respectively. The modal analysis showed values of 2.25 sec and 2.09 sec, respectively. The third mode notes similar performance (1.05 sec and 0.97 sec from recorded data corresponding to 1.23 sec and 1.0 sec from the modal analysis).

Figure 8 FFT of transfer function during the Loma Prieta earthquake

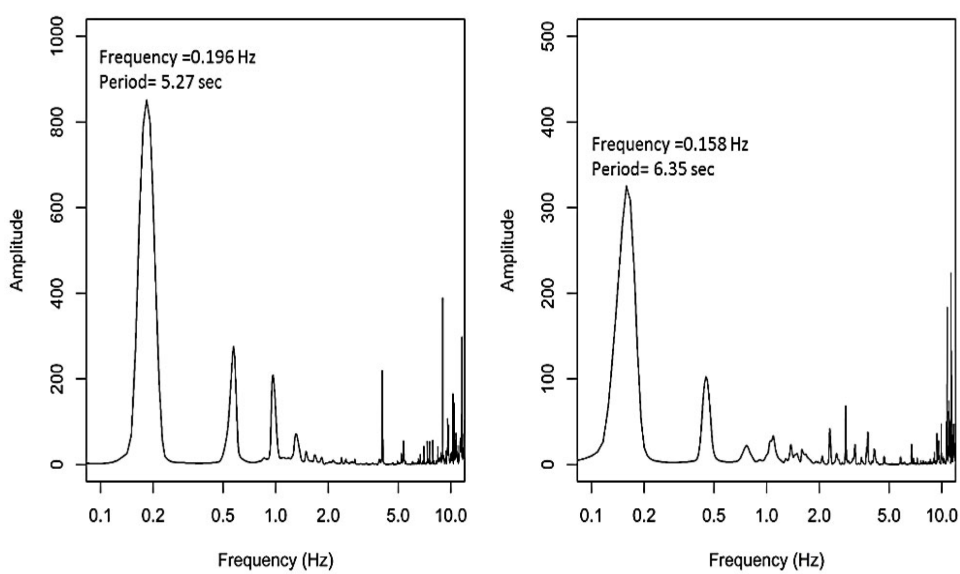
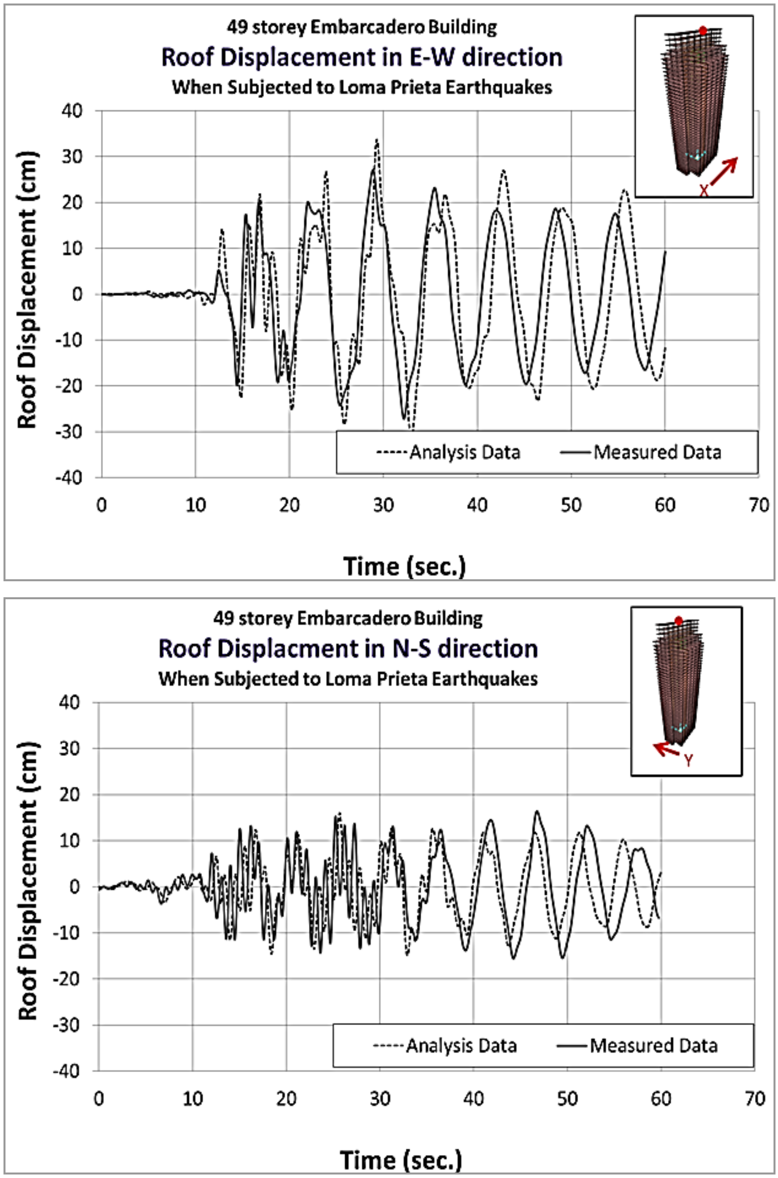


Figure 9 Roof displacement response of model and structures (see online version for colours)



Displacement response of measured data and analysis data also show a good correlation. The plot of the displacement time history in Figure 9 at the 44th floor for measured response compared to the model shows that the response of the moment frame (E-W) is initially very sensitive to the ground shaking. The peaks predicted by the model exceed slightly the measured peaks. Measured data show that peak displacement in E-W direction is 27.2 cm, and the models predict that the peak displacement is 29.2 cm, indicating that the value of mass used in the model may have been less than the actual mass in the building at the time of the earthquake. Model response and measured response overlap for some time until a time is observed in the model in the E-W direction, that part of the structure designed with moment frames. The response of the eccentrically braced frames in the N-S direction shows a very good correlation with the measured data during the shaking. A peak displacement value of 15 cm is measured in this direction compared to a peak displacement value of 16 cm obtained by the model. There is a hint of the time-lag in this direction as well, although it is, very small compared to the time-lag in the E-W direction. It is assumed that this time-lag is primarily due to the difference in the period estimates between the model and the actual structure; the E-W direction had a difference of 0.18 sec compared to the N-S direction where the difference in estimating the period is just 0.13 sec.

The model developed herein has periods in the vicinity of the actual period of the structure in both directions and can predict the peak values closely; therefore, it is assumed that the model can represent the dynamic properties of the structure reasonably well and can be used for further studies. However, the output of the model will have some time-lag in the E-W direction from the actual event.

7 The response of the structure to near-field and far-field (long-distance) earthquakes

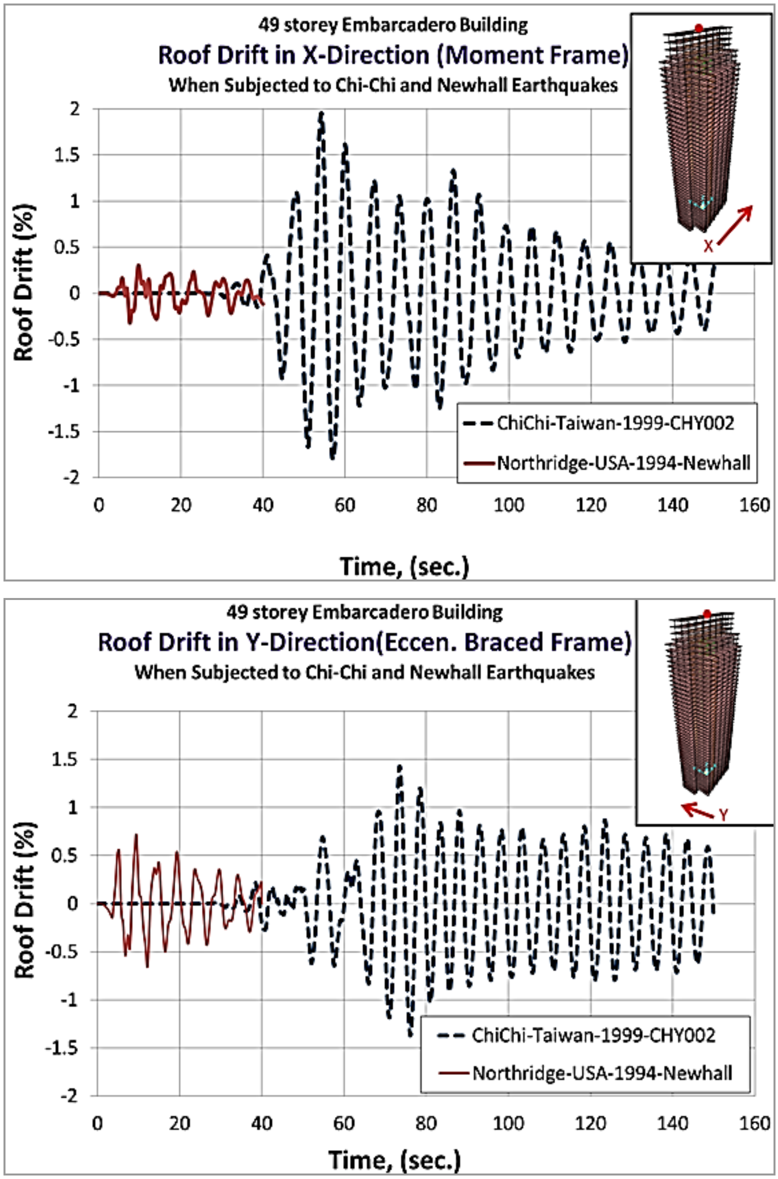
7.1 Displacement response

The structural model was subjected to the Northridge-Newhall and the ChiChi-002 acceleration records, and the responses of the structure to these near-field-strong and far-field-weak ground motions are compared. The horizontal components of the acceleration for both motions were shown in Figure 5 earlier. The roof drift response is shown in Figure 10 and reveals behaviour worth further investigation.

The strong, near-field Northridge-Newhall ground motion with PGA of 0.6 g generates maximum roof drift of only 0.7% for the braced frame direction (N-S), and for the moment frame direction (E-W) an even smaller value of 0.4%. On the other hand, for the weak, far-field ChiChi-002 ground motion, the roof drifts were 1.5% in the braced frame direction (N-S) and 1.85% in the moment frame direction (E-W). The values for the weak, far-field ground motion are more than twice the values for the strong near-field earthquake.

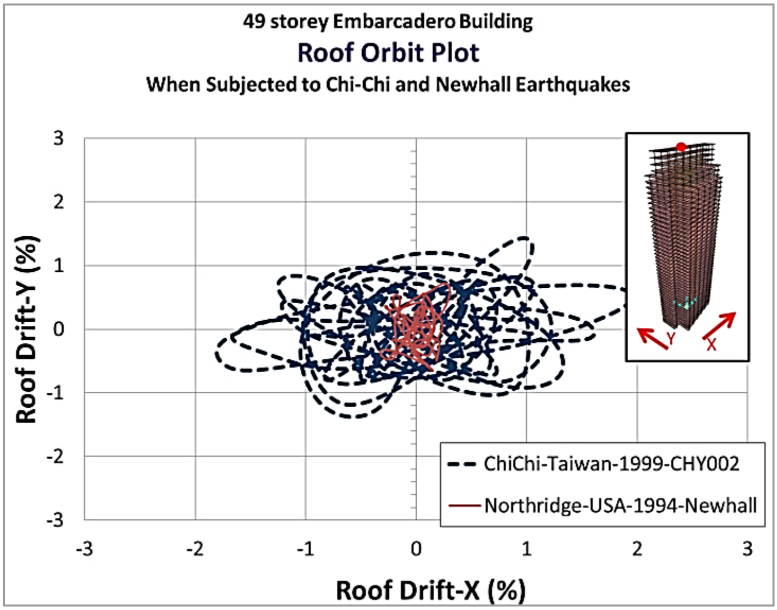
According to Liu and Astaneh-Asl (2004), structures with the pre-Northridge welded moment connections such as this structure will develop plastic hinge formations at about 1.2% drift and may reach ultimate moment capacity of around 1.5%. The structure under consideration is a pre-Northridge building; therefore, it may suffer structural damage in the event of a far-field earthquake. Non-structural damage is also expected to be greater for the far-field earthquake compared to damage from near-field earthquakes. Under such drift values, damage may occur in the precast concrete cladding, in-fill partitions, and glazing (Goodno, 1979). Interestingly, when subjected to the Northridge-Newhall ground motion, the structure has greater displacement demand in the braced-frame direction than the moment frame direction, contradicting what is predicted from the modal analysis.

Figure 10 Roof drift response of the model to near-field, strong and far-field, weak earthquakes (see online version for colours)



This is also evident from the roof orbit plot shown in Figure 11, which shows the structure vibrating predominantly in N-S braced-frame direction in response to the Northridge-Newhall ground motion. For the ChiChi-002 ground motion, the E-W shaking dominates initially. Later, E-W and N-S motions become coupled and create a rotational twisting motion that is considerably larger than the motion due to the Northridge-Newhall motions.

Figure 11 Roof orbit response of the model to near-field, strong and far-field, weak earthquakes (see online version for colours)



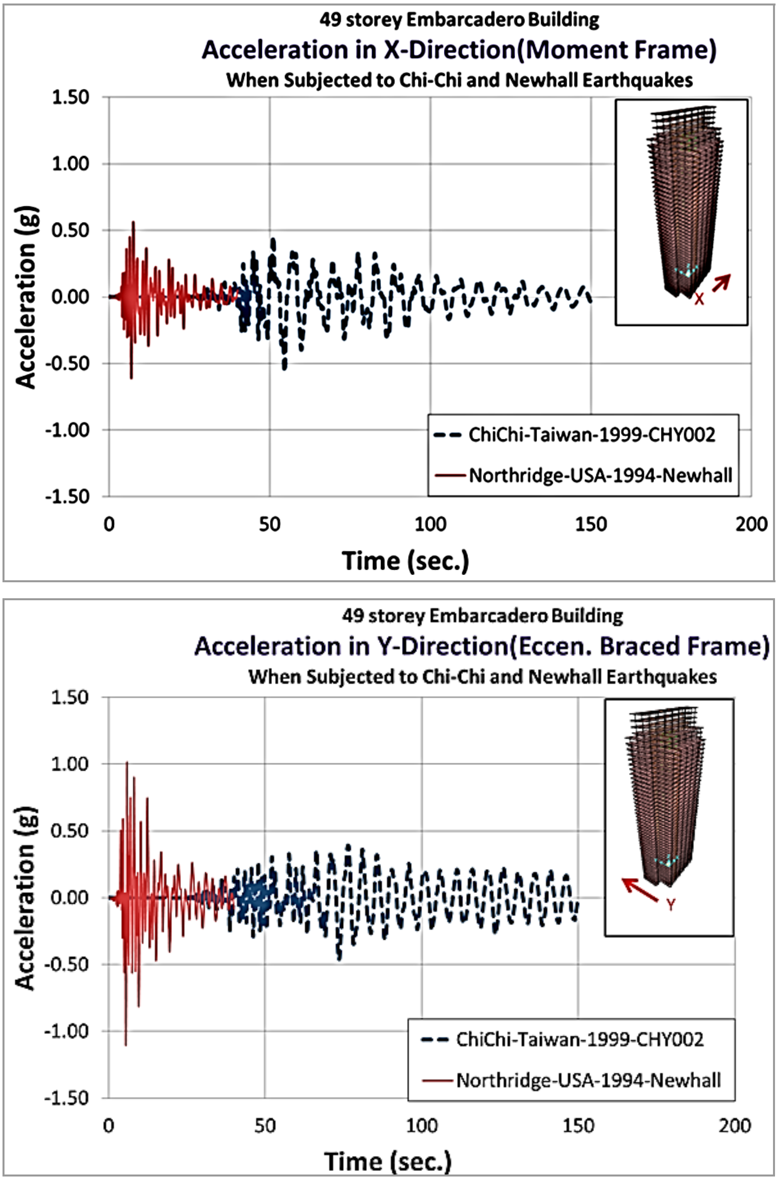
7.2 Acceleration response

Figure 12 shows the roof acceleration time histories for the studied structure for the two earthquakes. The peak accelerations for ChiChi-002 earthquake in both directions are close to 0.5 g (0.56 g in the moment frame and 0.46 g in the EBF direction). The ratio of the roof to base peak acceleration is 4 and 3.93, respectively. For Northridge-Newhall ground motion, peak accelerations are considerably higher than the ChiChi-002 earthquake. The peak roof acceleration reaches 0.61 g in moment frame direction, and 1.08 g in the EBF direction; the ratio of the roof to base peak accelerations are 1 and 1.8 for Northridge-Newhall and ChiChi-002 earthquakes, respectively. The ratios show that the variation of acceleration demand along the height of the structure will be greater for ChiChi-002 earthquake compared to the Northridge-Newhall earthquake.

For rigid or nearly rigid non-structural components, PGA can be used to estimate seismic demand. For flexible components, floor acceleration spectra are used to estimate the acceleration demands.

Figure 13 shows the 5%-damped roof acceleration spectra of the structure. The spectral ordinates for Northridge-Newhall earthquake can reach up to 2.3 g and 3.8 g in the moment frame direction and eccentrically braced-frame direction, respectively. For ChiChi-002, spectral ordinates as high as 1.8 g and 2.5 g were reached. Note: for the ChiChi-002, the first mode produces this maximum spectral acceleration for a component attached to the roof; in contrast, for Northridge-Newhall, the maximum ordinate corresponds to higher modes. Therefore, even when the building is only shaken in the first mode in response to far-field earthquakes, non-structural components may be damaged significantly.

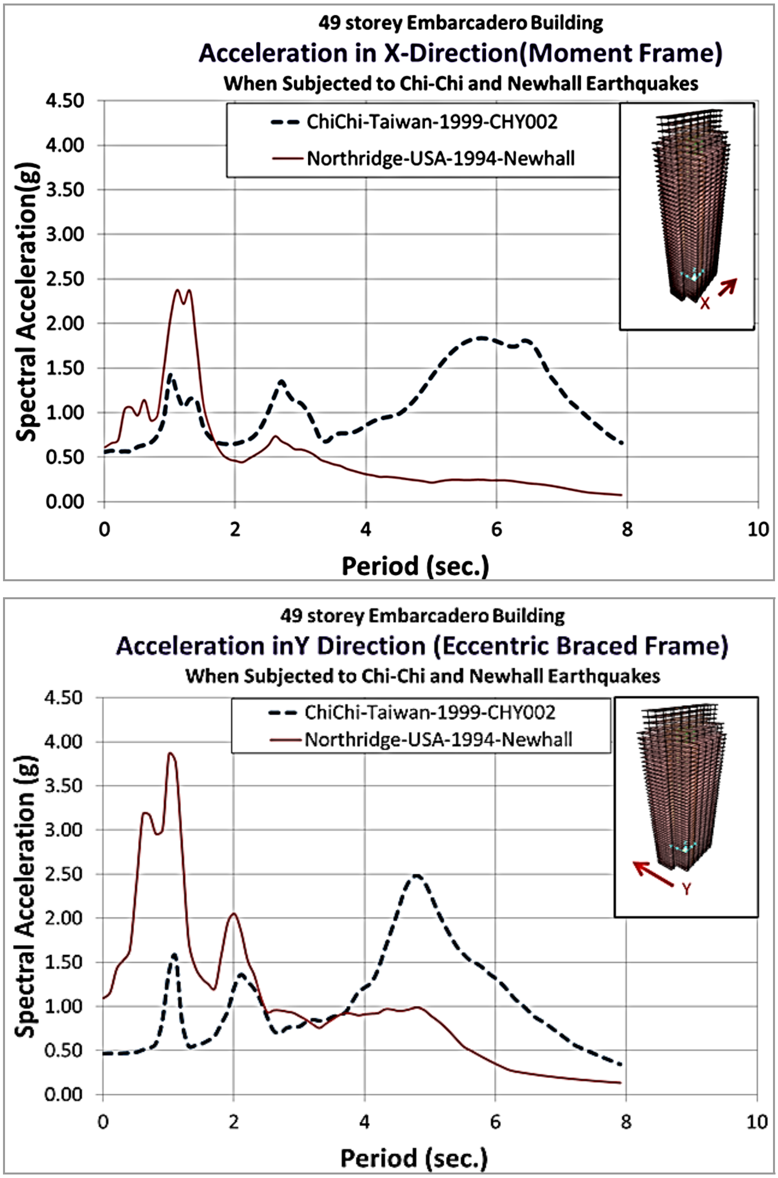
Figure 12 Acceleration time history of the model to near-field, strong and far-field, weak earthquakes (see online version for colours)



7.3 *Force response*

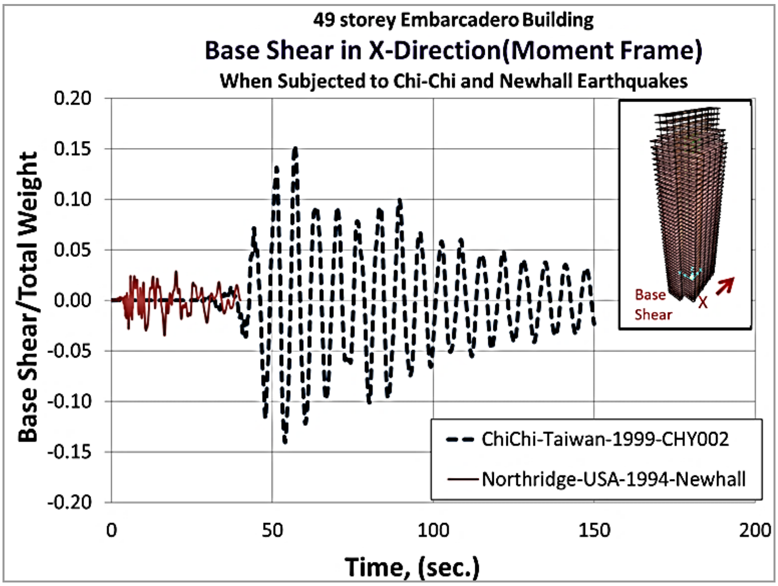
The main force response of interest is the base shear of the lateral load-resisting systems and axial force in the exterior columns. Base shear is normalised by the total weight of the structure, the weak, far-field ChiChi-002 created larger base shear forces in both moment frame and braced frame directions than the base shear generated by the strong, near-field Northridge-Newhall. Figure 14 shows that peak base shear for the ChiChi-002 in braced frame direction is 0.15 W (W = total weight of the structure), which is 50% more than the peak base shear of 0.098 W generated by the Northridge-Newhall.

Figure 13 Roof acceleration spectra of the model to near-field, strong and far-field, weak earthquakes (see online version for colours)



In the moment frame direction, the Northridge-Newhall produced a relatively low peak base shear of 0.03 W; however, ChiChi-002 ground motion generated a peak base shear of 0.15 W, which is five times that of the Northridge-Newhall-generated base shear. Apart from higher peak values, the ChiChi-002 also generated a higher number of large amplitude cycles of base shear in both directions. The higher number of large amplitude cycles can result in cyclic local and overall buckling, as well as low-cycle fatigue fracture of the members.

Figure 14 Base shear response of the model to near-field, strong and far-field, weak earthquakes (see online version for colours)



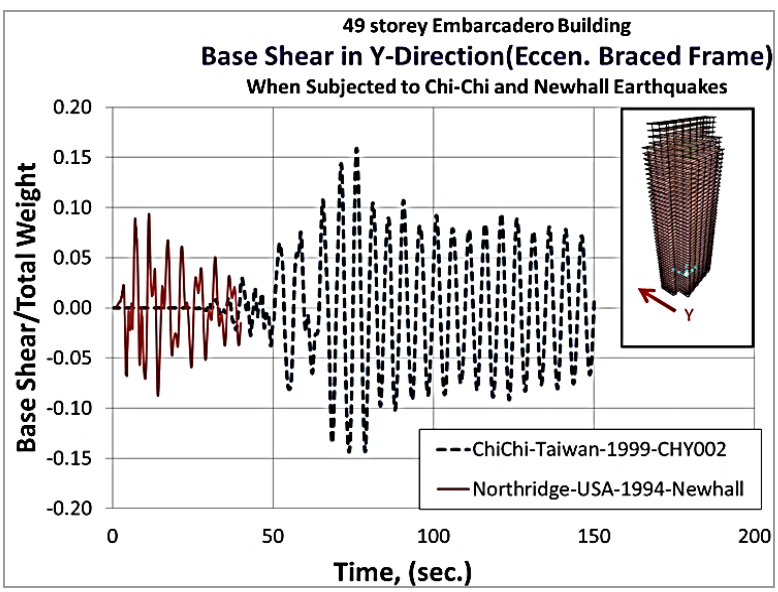
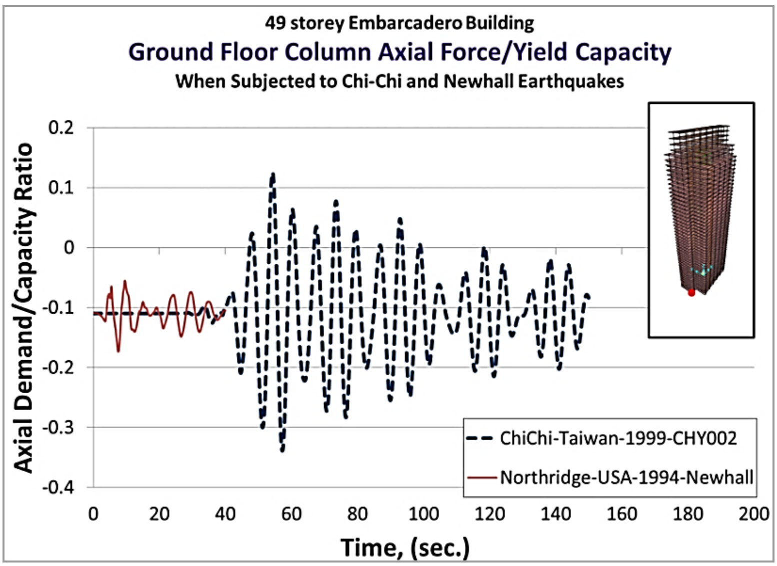


Figure 15 plots the axial force-time history of the external column in the N-S direction suggesting the possibility of the failure of this column and overturning failure of the structure. An overturning moment due to seismic forces combined with gravity forces produces the total axial force in the column. The plot in Figure 15 is normalised by the axial capacity of the column and shows that the exterior column remains in compression;

it was not subjected to a tensile force during the Northridge-Newhall earthquake. For the ChiChi-002, the peak tensile force on the column reached 12% of capacity. This tensile force is critical for the column-to-foundation connection as well as for pile-to-foundation connections. If the structure is sited on soft soil, there is the possibility of uplift of the foundation/pile system. Figure 16 shows the M-P relation for this column, along with the M-P curve in the AISC (2016) specifications. Note: the response to the Northridge-Newhall is well inside the M-P failure curve and within the elastic behaviour range; however, the response to the Chichi-002 exceeds the AISC (2016) M-P failure curve, indicating the possible yielding of the column.

Figure 15 Ground floor exterior column axial force response to near-field, strong and far-field, weak earthquakes (see online version for colours)



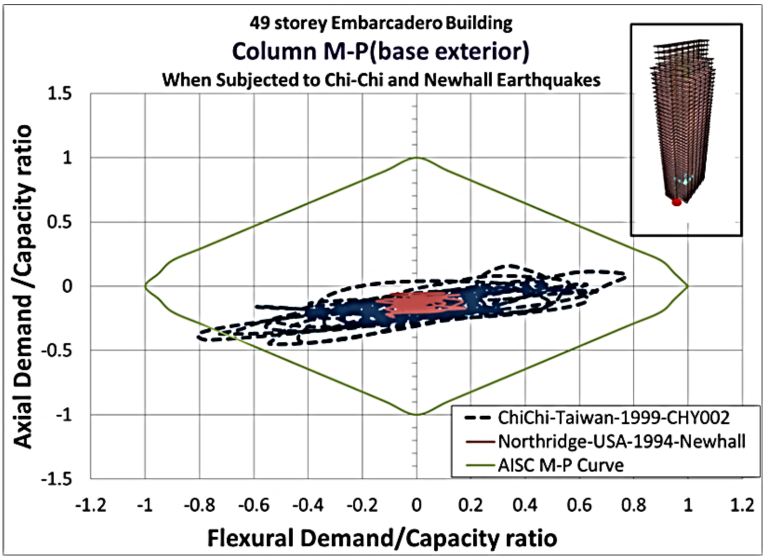
7.4 Response attenuation

Drift response, as well as the base shear response to the ChiChi-002 earthquake, attenuates at a lower rate than the response to the Northridge-Newhall earthquake. As shown in Figure 10, the drift response in the braced frame direction decreased 60% in five cycles (25.1 seconds) for Northridge-Newhall and 46.2% for ChiChi-002 in five cycles (24.8 seconds). Damping estimates from equation (1) corresponding to these values are 2.9% and 2%, respectively. The moment frame shows similar attenuation for both earthquakes. For the moment frame, the drift amplitude decreased 56% in four cycles (26.6 seconds) during Northridge-Newhall and 51% during ChiChi-002.

$$\xi = \frac{\ln\left(\frac{u_1}{u_{j+1}}\right)}{2 * j * \pi} \quad u_j = \textit{displacement at } j \textit{ cycle peak} \tag{1}$$

Attenuation rates between these two earthquakes differ more when considering the base shear response. For the Northridge-Newhall, the response decreases to 70% in five cycles, and for Chichi-002, it decreases by only 43% in five cycles. This slow attenuation for ChiChi-002 means that the structure will be subjected to large forces for a longer duration, which increases the probability of damage due to cyclic local and overall buckling, and low-cycle fatigue fracture.

Figure 16 Response of the exterior column on the ground floor to near-field, strong and far-field, weak earthquakes (see online version for colours)



8 Future research and development needs

This study shows that although far-field, low-intensity (i.e., weak) earthquakes are considered non-threatening, it can generate an alarming response in tall, long-period structures. Based on the preliminary results of this study, it is obvious that additional research of far-field earthquakes in the design of tall structures is critical. Elastic analysis of more long-period structures – such as structures higher than 150 m or long-span bridges – should be carried out with many more far-field input motions. Moreover, nonlinear time history analysis is also needed to validate the results obtained in the elastic analysis. Such analyses of the realistic structural models may be time-consuming and costly. Therefore, it is recommended that the inelastic analysis of simplified models to be done first to identify the main parameters affecting the response of long-period structures to long-distance earthquakes. Apart from looking at global displacement and force response, a local response such as stress-strain response should also be evaluated. Also, the effects of far-field, long-period ground motions on connections of steel and composite structures need investigation.

Effect of the far-field earthquake on different structural systems should also be studied. Nowadays, tall-structures utilise a variety of structural systems as well as shapes, especially steel and composite structures (Ali and Moon, 2007). Comparative studies on the effect of far-field motions on these structures are needed to help the engineering community to understand their behaviour better. Moreover, super tall structures usually have innovative, intricately detailed façade systems (Muin et al., 2015), which may be vulnerable to the long-period motions. The effect of a far-field earthquake on such non-structural elements is of utmost importance since the economic loss due to damage to the non-structural elements can be comparable or even exceed the loss due to the structural damage, especially in the case of tall structures.

Structures located near zones of moderate to low seismicity are of special concern for long-period earthquakes. Research on tall structures sited 100 km to 300 km from active faults, or fracking sites will uncover immensely important information about the effects of far-field earthquakes, which should be an area of much future research given the rapid increase in the number of these types of structures in urban environments.

9 Conclusions

This study has developed a finite-element model of an existing tall building that closely simulated the actual behaviour of the structure. This is verified by comparing the CSMIP instrumentation data with the analysis result of the model. This model was subjected to two earthquakes: one representative of a near-field, strong earthquake and the other representative of a far-field, weak earthquake.

Elastic time history analysis results showed that:

1. The representative far-field, weak earthquake caused roof drift of 1.5% and 2% in the structure in the E-W and N-S directions, respectively. These drift values could be cause for concern for non-structural as well as structural elements.
2. Acceleration amplification in a far-field earthquake may damage non-structural, flexible components attached to the higher floors.
3. The base shear time history for the two earthquakes also illustrated that the base shear generated by the far-field, weak ground motion was considerably larger than the base shear generated by the much stronger, near-field ground motion.
4. The axial force in the exterior ground floor column of the braced frames due to far-field ground motions became a tension force that uplifted the column but remained in compression during the stronger near-field ground motion.
5. The combined effect of the axial force and bending moment during far-field weak ground motion resulted in yielding and plastification of the exterior ground floor column; the column remained essentially elastic during the strong, near-field ground motion.
6. Much more research is needed in this area to investigate the effects of long-distance (far-field) earthquakes on tall buildings, especially in terms of the behaviour of various structural systems, connections, local and overall buckling, and low-cycle fatigue, as well as the behaviour of the foundations and soil-structure interaction.

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