Structural Response to the Predicted Design Earthquakes in Perth Metropolitan Area (PMA)

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Abstract

With the rapid increase in its population and the associated construction of more high rise buildings and more dense residential structures, and increase in earthquake activities in recent decades just east of Perth in an area known as the South-West Seismic Zone, the seismic risk of PMA increases significantly. The latest seismic hazard analysis indicated that the code value may underestimate PGA in most of the PMA rock sites especially in the north-east area. The corresponding site response study also showed that the calculated mean spectral values for shallow sand and mud-dominated site exceed significantly those of the code spectrum at period lower than 1sec, implying that structures with natural periods lower than 1sec are expected to be at risk of earthquake damage at these sites. Hence, an updated assessment of the performance of structures in response to possible earthquakes is necessary. In this study, the structural responses of three typical Perth structures, a masonry house, a low-rise building (reinforced concrete frame) and a high-rise building (reinforced concrete frame with core wall), on various rock and soil sites to earthquake ground motions of different return periods are investigated using numerical dynamic analysis. Seismic damage assessment of these buildings is presented.

Keyword: structural response, design earthquake, damage assessment

1. INTRODUCTION

Located in southwest Western Australia (SWWA), at large distance from any tectonic region, Perth is considered to be a region of relatively low to moderate seismicity. The seismic risk of Perth Metropolitan Area (PMA) was considered low due to the infrequency of high magnitude earthquakes and the low population density. However, during the recent decades, the situation has dramatically changed, in line with rapid population growth in PMA and increasing earthquake activity just east of Perth in a zone that has been named the "Southwest Seismic Zone" (SWSZ). Seismic risk in Western Australia is obviously increasing and it has become important for engineers to evaluate the performance of structures in response to possible earthquakes.

The recent seismic hazard study of PMA presented by Liang et al. (2008a) showed that PGA on rock site in PMA ranges from 0.14g in the north-east through to 0.09g in the south-west region of PMA for a return period of 475 years. Comparing to the PGA of 0.09g with the same probability given in the Australian earthquake loading code, the code underestimates PGA in most of PMA. A site characterization study of 16 sites in PMA was performed using the spatial autocorrelation (SPAC) method and the clonal selection algorithm (CSA) technique (Liang *et al*. 2008b). The site response of the 16 sites also showed that the code underestimates the spectral acceleration of shallow soil site corresponding to the 475-year return period earthquake in the period range of 0.5 to 3.0 sec, and undervalues the spectral accelerations of most of the 16 sites corresponding to the 2475-year return period earthquake in the long period range. Hence, the assessment of the seismic vulnerability of building is necessary.

In this study, the responses of three typical Perth structures, namely a masonry house, a low-rise reinforced concrete frame structure, and a high-rise building of reinforced concrete frame with core wall on various soil sites subjected to predicted earthquake ground motions of different return periods are calculated. Numerical results are used to assess the seismic damage of these buildings. The seismic safety of building structures in PMA is evaluated according to the various design and safety criteria for nonductile building frames.

2. STRUCTURAL RESPONSE

To assess the seismic vulnerability of building structures, three typical buildings in PMA are selected to perform structural response analysis in this study. Three-component seismic bedrock acceleration time-histories corresponding to the 475-year return period earthquake and 2475-year return period earthquake are simulated. Using these simulated time histories as input, the surface ground motion time histories of the 16 sites investigated by Liang *et al*. (2008b) are calculated using SHAKE2000. These simulated surface ground motions are applied along the three principal axes of the structure to estimate nodal displacement and element force of the structure. The amplitude of the vertical component is assumed to be 2/3 of the horizontal component. Time history analyses are carried out using SAP2000 to determine the response of buildings. The damage level of the structure is determined by comparing the inter-storey drift against the seismic performance levels of structures defined by FEMA356 (2000).

Unreinforced masonry building (UMB)

A wide range of buildings, including residential houses, shops, schools, churches and hospitals, are constructed of unreiforced masonry in PMA and is the most common construction form for new residential structures. A typical one story residential house is chosen to be modelled as shown in Figure 1. It should be noted that this kind of structure does not require specific earthquake-resistant design as specified in the Australian Code (AS 1170.4-2007) because the structure height is less than 8.5m and the hazard factor (k_pZ) for PMA is less than 0.11. However, lives will be placed at risk if such structures fail to resist predicted earthquake forces as this type of structures occupy the highest proportion of existing buildings in PMA (up to 88%). Hence, the building performance under the predicted ground motions is investigated in this study. The house uses jarrah trusses for the roof framing and unreinforced masonry for the wall. The thickness of the unreinforced masonry wall (UMW) is 10cm. Frame elements are used to model all jarrah truss members. UMW is assumed to be uniform elements representing the combination of brick and mortar and is modelled as shell element with 4 nodes and 6 degrees of freedom at each node. The material properties of jarrah are derived from HB 2.2 (2003). The material properties of UMW are obtained from a study of homogenized dynamic masonry properties proposed by Wei and Hao (2008). Dead load is the self-weight of the structural component. No imposed load is considered in the analysis.

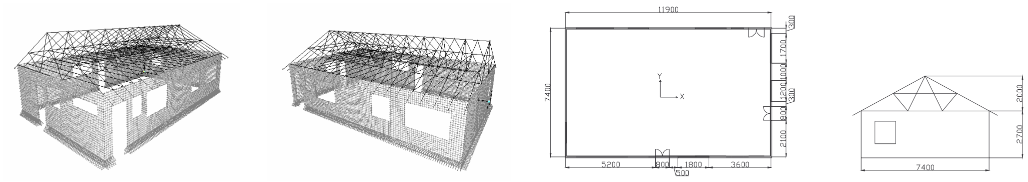

Figure 1. 3D model and building dimension of a typical one story residential house

The structure responses of the model on 16 sites of different conditions subjected to predicted earthquake ground motions of 475-year and 2475-year return periods are analysed using SAP2000's linear elastic dynamic analysis option. The natural period of the first three modes are calculated and listed in Table 1. The maximum drifts of masonry wall in two horizontal directions for different cases are obtained from the displacement response time-histories. The maximum drift of masonry wall is defined as the ratio of the maximum displacement on the top of the wall to the wall height. The numerical results show that, under the 475-year return period ground motion, the largest drift ratio of UMW is 0.1%, which is observed at the shallow soil site. Based on the seismic performance levels given in FEMA356 (2000), the structure of the residential house is safe for immediate occupancy after a 475-year return period ground shaking. Under the 2475-year return period ground motion, the maximum drift ratio of UMW observed at the shallow soil site is 0.14%, which is also within the level for immediate occupancy, implying the structure is unlikely to be damaged.

RC structure with masonry infill wall

To evaluate the seismic performance of RC buildings, an example RC structure is chosen to be modelled. The plan view of the building as well as the elevation of the building is illustrated in Figure 2.

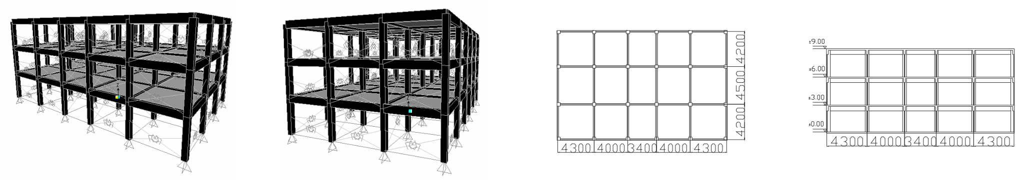

Figure 2. 3D model of the RC building

The column dimension is 0.4×0.4m whereas beam is 0.2×0.6m. The concrete material is C30 with uniaxial compressive strength of 30N/mm^2. The specified yield strength of steel is taken as 250MPa. Significant effect of masonry infill wall on the stiffness and strength of frame buildings has been observe in the literatures (i.e. Lee and Woo 2002, Lu 2002). Mehrabi et al. (1996) also indicated that the ratio of stiffness of infilled frame to bare frame can be 50 and the ratio of lateral resistance strength of infilled frame to bare frame is more than 2 under the condition of weak frame and strong panel. This study therefore considers the contribution of infill masonry walls in resisting the seismic force. The masonry infill walls are modelled as the equivalent diagonal compression-only struts. The width of the equivalent strut is estimated using Eq (1) and (2) proposed by FEMA356 (2000) guidelines.

$$W = 0.175(\lambda_1 h_{col})^{-0.4}\sqrt{H^2 + L^2} \tag{1}$$

$$\lambda_1 = \left[\frac{E_{me} t_{\inf} \sin 2\theta}{4E_{fe} I_{col} h_{\inf}}\right]^{\frac{1}{4}} \tag{2}$$

in which W is the width of the equivalent diagonal compression strut in inch, H and L are the height and length of the frame in inch, E_{me} and E_{fe} are the elastic moduli of the column and of the infill panel in ksi, t is the thickness of the infill panel in inch, θ is the angle whose tangent is the infill height-to-length aspect ratio in radians, I_{col} is the moment of inertia of column in in^4 and h_{inf} is the height of infill panel in inch.

Static load applied to the structure consists of dead and imposed load. Dead load is the self-weight of the structural components. Imposed load on the floor area is 3kN/m^2. Loading on beams are 2.2kN/m and 4.4kN/m, which are applied to the beams that support hollow brick walls with 1.2m and 2.4m height.

The natural periods of the first three modes are calculated and listed in Table 1. The investigation focuses on calculating the inter-storey drifts of the reinforced concrete frame and masonry infill wall. The maximum inter-story drift of the 3-storey building to the surface ground motions with different site conditions are calculated and then used as a criterion to assess the structure performance. Numerical results show that the largest storey drift ratio (0.051%) is less than 1%, indicating that the concrete frame of the 3-storey building is unlikely to be damaged when subjected to the 475-year return period event. However, under the 2475-year return period earthquake, when the structure is on the shallow sand site, the largest storey drift ratio is between 0.1% to 0.5%, implying the infill masonry wall will suffer light damage, but it will still be operational.

High-rise building of RC frame with core wall

The high-rise building shown in Figure 3 is a 34-storey RC frame with core wall. It mainly serves as business office. As shown, the building is irregular in shape and the plan dimension is 40.05×20m with a total height of 136.26m from the base. The lateral force resisting structural system consists of RC moment resisting frames attached to the concrete core walls at the centre. Typical section dimension of the column is 900×900mm and the depth of the beams is 700mm. The typical thickness of walls is 300mm. The specific grade strength of concrete f_{cu} for beam is 30MPa and for column and core wall are 60MPa. Static load applied to the structure consists of dead and imposed load. Dead load is the self-weight of the structural components. Imposed load on the floor area is 3kN/m^2. The self-weight of all brick walls, both internal and external, are calculated and are applied to the beams.

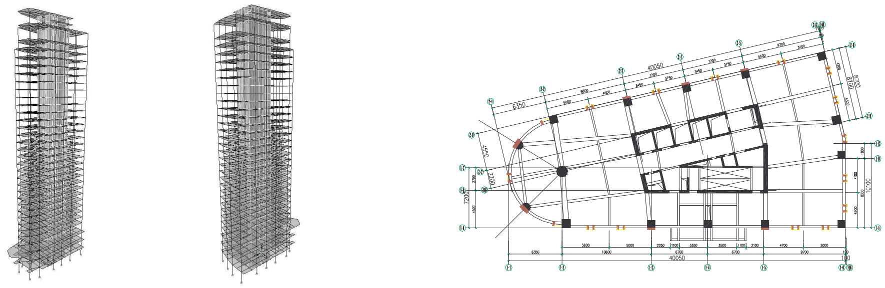

Figure 3. 3D model and building dimension of 34 story high-rise building

The natural periods of the first three modes are calculated and listed in Table 1. Numerical results show that the maximum inter-storey drift during the 475-year return period ground motion is 0.22% when the building is located on very soft site, indicating the building is safe since the largest storey drift is less than the critical value for immediate occupancy of concrete wall (0.5%) and concrete frame (1%). However, the largest storey drift of the building at most of the 16 sites considered in this study to the 2475-year return period ground motions is between 0.5% and 1%. These results indicate that the building will experience light to moderate damage at concrete walls. The estimated response is capable of causing damage to non-structural components which may lead to significant direct and indirect economic losses, but the building will satisfy a life-safe objective since the largest storey drift is less than 1%.

Table 1. Vibration periods

Mode	Period (sec)		
	UMB	3-story RC Building	34-story High-rise Building
First mode	0.18	0.22	3.70
Second mode	0.12	0.19	2.55
Third mode	0.09	0.17	1.97

3. CONCLUSION

The performance of three typical Perth structures, namely a masonry house, a low-rise reinforced concrete frame structure, and a high-rise building of RC frame with core wall at various soil sites subjected to predicted earthquake ground motions of different return periods are investigated. In most cases, the estimated drifts for structural components subjected to predicted earthquake ground motions of different return periods satisfied the drift demand for immediate occupancy level, indicating that the structural responses are mainly elastic.

The study revealed that one-storey UMW building is unlikely to be damaged when subjected to the 475-year return period ground motion and the 2475-return period ground motion. The three-storey RC frame with masonry infill wall is safe under the 475-year return period ground motion. However, the infill masonry wall will suffer light damage during the 2475-year return period ground shaking, but the building will still be operational. The 34-storey RC frame with core wall will not suffer any damage to the 475-year return period ground motion. The building will experience light to moderate damage under the 2475-year return period ground motion, but the damage is not life threatening.

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