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Seismic design strategy of cable stayed bridges subjected to strong ground motions

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Abstract. In this paper, we present an alternative seismic design strategy for cable stayed bridges with concrete pylons when subjected to strong ground motions. The comparison of conventional seismic design using supplemental dampers (strategy *A*) and the new strategy using nonlinear seismic design of pylon columns (strategy *B*) is exemplified by one typical medium span cable stayed bridge subjected to strong ground motions from 1999 Taiwan Chi-Chi earthquake and 2008 China Wenchuan earthquake. We first conducted the optimization of damper parameters according to strategy *A* in response to the distinct features that strong ground motions contain. And then we adopted strategy *B* to carry out seismic analysis by introducing the elastic-plastic elements that allowing plasticity development in the pylon columns. The numerical results show that via strategy *A*, the earthquake induced structural responses can be kept in the desired range provided with the proper damping parameters, however, the extra cost of unusual dampers will be inevitable. For strategy *B*, the pylon columns may not remain elastic and certain plasticity developed, but the seismic responses of the foundation will be greatly decreased, meanwhile, the displacement at the top of pylon seems to be not affected much by the yielding of pylon columns, which indicates the pylon nonlinear design can be an alternative design strategy when strong ground motions have to be considered for the bridge.

Keywords: seismic design; cable stayed bridge; strong ground motion; concrete pylon; nonlinear seismic behavior

1. Introduction

Strong ground motions recorded in recent major earthquakes, such as 1999 Taiwan Chi-Chi earthquake and 2008 China Wenchuan earthquake, usually contain special features, including great PGA, velocity pulse, large ground residual displacement and etc., which are quite different from ordinary ground motions in common sense, and hence resulted more severe damages to many civil structures (Nakashima *et al*. 2000, Liao *et al*. 2004, Chopra *et al*. 2001). The related study is therefore a very important topic for both the seismological and civil engineering communities

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(Makris *et al*. 2004, Park *et al*. 2004, Alavi *et al*. 2004).

On the other hand, cable-stayed bridge has become the most widely built bridge type with the rapid development in recent decades. However, as one of the important lifelines, its seismic performance under the strong earthquakes is still unknown, and there is no specific seismic design strategy in domestic and foreign codes (GSDHB 2008, AASHTO 2007). Traditionally, bridge pylons are designed to be elastic even under occasionally happened earthquakes, however, Chang *et al.* (2004) reported that the Chilu cable-stayed bridge sustained significant damage in the 1999 Chi-Chi strong earthquake. In the bottom region of the pylon, spalling and splitting of the concrete around the core was evident and plastic hinge was exposed, which demonstrated the fact that strong ground motion caused much sever damages even to the bridge that has been carefully designed. In addition, some important cable-stayed bridges (such as Rion-Antirion in Greece and new San Francisco-Oakland Bay Brige in USA) are designed with considering the ductile behavior in the towers to reduce responses under unexpected large earthquakes (Combault *et al.* 2005, Okamoto *et al.* 2011), and hence brought the attention to the nonlinear seismic behavior of these bridges (Camara and Astiz 2012).

Practically, seismic energy absorption such as viscous dampers will be used to mitigate the seismic induced responses (Ye *et al*. 2004), and they have been used quite widely in recent years (Miyamoto *et al*. 2010, Ribakov 2011, Cheng *et al*. 2010, Vader *et al.* 2007, Hwang *et al*. 2005) to mitigate the earthquake induced structural responses even subjected to strong earthquakes in an aim to achieve the desired design purposes, such as keep the key structure components in elastic and limit the displacement at the key position, but needless to say, extra cost of unusual dampers will be inevitable. Therefore, effective and economic seismic design strategy should be applied to ensure the safety of these important lifelines structures under the strong earthquake shaking.

In this paper, under the subject of developing effective seismic design strategy of cable stayed bridges with concrete pylons subjected to strong ground motions, two seismic design strategies of cable stayed bridges are investigated and compared based on the longitudinal seismic responses of a typical medium span cable stayed bridge under the strong ground motions from 1999 Taiwan Chi-Chi earthquake and 2008 China Wenchuan earthquake. Strategy A is the conventional seismic design using supplemental dampers based on pylon elastic design according to the practices from most ordinary ground motions. Strategy B is the pylon plastic design, i.e., by allowing the pylons develop certain plasticity during strong ground shaking provided that the bridge is still seismically safe after the shaking.

2. Analysis model

The analyzed bridge, with a 90 m high H-shape reinforced concrete pylon, is a medium-span semi -floating system of cable stayed bridge with 298m center span. A three-dimensional dynamic finite element model was developed to simulate the bridge using the SAP2000 computer program (SAP2000 1999). The computer model is shown in Fig. 1. The beam, pylons and piers are modeled by frame elements, while truss elements are used to model the inclined cables. To reflect softening of the concrete columns after cracking, the effective stiffness of pylon column and piers is equal to one-half the gross stiffness. The geometric nonlinearity has little influence on the seismic response behavior, even under strong ground motion inputs, but the effect of the dead load must be considered (Ren Wenxin *et al*. 1999). Therefore, the geometric nonlinearity is not considered herein, but both linear and nonlinear analysis start from the deformed configuration and the stress

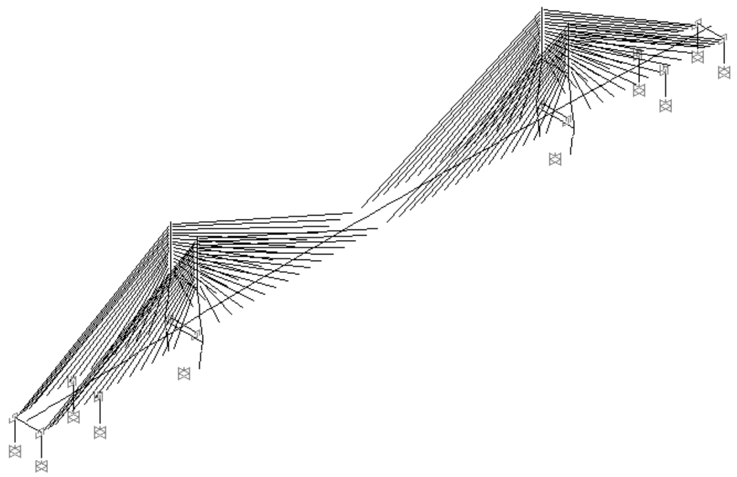


Fig. 1 Dynamic analysis model

Table 1 Information pertinent to the ground motions used in this paper

Earthquake record	Magnitude	PGA (g)	PGV (cm/s)	PGD (cm)
1999 TCU076 EW	7.6	0.343	69.2	108.0
2008 51MZQ NS	8.0	0.840	129.8	396.8

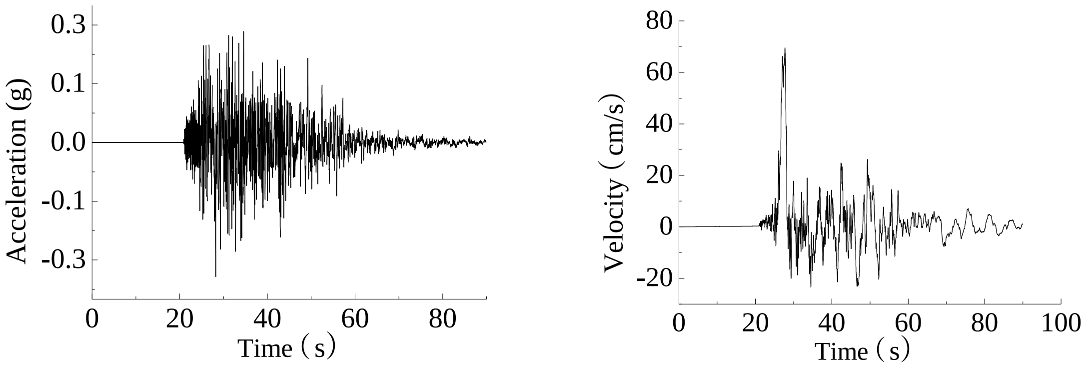
state due to dead load.
The bridge is assumed to stand on rigid foundation, the dynamic soil–structure interaction is hence neglected and the bottom of pile caps are fixed to the ground. As the start of a series of research on the seismic performance of cable stayed bridges subjected to strong ground motions, only longitudinal motion is considered herein.

3. Input ground motions

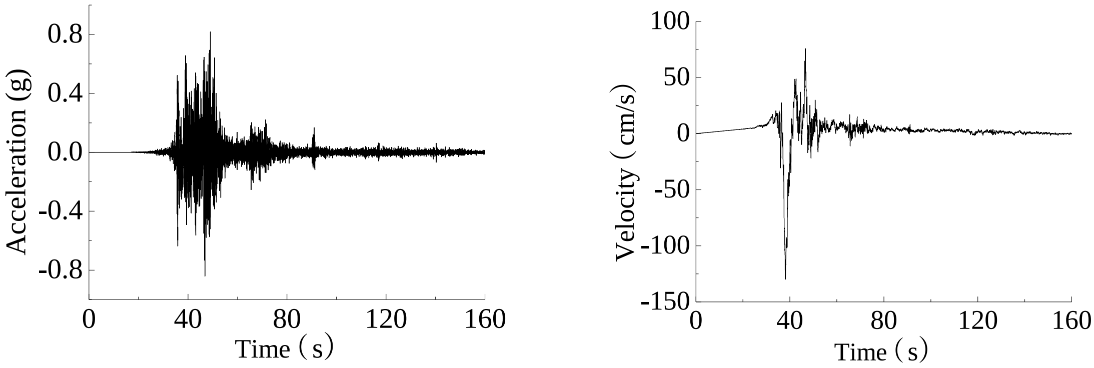
To evaluate the difference of strong ground motions on the damper parameters' design, two earthquake records are used in the analysis. One was recorded during the 1999 Chi-Chi earthquake in Taiwan, named as TCU076 record, and the other was recorded during the 2008 Wenchuan earthquake in China, named as 51MZQ record. The E-W component of TCU076 and the N-S component of 51MZQ are selected for the analysis. Information pertinent to the ground motions are listed in Table 1, the acceleration and velocity time histories are shown in Fig. 2, and the corresponding response spectra with damping ratio of 3% is shown in Fig. 3.

4. Modal and response spectrum analysis

The modal periods and modal shapes of the analyzed bridge were first calculated. The first mode is the floating vibration in the longitudinal direction, shown in Fig. 4, and the corresponding modal period is 6.1s. As the dominated mode in the longitudinal direction, the modal participating



(a) TCU076



(b) 51MZQ

Fig. 2 Time histories of selected ground motions

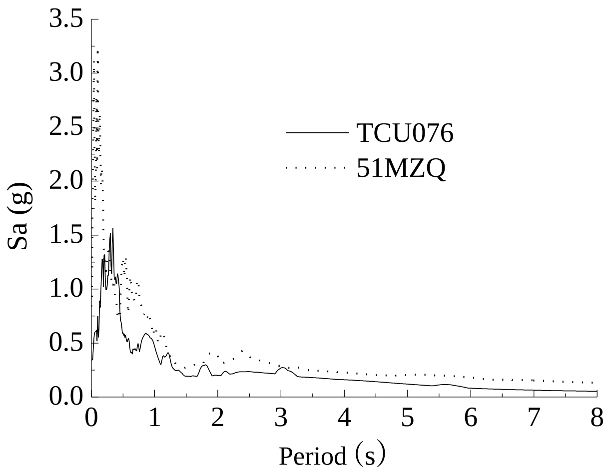


Fig. 3 Response spectra of selected ground motions

mass ratio of this mode reaches 76%. Based on the results of the modal analysis, the longitudinal response spectrum analysis was carried out by input the response spectra described in Fig. 3. The modal combination method is CQC. The resulted responses of focused locations are listed in Table 2.

As indicated in Table 2, the displacement and the bending moment at the key positions are both considerable large, even though TCU076 record only has a PGA around 0.3g, not to mention 51MZQ record, which induced approximately twice as much as response than TCU076 record, therefore the effective seismic design strategy should be applied to the bridge to ensure the seismic safety.

5. Seismic design strategy

5.1 Strategy A-using supplemental dampers

In this study, four nonlinear viscous dampers expressed by Eq. (1) are selected to install between each pylon and the beam in the longitudinal direction of the analyzed bridge.

$$F=CV^{\alpha} \tag{1}$$

Where F= damping force; C= damping coefficient; α= damping exponent.

The allowable bending moment at the bottom of pylon is limited to the equivalent yield bending moment, calculated by Xtract computer program (Xtract 2002), and the allowable displacement at the end of beam is limited to 50cm by considering the application of normal expansion joints. Optimization of the damping parameters was then carried out under TCU076 record and 51MZQ record inputs, respectively. A time integration procedure, the Newmark β method, was used for the longitudinal direction analysis with the consideration of the friction effect of the slide bearing and the energy dissipation of the longitudinal viscous dampers.

Fig. 5 shows the results of damping coefficient studies in the case of damping exponent of 0.3. As the damping coefficient increases, the displacement at the end of beam, the bending moment at the bottom of pylon and the bending moment at the bottom of pylon pile caps decrease, while the

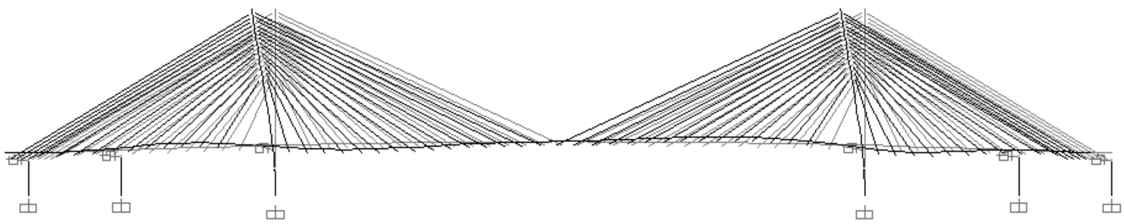
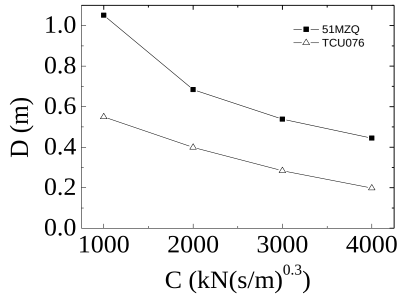


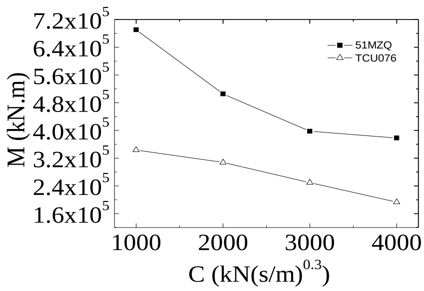
Fig. 4 The first mode shape

Table 2 Results of response spectrum analysis

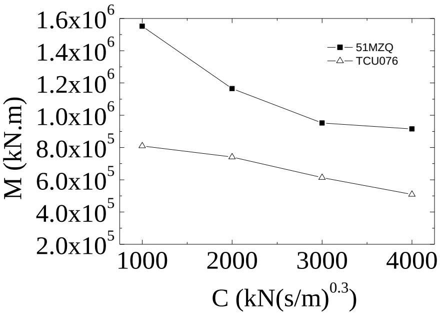
Record	Displacement at the end of beam (m)	Bending moment at the bottom of pylon (kN.m)	Bending moment at the bottom of pylon pile cap (kN.m)
TCU076	0.773	501996	1100806
51MZQ	1.701	1081525	2342253



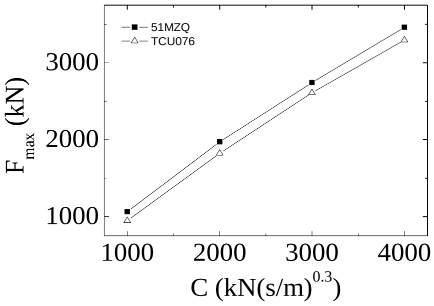
(a) Displacement at the end of beam



(b) Bending moment at the bottom of pylon

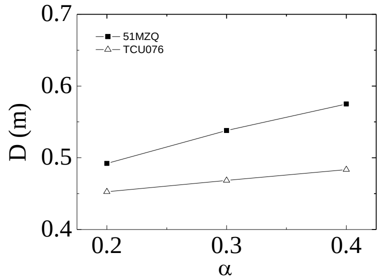


(c) Bending moment at the bottom of pylon pile cap

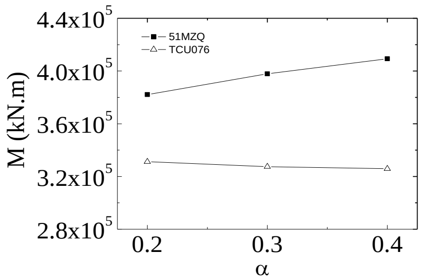


d) The maximum damping force of each damper

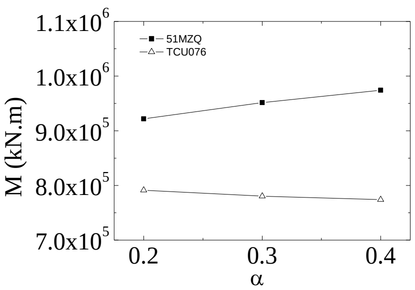
Fig. 5 Relationship curves between damping coefficient C and seismic response at the key location and the maximum damping force F_{max} of each damper under TCU076 and 51MZQ records



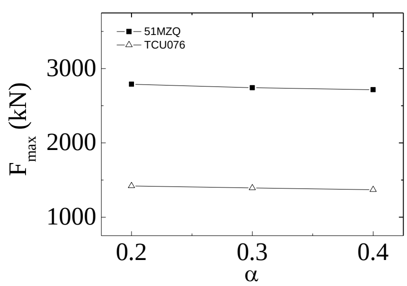
(a) Displacement at the end of beam



(b) Bending moment at the bottom of pylon



(c) Bending moment at the bottom of pylon pile cap



(d) The maximum damping force of each damper

Fig. 6 Relationship curves between damping exponent α and seismic response at the key location and the maximum damping force F_{max} of each damper under TCU076 and 51MZQ records maximum damping force of each damper increases. All the relationship curves are nearly linear. Fig. 6 shows the results of damping exponent studies in the case of damping coefficient of 1500 for TCU076 record and 3000 for 51MZQ. For the damping exponent, the effects on the seismic response at the key location and the maximum damping force of each damper are very small compared with the case of damping coefficient. From these figures, it can be observed that the relationships between structure seismic responses and damping parameters do not depend on the characteristics of earthquake records, and the damping coefficient is the key parameter for the damper design.

Table 3 presents the results of the optimizations of the damping parameters for this study. For both the two records, the earthquake induced structural responses are kept in the desired range by the seismic dissipation design with the proper parameters described in Table 3, i.e., the displacement at the end of beam is less than 50cm and the pylon remains elastic.

5.2 Strategy B-pylon plastic design

Pylon plastic design is to make use of plastic capacity of pylon on the premise of the seismically safety of the structure after the shaking. Since the deformation and yielding are concentrated at the specific locations of pylon, damage to other structural components may be reduced or shifted.

The nonlinear link elements are adopted to simulate plasticity development in the pylon. Fig. 7 presents the typical bilinear hysteresis curve of nonlinear link element. The equivalent yield bending moment *My*, ultimate bending moment *Mu*, equivalent yield curvature φ_y, and ultimate curvature φ_u are calculated by Xtract (2002). Furthermore, these models require an approximate

Table 3 Results of optimizations of the damping parameters

Record	*C* (kN(s/m*α*))	*α*	Fmax (kN)	Displacement at the end of beam (m)		Bending moment at the bottom of pylon (kN.m)		Reduction for the bending moment at the bottom of pylon pile cap (%)
				Demand	Limit	Demand	Limit	
TCU076	1500	0.3	1393	0.468	0.500	327506	590000	33.9
51MZQ	3000	0.2	2790	0.492	0.500	382183	590000	58.9

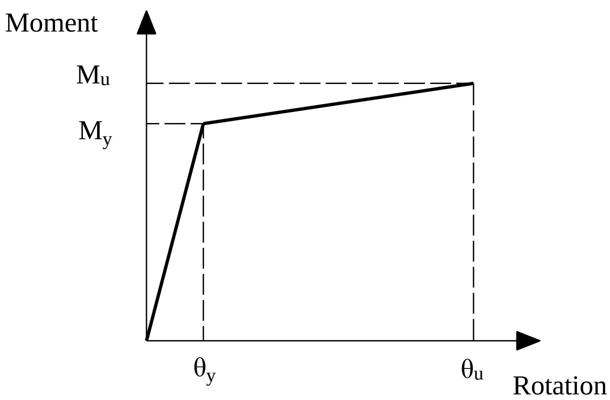


Fig. 7 Typical bilinear curve of nonlinear link element

plastic hinge length to convert plastic curvature to plastic rotation as defined in US Caltrans Seismic Design Criteria 2004.

The acceleration time history of TCU076 record and 51MZQ record, shown in Fig. 3, was adjusted based on the PGA as the intensity index with a increment of 0.2g, and a series of longitudinal nonlinear time history analysis with Newmark β integration method were carried out.

Fig. 8 and Fig. 9 show the development of plastic curvature distribution along the height of pylon column with the increment of PGA for TCU076 record and 51MZQ record, and also show the equivalent yield curvature and ultimate curvature as the comparison. In these figures, the bottom of pylon pile caps is defined as origin of height, and the symbol B and L represent the position of the bottom of pylon column and the lower strut, respectively. From Fig. 10, it can be observed that with the increment of PGA from 0.6 g to 1.0g for TCU076 record, the plastic curvatures above the bottom of pylon increased rapidly while slightly at other positions. In the case of PGA of 1.0 g, the plastic curvatures above the bottom of pylon is much larger than those at other positions. Fig. 11 shows the corresponding results for 51MZQ, the development trend of the plastic curvature is the same as that for TCU076, the plastic curvature above the bottom of pylon is also the largest. For both the two records, the maximum plastic curvature demand is far

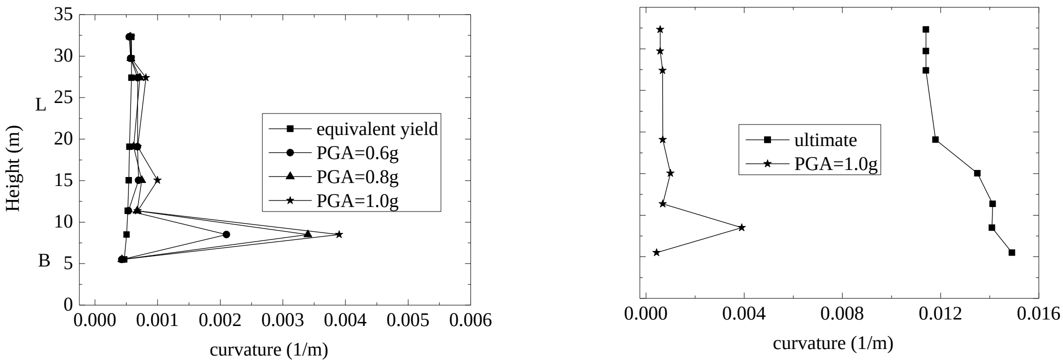


Fig. 8 The equivalent yield curvature, ultimate curvature and the curvature demand with the increment of PGA for TCU076 record along the height of pylon column

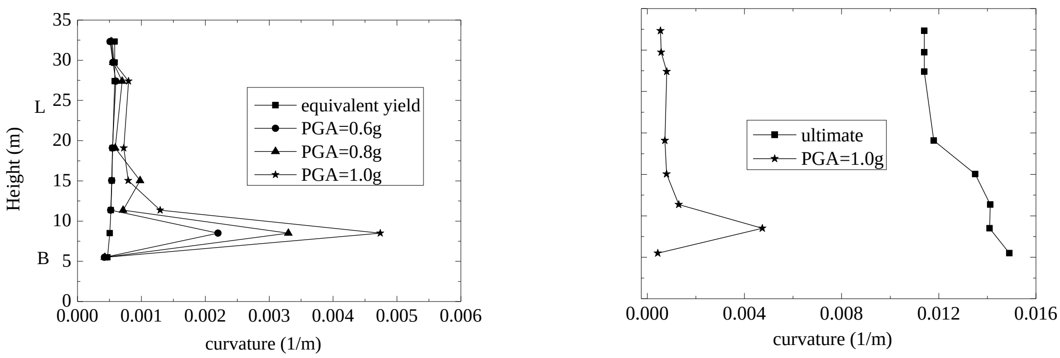


Fig. 9 The equivalent yield curvature, ultimate curvature and the curvature demand with the increment of PGA for 51MZQ record along the height of pylon column

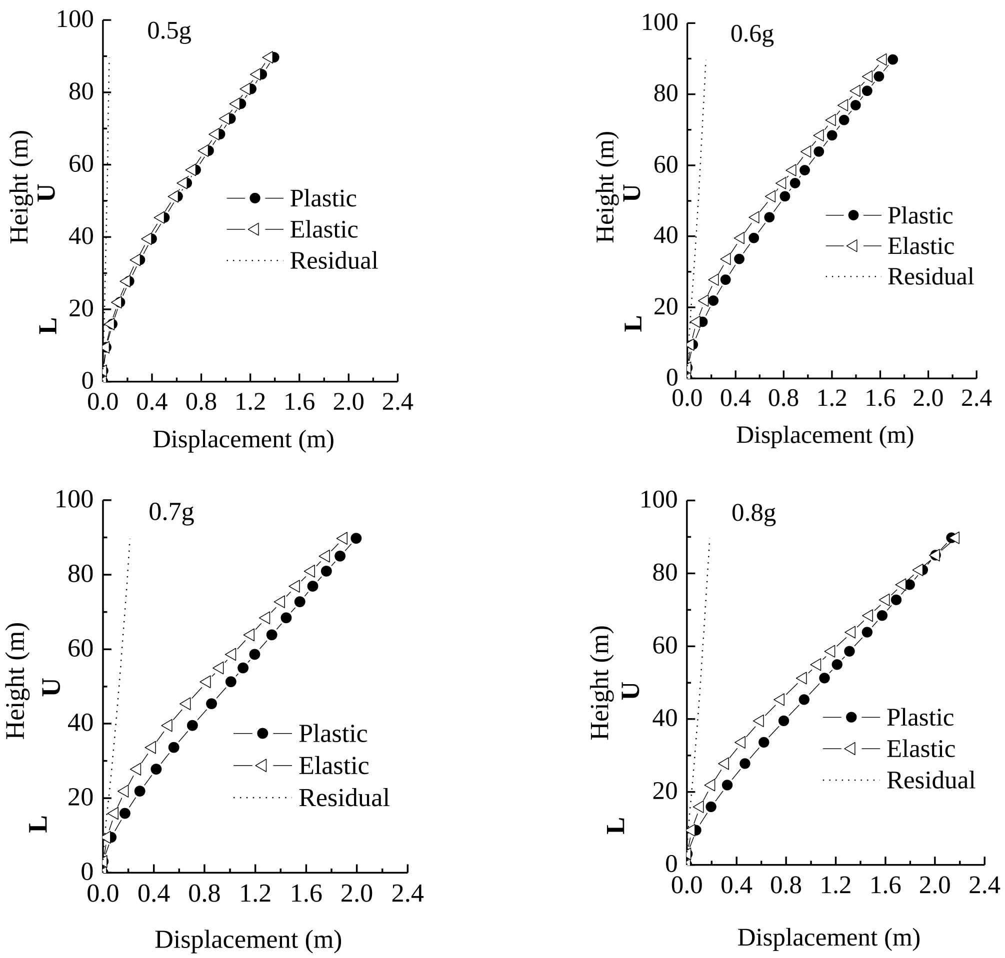


Fig. 10 Comparison between deformed shape of pylon columns with and without the consideration of the nonlinear behaviour of pylon

less than the ultimate curvature. Therefore, after the yielding, pylon column can reserve considerable plasticity capacity.

In order to further investigate the effect of pylon yielding on the seismic response of cable stayed bridge, a comparative analysis between the bridge seismic response with and without consideration of the nonlinear behavior of pylon for TCU076 EW record was carried out with smaller increment of PGA. A comparison of deformed shape of pylon columns is showed in Fig. 10, also shown in the figure as dotted lines are the residual displacements along the tower column. At PGA of 0.5 g, the results in two cases are very approximate, and the residual displacement is very small, which means slightly plasticity occurred yet at this PGA level. With increment of PGA, the differences between the deformed shapes of pylon columns in two cases are generated progressively, but as can seen the residual displacements are still very small.

In these figures, the symbol L and U represent the position of the lower strut and the upper strut, respectively.

Table 4 gives the effect of pylon yielding on the bridge key seismic induced displacement. The result indicates that there is not a remarkable effect, which indicates the nonlinear plastic pylon design may not increase the seismic induced displacement that is not favored.

Fig. 11 shows the curvature envelopes along the height of pylon columns with increment of PGA for TCU076 EW record. As can be seen, the first yielding developed at around the bottom of pylon at PGA of 0.5 g, and then the plastic region was generated with the increment of curvature at PGA of 0.6 g. When PGA was up to about 0.7 g, the plastic region extended to the whole lower pylon columns, and further to the middle columns at PGA of around 0.8 g. It also can be observed

Table 4 Effect of pylon yielding on the seismic induced displacement

Position	PGA			
	0.5g	0.6g	0.7g	0.8g
Pylon tip	2.9%	5.0%	5.2%	-1.5%
End of girder	5.2%	7.7%	8.1%	2.1%
Connection between pylon and girder	-0.9%	-0.8%	-2.3%	-9.3%

Note: the percentage values in the table are the seismic induced displacements with considering yielding divide those of elastic results, respectively.

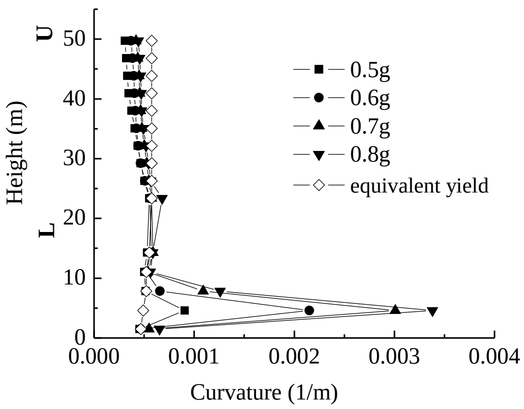


Fig. 11 Curvature envelopes along the height of pylon columns with increment of PGA

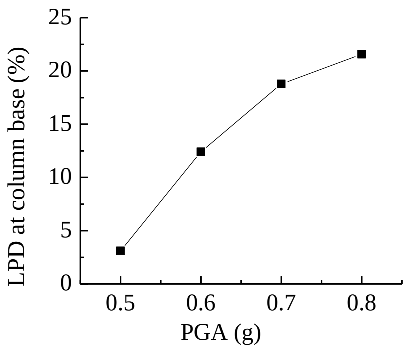


Fig. 12 Variation trend of the level of plastic development at the column base with increment of PGA

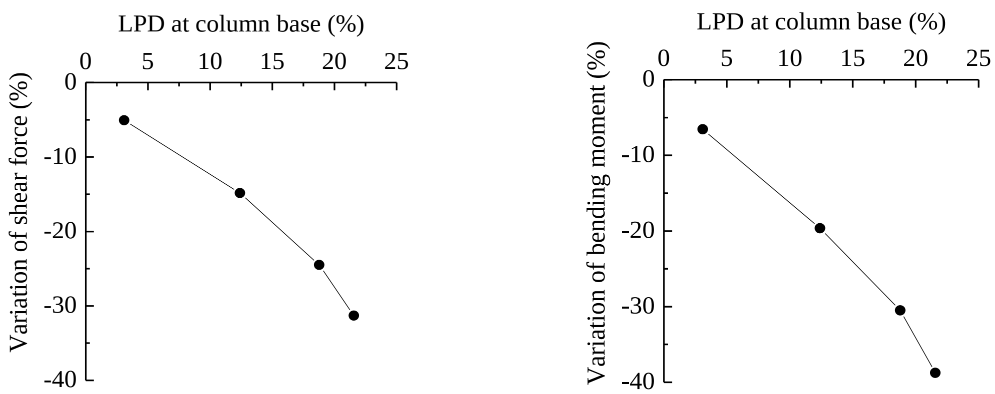


Fig. 13 Effect of pylon yielding on the seismic induced force at the bottom of pylon pile cap

that the curvature demand at the column base is the largest, and also increases most rapidly.

In this paper, the ratio of plastic curvature demand to the ultimate plastic curvature is defined as the Level of the Plastic Development (LPD for short). The level of the plastic development at the column base is the highest. Fig. 12 illustrates the variation trend of the level of plastic development at the column base. With increment of PGA, the level of plastic development is generally linear up at the column base, but the level is not very high with only about 20% even at PGA of 0.8 g, which again indicates that after the yielding, pylon column can reserve considerable plasticity capacity.

Fig. 13 presents the relationship between the LPD at the pylon column base and the effect of pylon yielding on the seismic induced force at the bottom of pylon pile cap. Due to the yielding of pylon columns, the shear force and bending moment at the bottom of pylon pile cap won't continue to increase, instead, they decrease significantly with the development of nonlinear behaviour of pylons. However, if the column section is strengthened to be elastic (which is usually the first choice of the engineering practice in China), the seismic response at the bottom of pylon pile cap will grow linearly with increment of strong ground motion intensity, which will result in more pile numbers or bigger pile diameter design for the pile foundation eventually.

From the discussion above, it can be observed that with the development of nonlinear behavior of pylons, the longitudinal displacement at the key locations won't increase too much but the seismic demand of the substructure will be obviously decreased. Meanwhile, after the yielding, pylon column can still reserve considerable plasticity capacity to ensure the safety of the bridge.

5.3 Some discussions on Strategy A and Strategy B

For this case study, as shown in Figs. 5 and 6 and Table 3, the reduction of the earthquake induced structure response increases as the maximum damping force of each damper increases. For 51MZQ record, when the maximum damping force of each damper is up to 2790 kN, the earthquake induced structure response can be kept in the desired range. However, for the earthquake record stronger than 51MZQ record, in order to limit the seismic response, a even bigger damper with the maximum damping force of more than 2790 kN will be anticipated. At present, the maximum damping force of the largest damper applied is 3025kN in the Sutong bridge

(Ye 2004). Therefore, although, using supplemental dampers might be an effective way in keeping the main structure still in elastic stage, the cost of extra big dampers will be obviously inevitable.

Due to this reason, also illustrated through this case study, the plastic pylon design seems to be another option, since the seismic demand of the substructure can be reduced by allowing the pylon develop certain plasticity, and the reduction increases as the development of plasticity in pylon column increases. Meanwhile, with the development of nonlinear behavior of pylons, the longitudinal displacement at the key locations doesn't seem to be affected much by the yielding of pylon columns. In fact, the strength of the pylon observed through the comparison of the displacement distributions along the pylon column with and without the consideration of the plasticity in Fig. 10 indicates that the stiffness of the overall pylon structure was not degraded under the excitation from 0.5 g up to 0.8 g. Moreover, in Fig. 11, it can be seen that except the lower column part, the curvatures along the pylon column are still less than the equivalent yield curvature, which means the upper part are almost elastic, and the max. curvature near the pylon bottom is only 0.0035 while the capacity is about 0.016 seen from Fig. 8 and Fig.9, which means after the yielding, the pylon can still reserve considerable plasticity capacity as shown in Fig. 11, and enough stiffness for the overall pylon will be remained after the input of large earthquake to ensure the safety of the bridge.

However, it is worth pointing out though not covered in this paper, the use of other readily available displacement restrainers such as elastic cables or shape steels are still necessary considering the practical design of bridge expansion joints, when strategy B is adopted.

Therefore, on the premise of ensuring the seismic safety of the bridge, the plasticity capacity of pylon columns might be made use of to some extent to reduce the seismic demand of the substructure, which may avoid increasing the number or diameter of the piles and meanwhile remain the same seismic performance of the bridge through pylon plastic design.

6. Conclusions

In this paper, we illustrated two seismic design strategies of cable stayed bridges subjected to strong ground motions via a case study of a real typical medium span cable stayed bridge in China. The ground motions recorded during 1999 Taiwan Chi-Chi earthquake and 2008 China Wenchuan earthquake were utilized as input to analyze seismic responses of the bridge. Based on the investigation, the following conclusions can be drawn:

1) For traditional pylon elastic design, the earthquake induced structural responses can be kept in the desired range provided with proper parameters, however, the extra cost of custom specified viscous dampers will be inevitable if strong ground motion has to be considered.

2) According to the current bridge design specifications for the concrete cable stayed bridge pylons in China, the pylon columns may still yield even with carefully seismic design if subjected to strong ground motions without using supplemental dampers. But it can reserve considerable plastic capacity after the yielding.

3) By allowing the pylon develop certain plasticity, the displacements at key locations won't increase too much compared to the elastic design, but a significant decrease of the seismic response at the pylon base can be expected, which indicates the pylon plastic design might be an alternative design strategy when strong ground motion has to be considered.

But still, it cannot be concluded herein to say which one is better, under the subject of developing effective seismic design strategy for cable stayed bridges with concrete pylons subjected to strong ground motions in China, this preliminary research shed a light on the further investigations on seismic performance of such kind of bridges with the consideration of using certain plastic capacity of concrete pylons.

Acknowledgments

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