Performance of Power Transmission Tower in PMA under Simulated Earthquake Ground Motion

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ABSTRACT :

Transmission towers play an important role in the operation of a reliable electrical power system that is considered as a lifeline system. The performance requirement of fully operational under damage-limitation earthquake is assigned to lifeline system in many current seismic codes to provide protection in the immediate post-earthquake period. Many studies and post earthquake investigations have revealed that the material and geometric non-linearity have a major effect on the ultimate strength of towers and the tower collapse is due to either spread of plasticity or premature buckling. Hence, transmission towers designed by equivalent static analysis method should be examined to check their performance under dynamic loading conditions. In this paper, a study of the performance of transmission tower under damage-limitation earthquake and rare earthquake derived from the seismic hazard analysis of Perth Metropolitan Area (PMA) is carried out. The results are compared with code provisions and recommendations for the design of transmission towers.

KEYWORDS: performance of power transmission tower, earthquake ground motion

1. INTRODUCTION

Transmission towers play an important role in the operation of a reliable electrical power system that is considered as a lifeline system. Transmission lines in PMA are mainly designed for wind loads in the transverse direction since steel lattice towers are deemed to be less sensitive to earthquake loads than most other types of structures (AS3995-1994). However, seismic analysis of transmission towers is important as the response of transmission towers subjected to earthquake may exceed their response to wind loads. Several recent cases of damage to transmission towers during earthquakes have been reported, e.g. two transmission towers collapsed due to large ground motion during the 1994 Northridge earthquake, and a significant number of electric power transmission towers suffered serious structural problems during the 1999 Chi-Chi earthquake. Damage to electric power transmission towers during the 1999 Chi-Chi earthquake led to blackouts in the central and northern regions of Taiwan after the earthquake. Wire and wireless telephone communication were also interrupted and not restored until 36 hours after the earthquake (Loh and Tsay, 2001). It is unrealistic to assume that the transmission towers satisfy the performance requirement under damage-limitation earthquake and rare earthquake without adequate analysis. The performance requirement of fully operational under damage-limitation earthquake is assigned to lifeline system in many seismic codes to provide protection in the immediate post-earthquake period. It is also expected that after rare earthquake, lifeline systems can be successfully repaired and reinstated to full service in a short time. Therefore, reliability and safety of the transmission towers are essential to minimise the risk of disruption to power supply or communication that may result from tower failure subjected to the damage-limitation earthquake or rare earthquake.

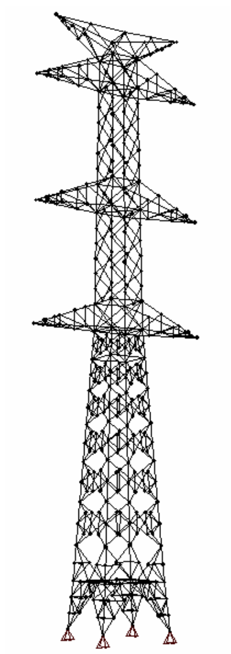
Based on an updated attenuation model of PGA and ground motion spectral accelerations developed by a combined stochastic and Green's function simulation method for Southwest Western Australia (SWWA) (Liang *et al.*, 2008b), a more reliable seismic hazard study for Perth Metropolitan Area (PMA) was carried out (Liang *et al.* 2008a). The study indicated that the current code value slightly underestimates PGA in the northeast PMA and the code spectrum for rock site might underestimate the spectral accelerations in PMA at periods below 1sec. The dynamic site response analyses of three typical site classes in PMA also showed that the calculated spectral values for shallow sand site and mud-dominated site exceeded significantly those of the code spectrum at periods lower than 1sec. Structures with natural periods lower than 1sec and designed by the current code might be at risk of earthquake damage at these sites.

In this study, responses of two typical transmission towers designed in accordance to the current seismic code, which are located around PMA as shown in Figure 1, are analysed using the estimated seismic ground motions for PMA. A study of the performance of transmission towers under damage-limitation earthquake and rare earthquake presented in Liang *et al.* (2008a) is carried out to investigate the reliability and safety of the transmission towers. The results are compared with code provision and recommendations for the design of transmission towers.

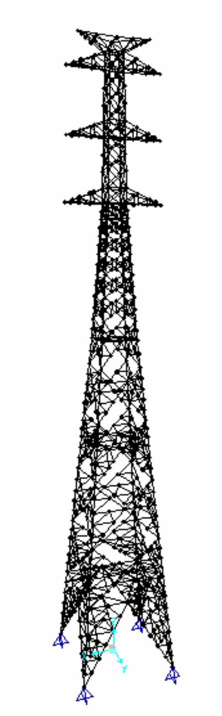
Figure 1. Transmission towers located in the northeast of PMA

2. TRANSMISSIONTOWER DESIGNED IN ACCORDANCE TO THE CURRENT CODE

Two 275 kV double circuit transmission towers with different height are designed in accordance to the current code. The computer program SAP2000 is used in this study to model the towers as shown in Figure 2.



(a)



(b)

Figure 2. Tower models (a. Tower1, b. Tower2)

Tower1 has a square base of 4.625m×4.625m and a height of 35.2m. The transmission line span supported by this tower is 330m. It is modelled using 597 truss elements with 400 nodal points. Tower2 is 71.2m height with a square base of 10.925m×10.925m. It is modelled by 1117 truss elements with 730 nodes. The base of the tower is assumed fixed. Dead load includes self-weight of the tower and vertical load corresponding to a normal condition. The vertical load corresponding to a normal condition for each conductor point of a 275kV transmission tower is given in Table 1.

Table 1.The vertical load corresponding to a normal condition for a 275kV transmission tower at each conductor point

Load Item	Weight (kg)
Weight of one span (kg) × 1.25	1.5×330×1.25=618.75
Weight of insulator string (kg)	120
Total weight (kg)	738.75

The wind action is calculated based on the current code (AS/NZS1170.2:2002 and AS3995-1994) as follows:

$$V_{sit,\beta} = V_R M_d (M_{z,cat} M_s M_t) \tag{2.1}$$

in which

$V_{sit,\beta}$= site wind speeds;

V_R= regional gust wind speed, in metres per second, for annual probability of exceedance of 1/R;

M_d= wind directional multipliers;

$M_{z,cat}$=gust wind speed multiplier for a terrain category at height z;

M_s= shielding multiplier;

M_t= topographic multiplier.

In this study, the design wind speeds ($V_{des,\theta}$) is taken as the maximum site wind speed ($V_{sit,\beta}$). V_R for a 2475-year return period around PMA is about 48m/s. A value of 1.0 is assigned to M_d when the orientation of the structure is not known. $M_{z,cat}$ is derived from Table 3.1(A) in AS/NZS1170.2:2002. M_s is 1.0 since the effect of shielding is ignored. M_t is 1.0. These values correspond to the worst scenarios of wind load acting on the tower.

For lattice tower, the wind force (F) can be determined as follows:

$$F = (0.5\rho_{air})[V_{des,\theta}]^2 C_{fig} C_{dyn} A_{ref} \tag{2.2}$$

in which,

ρ_{air}= density of air, which shall be taken as 1.2kg/m3;

C_{fig}= aerodynamic shape factor;

C_{dyn}= dynamic response factor;

A_{ref}= reference area of tower section.

The solidity ratio of the structure for the lattice tower is less than 0.1. Based on Table E6(A) in AS/NZS1170.2:2002, C_{fig} is 3.5. C_{dyn} is assumed to be 1.0. Based on Eqn. 2.1, Eqn. 2.2 and the tower dimension, wind pressure is calculated and shown in Figure 3.

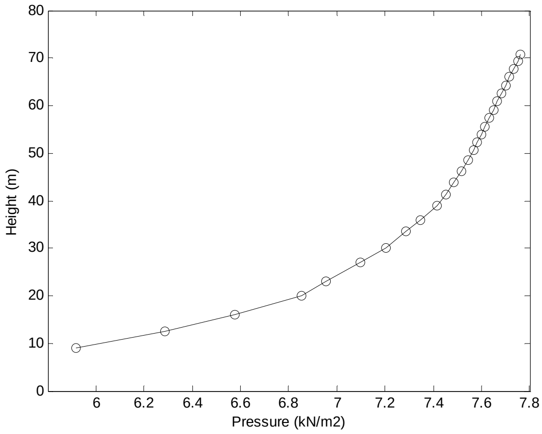


Figure 3. Wind pressure

Three load combinations are defined based on item 1.6.5 in AS3995-1994 and a value of 1.0 for the importance factor γ_w is adopted. A max/min Envelope of the defined analysis cases is evaluated. As Tower1 and Tower2 are less than 100m high and do not have significant mass concentrations, according to guidance for earthquake design (Appendix C) in AS3995-1994, Tower1 and Tower2 need not be designed for earthquake. In other words, seismic load is not considered in the design load. The section design is performed iteratively until the designed sections are the same as the analysis sections. The displacement for each joint is also derived. Results from the analysis indicated that the maximum displacement for Tower1 and Tower2 are 261mm and 681mm, respectively. The natural frequencies of the first 3 modes of the transmission towers are calculated and listed in Table 2.

Table 2. Model periods and frequencies

Mode	Period (sec)	
Mode	Tower1	Tower2
First mode	0.33	0.81
Second mode	0.32	0.53
Third mode	0.23	0.53

3. SEISMIC RESPONSE OF TRANSMISSION TOWER

The proposed design response spectra for the 475-year return period and 2475-year return period earthquakes were derived from probabilistic seismic hazard Analysis (PSHA) (Liang *et al.* 2008a). In this study, three-component seismic ground acceleration time-histories corresponding to the 475-year return period and 2475-year return period for four typical sites around PMA are simulated. Four typical sites, namely rock site, shallow sand site, deep sand site and mud-dominated site, are defined in (McPherson and Jones 2006). Due to the length restrictions, only one horizontal component seismic acceleration time-histories corresponding to the 475-year return period and 2475-year return period for four typical sites are shown in Figure 4 and Figure 5. These simulated surface ground motions are applied along the three principal axes of the structure to estimate nodal displacement and element force of the tower. The amplitude of the vertical component is 2/3 of the horizontal component. A comparison of the nodal displacement and element force of the towers subjected to earthquake ground motion and the defined load combinations in design code will be carried out.

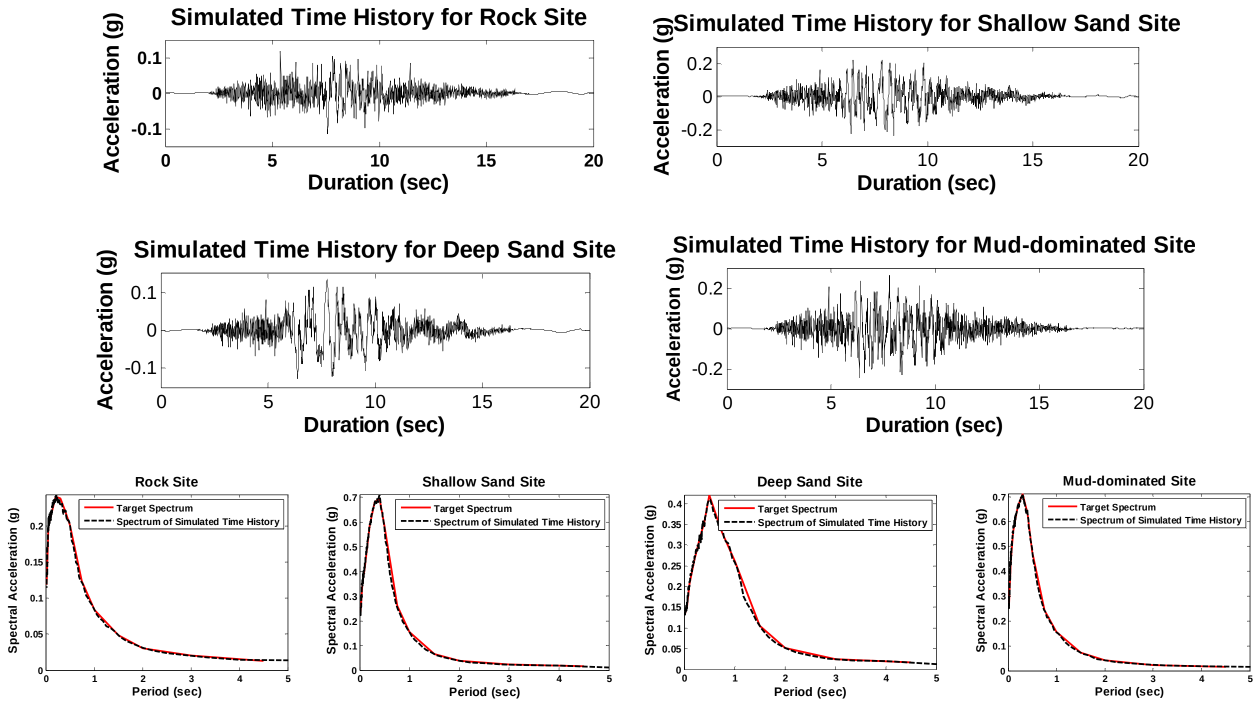


Figure 4. Typical simulated time histories and response spectra of the 475-year return period earthquake at the four sites

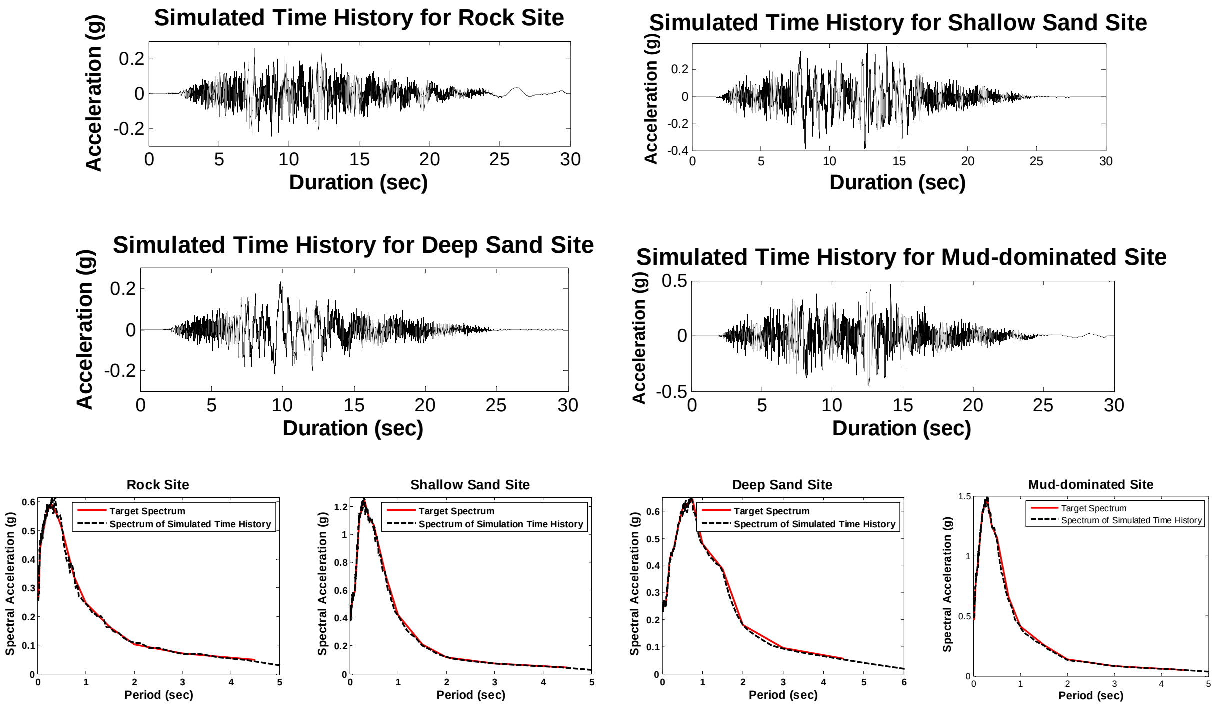


Figure 5. Typical simulated time histories and response spectra of the 2475-year return period earthquake at the four sites

3.1 Tower1

To gain a better insight on the performance of the tower subjected to seismic loading, the element forces for each member and nodal displacement are calculated. The member is defined as seismic loading dominated element when its force is more than that under design loading condition. The displacement ratio is the ratio of the displacement at the top of the tower subjected to earthquake motion to that under design loading condition. The number of seismic loading dominated elements, displacements at the top of the tower and the displacement ratio are listed in Table 3. As shown, during the 475-year return period earthquake, Tower1 at rock site and deep sand site has no seismic loading dominated element, indicating that the designed Tower1 is governed by the wind loads. 2 and 3 seismic loading dominated elements are recorded when Tower1 is located at shallow sand site and mud-dominated site, respectively. Among the value obtained from those four sites, the value for displacement at the top of the tower and displacement ratio observed at mud-dominated site are the highest and are found to be 42.87mm and 16.39%, respectively. It is noted that the displacement ratio is the ratio between the displacement to the seismic loading and to the wind loading. For the 2475-year return period ground motion, the number of seismic loading dominated elements and displacement response increase. The largest response still occurs when the tower is located at mud-dominated site. However, as shown in Figure 6, the responses of the tower to earthquake ground motion are small as compared to those to the design load, indicating wind load governs the design of the tower.

Table 3. Comparison of the Tower1 performance subjected to ground motion and design loading

Ground motion level	Site condition	No. of seismic loading dominated element	Displacement at the top of the tower		Displacement ratio (%)	
			UX (mm)	UY(mm)	UX	UY
475-year return period	Rock	0	13.41	13.08	5.13	4.96
	Shallow Sand	2	39.51	42.49	15.11	16.10
	Deep Sand	0	19.52	25.07	7.46	9.50
	Mud-dominated	3	42.87	40.77	16.39	15.45
2475-year return period	Rock	1	35.85	39.76	13.71	15.06
	Shallow Sand	4	63.19	71.02	24.16	26.91
	Deep Sand	0	26.09	27.99	9.98	10.61
	Mud-dominated	6	74.48	69.97	28.48	26.51

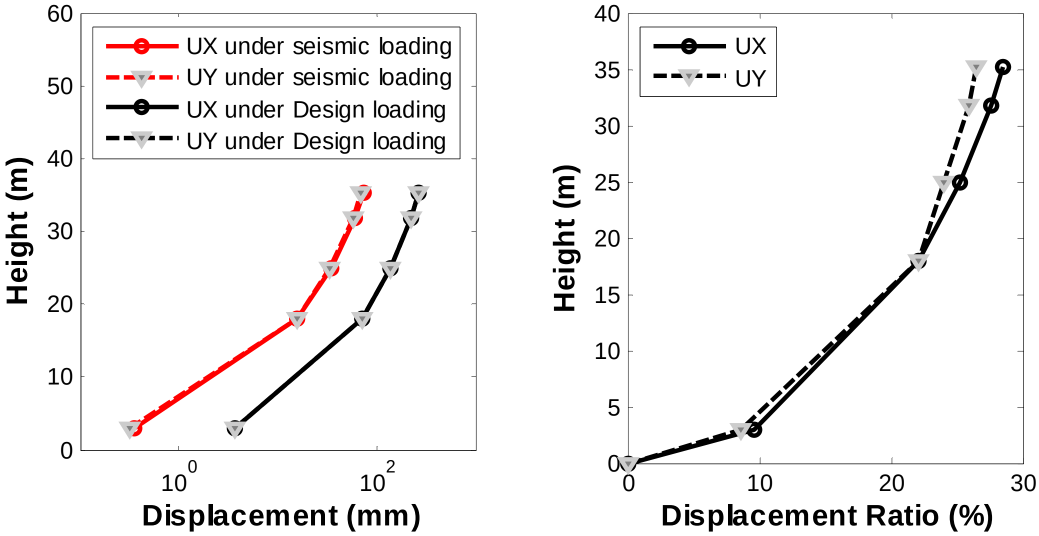


Figure 6. Displacement and displacement ratio of tower1 under the 2475-year return period ground motion at mud-dominated site

3.2 Tower2

As can be seen in Table 4, under the 475-year return period ground motion, Tower2 has 4 seismic loading dominated elements at rock site and deep sand site and 7 and 8 seismic loading dominated elements at shallow sand site and mud-dominated site, respectively. Among the four sites, the displacement at the top of the tower and the displacement ratio at shallow sand site are the highest and are calculated to be 97.37mm and 14.29%, respectively. Under the 2475-year return period ground motion, among 1117 elements, there are 9 seismic loading dominated elements at rock site and deep sand site, 28 and 32 seismic loading dominated elements at shallow sand site and mud-dominated site, respectively. The maximum displacement (230.99mm) and the displacement ratio (33.84%) occur when the tower is located at shallow sand site. As shown in Figure 7, the displacement along the height of the tower subjected to seismic loading is far smaller than that due to design loading condition. As a result, displacement ratio is relatively small, indicating that the designed tower according to wind load is unlikely to be damaged by earthquake loads

Table 4. Comparison between Tower2 performance subjected to ground motion and design loading

Ground motion level	Site condition	No. of seismic loading dominated element	Displacement at the top of the tower		Displacement ratio (%)	
			UX (mm)	UY(mm)	UX	UY
475-year return period	Rock	4	33.89	36.15	4.97	5.30
	Shallow Sand	7	97.37	91.52	14.29	13.41
	Deep Sand	4	75.15	68.50	11.03	10.03
	Mud-dominated	8	77.11	74.69	11.32	10.94
2475-year return period	Rock	9	80.72	115.63	11.85	16.94
	Shallow Sand	28	221.55	230.99	32.51	33.84
	Deep Sand	9	98.99	119.79	14.53	17.55
	Mud-dominated	32	182.61	218.72	26.80	32.04

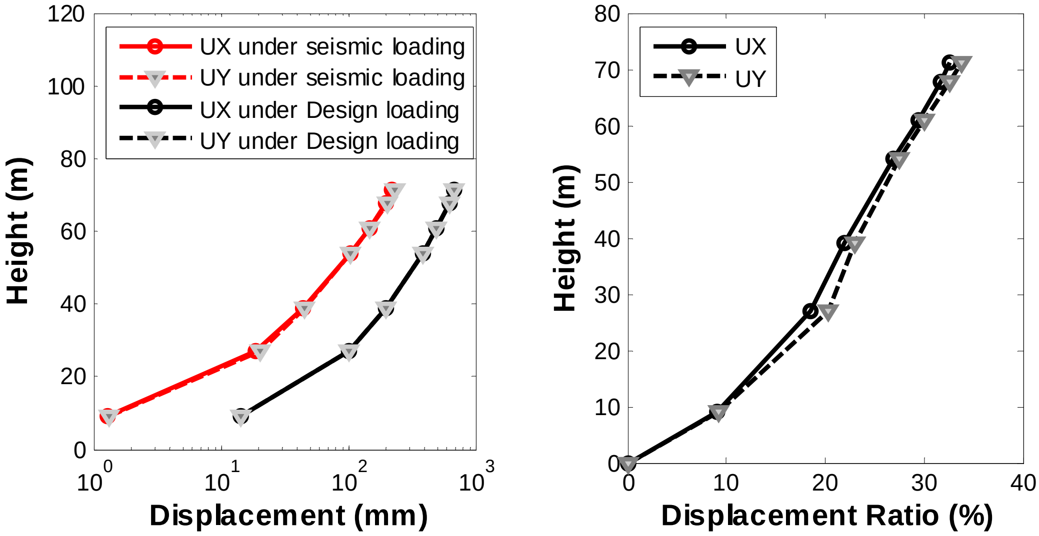


Figure 7. Displacement and displacement ratio of tower2 under the 2475-year return period ground motion at shallow sand site

4. CONCLUSION

The responses of transmission towers under damage-limitation earthquake and rare earthquake have been calculated in this study to investigate the reliability and safety of the transmission towers. The input acceleration time-histories corresponding to the 475-year return period and the 2475-year return period earthquake ground motion for four typical sites around PMA are simulated and used as input in the analysis. Two 275 kV double circuit transmission towers with 35.2m and 71.2m height are used in the study. The numerical results show that element force of most of members during damage-limitation earthquake and rare earthquake are less than that subjected to the design forces. The displacement along the height of tower subjected to seismic loading is far smaller than that due to the design loads, indicating wind load governs the design of the towers in PMA. However, there are still some seismic loading dominated elements at some sites. The investigation of the effect of seismic loading dominated elements on the reliability of transmission tower should be carried out in the future.

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