Seismic Design Strategy of Cable Stayed Bridges Subjected to Strong Ground Motions

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SUMMARY:
One typical medium span cable stayed bridge is selected to investigate its nonlinear plastic behavior and the effect on the seismic responses of other structural components. The nonlinear behavior is considered by different type of plastic elements based on the ratio of sectional capacity to linear seismic demand. A series of nonlinear time history analysis is then carried out under strong earthquake inputs with increasing intensities. The result shows that the yield sequence of the pylon columns and auxiliary piers may be different due to different ground motions and even under the same ground motion but with different intensities; meanwhile significant force redistribution is observed in columns owing to the yielding in plastic regions, however, the pylon still conserves certain plasticity margin according to the current design practice in China, which indicates the pylon limited ductility design might be an practical alternative subjected to strong ground motions.

Keywords: seismic design strategy, cable stayed bridge, strong ground motions, nonlinear plastic behavior

1. INTRODUCTION

Strong ground motions, such as the Northridge, Kobe, Chichi and Wenchuan earthquakes, usually contain special features that from ordinary ground motions in common sense, and hence resulted much more severe damages to many civil engineering structures[Nakashima et al. 2000, Liao et al. 2004 and Chopra et al. 2001]. The related study is therefore a very important topic for both the seismological and civil engineering communities [Makris et al. 2004, Park et al. 2004 and Alavi et al. 2004].To ensure the safety of these structures, especially some important bridges that serve as the lifelines after the earthquake attack, their seismic performances need to be re-evaluated. Considering the widely application and the importance in the traffic network, this research focuses on the type of cable stayed bridge that has become the most widely used type of bridges with the rapid development in the recent decades in China. However, as one kind of the important lifelines, its seismic performance under the strong earthquakes is still unknown, and there is few specific seismic design strategy clearly stated in most codes and criterions [GSDHB 2008 and AASHTO 2007].

Traditionally, bridge pylons are designed to be elastic even under occasionally happened earthquakes; meanwhile, seismic energy absorption such as viscous dampers will be used to restrain the longitudinal displacement [Ye et al. 2004]. However, excessive seismic responses may occur if strong ground motions are likely to happen, which may induce the visible cracks at the concrete pylon base or large displacement exceeding the stroke of the damper at the connection of longitudinal girder and pylons. K. C. Chang et al. [Chang et al. 2004] showed that the Chilu cable-stayed bridge sustained significant damage in the 1999 Chi-Chi earthquake. In the bottom region of the pylon, spalling and splitting of the concrete around the core was evident and plastic hinge was exposed. Therefore, effective seismic design strategy should be applied to ensure the safety of these important lifeline structures under the strong earthquake shaking. In an aim to develop possible alternative seismic design strategy of these bridges subjected to strong ground motions, the concept of pylon limited ductility design is proposed herein.

In this paper, one typical medium span cable stayed bridge is selected to investigate its nonlinear plastic behavior and the effect on the seismic responses of related structural components. The nonlinear mechanical behavior is considered by different type of plastic elements based on the ratio of sectional capacity to linear seismic demand. A series of nonlinear time history analysis is then carried out under strong earthquake inputs with increasing intensities.

2. ANALYTICAL MODEL

The analyzed bridge, with a 90m 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 [Habibullah et al. 1999]. The computer model is shown in Fig. 2.1.. The beam, pylon 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 columns and piers are 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 et al. 1999]. Therefore, the geometric nonlinearity is not considered, but both linear and nonlinear analysis start from the deformed configuration and the internal forces 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 is 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.

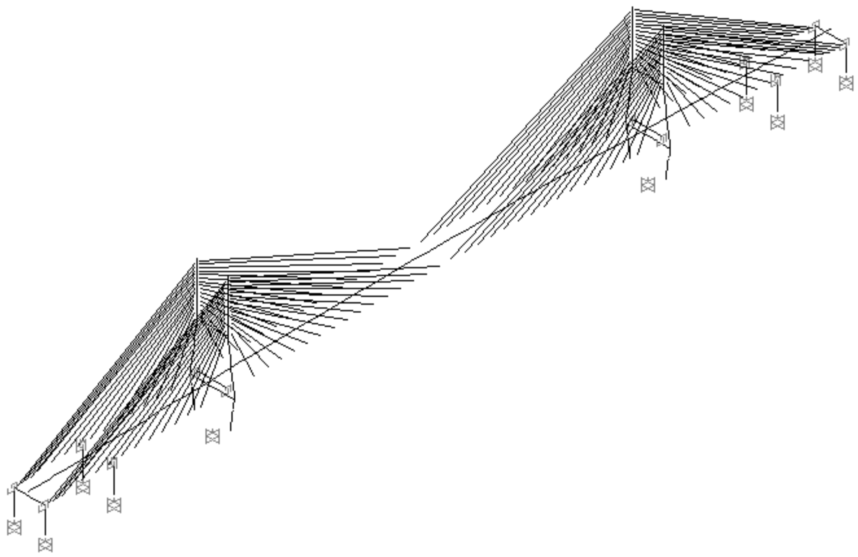


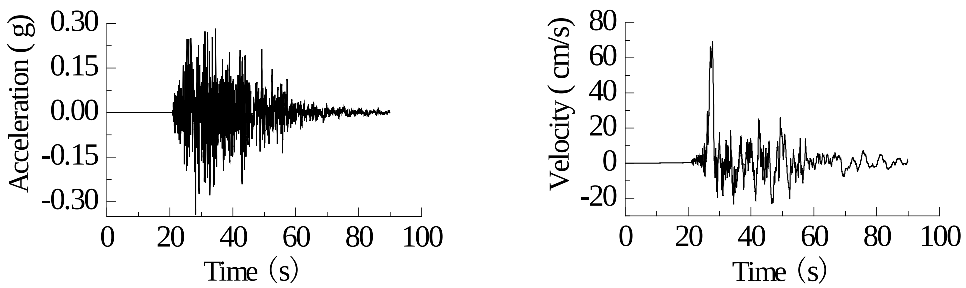
Figure 2.1. Dynamic analytical model

3. INPUT GROUND MOTIONS

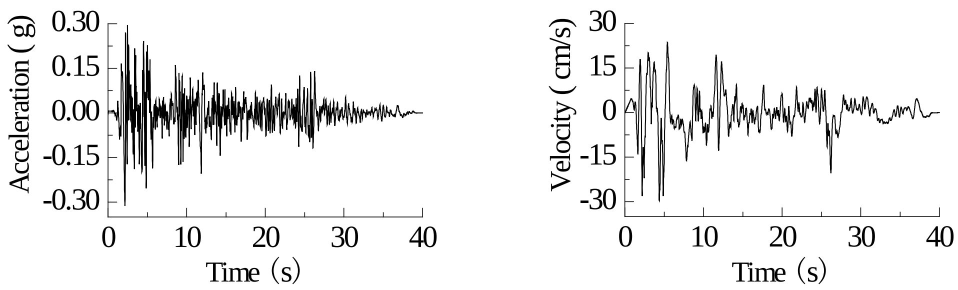
To evaluate the effects of different strong ground motions on the seismic response behavior of cable stayed bridge, two earthquake records are used in the analysis. One is recorded during the 1999 Chi-Chi earthquake in Taiwan, named as TCU076 record, and the other is El Centro record during the 1940 Imperial Valley earthquake. The E-W component of TCU076 and the N-S component of El Centro are selected for the analysis. Information pertinent to the ground motions is listed in Table 3.1., the acceleration and velocity time history curves are shown in Fig. 2., and the corresponding response spectra with damping ratio of 3% are shown in Fig. 3.

Table 3.1. 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
1940 Elcenro NS	7.0	0.313	29.8	13.3



a) Tcu076 EW



b) El Centro NS

Figure 3.1. Time histories of selected ground motions

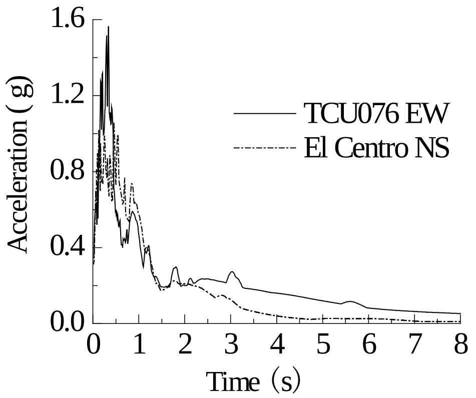


Figure 3.2. Response spectra of selected ground motions

4. THE SIMULATION OF THE NONLINEAR BEHAVIOR

The nonlinear mechanical behavior in the analyzed bridge is considered by the plastic hinge element for its clarity of concept, simplicity of modeling, and efficiency of calculation in relative to the solid element and the fiber element. In sap2000, there are different types of plastic hinge elements, such as uncoupled M hinge, interaction PMM hinge, fiber hinge and nonlinear link element. The effect of the variation of axial force on the plastic behavior can be considered by the PMM hinge and fiber hinge, and the M hinge and PMM hinge are not recommended for dynamic analysis as the convergence difficulties [Ady et al. 2008].

In order to determine where the plastic hinge elements should be properly inserted, the variation amplitude of axial force of different structural components subjected to the longitudinal ground motions should be calculated via linear analysis first. The modal analysis is then conducted. The first mode of the analyzed bridge is the floating vibration in the longitudinal direction, and the corresponding period is 6.1s. As the dominated mode in the longitudinal direction, the modal participating mass ratio of this mode reaches 76% of the total mass. Based on the results of the modal analysis, the longitudinal response spectrum analysis was carried out by input the response spectra described in Fig. 3.2.. Table 4.1. presents the ratio of seismic induced axial force to dead load axial force for the bottom of the pylon column and piers. The result shows that under the longitudinally input ground motions, the variation amplitude of axial force of pylon column is so small as to be ignored, while those of the piers are much larger. Therefore, the nonlinear link elements are adopted to simulate the nonlinear behaviour of pylon columns, while the fiber hinges are selected for the piers. Fig. 4.1. presents the typical bilinear curve of nonlinear link element, and the key parameters are calculated by Xtract [Imbsen 2002]. Fig. 4.2. shows the material model [Mander et al. 1988 and Chang et al. 1994] for the concrete fiber and steel fiber of the fiber hinge element. The modified Takeda model [Roufaiel et al. 1987] is used as hysteresis rules of all these plastic hinge elements.

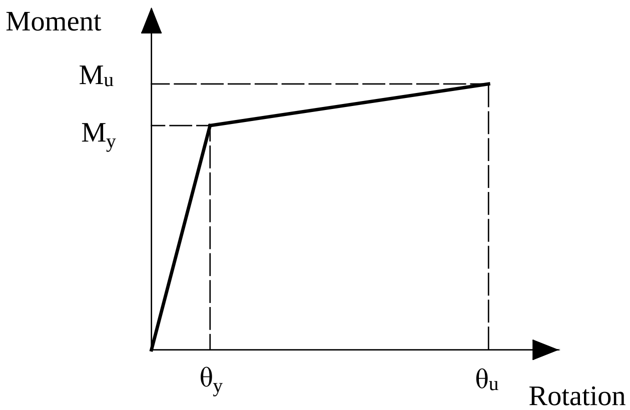


Figure 4.1. Typical bilinear curve of nonlinear link element

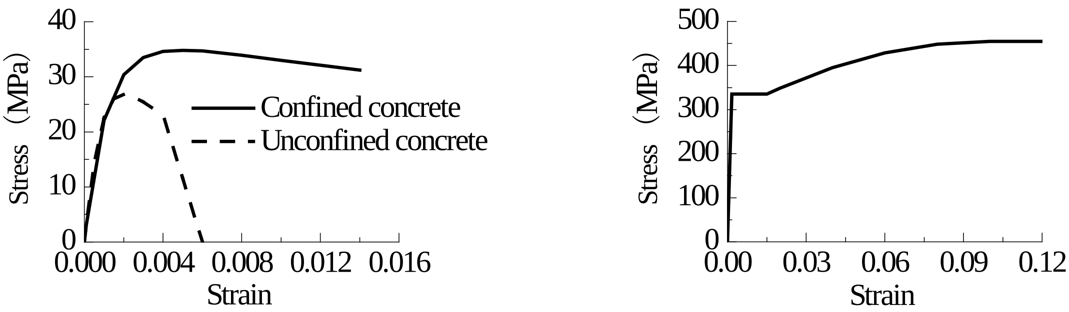


Figure 4.2. Material model of fiber hinge element

Table 4.1. variation of axial forces of different structural components

Position	Earthquake record	
	Tcu076 EW	El Centro NS
The bottom of the plyon	5%	3%
The bottom of the auxiliary pier	87%	39%
The bottom of the side pier	15%	17%

Note: the percentage value in the table represents the ratio of seismic induced axial force to the dead load axial force.

5. NONLINEAR SEISMIC DEMAND AND CAPACITY ANALYSIS

Based on the nonlinear analytical model with proper plastic elements, a series of nonlinear time history analysis were carried out under strong earthquake inputs using Incremental Dynamic Analysis (IDA) method starting from 0.1g for both Tcu076 EW record and the El Centro NS record, and with the increment of 0.1g. The dead load case was run first to get the correct initial state for the nonlinear dynamic analysis.

5.1. The Yield Sequence of Main Components

Table 5.1. presents the PGA of the two strong earthquake inputs corresponding to first yield of the main structural components. With respect to the El Centro NS record, the auxiliary pier and the side pier yield when the PGA is 0.5g and 0.4g respectively, while the pylon columns yield at the PGA of 1.2g. However, with respect to the Tcu076 EW record, the pylon columns yield before the piers. That is to say, the yield sequence of the pylon columns and the piers may be different due to different ground motions, and the characteristic of ground motions may cause different seismic responses in the main structural components of cable stayed bridge. Table 5.2. lists the moment when the pylon columns and the piers yield with the increment of PGA for Tcu076 EW record.

Table 5.1. PGA of strong ground motions corresponding to first yield of the main components

Earthquake record	Pylon	Auxiliary pier	Side pier
Tcu076 EW	0.5g	0.7g	0.7g
El Centro NS	1.2g	0.5g	0.4g

Table 5.2. The moment when the main components yield with the increment of PGA for Tcu076 EW record

PGA	Pylon	Auxiliary pier	Side pier
0.5g	30.00s	\	\
0.7g	27.62s	28.12s	26.62s
0.8g	27.58s	26.7s	26.36s

It can be observed that for the same ground motion but with different PGA, the yield sequence of the pylon columns and the piers may also be different.

Furthermore, in order to improve the persuasion of the conclusions above, the seismic capacity of the pylon column are enhanced by an increment of 30% for the reinforcement ratio (the original ratio is about 1%). Based on the modified bridge model, a series of nonlinear time history analysis with the IDA method were conducted again, and the result shows that the pylon columns still yield before the piers for Tcu076 EW record.

On the basis of the research above, even according to the present design method for the bridge pylons, the pylon columns may yield before the piers subjected to strong ground motions. Therefore, the seismic performance of cable stayed bridge with the consideration of the nonlinear behavior of pylon subjected to longitudinal strong ground motions should be paid more attention to.

5.2. The Nonlinear Behavior Development of Pylon

Based on the same anslysis method, in this section, the study gives emphasis on the nonlinear seismic behavior development of the bridge pylon for Tcu076 EW record. Fig. 5.1. and Fig. 5.2. show the deformation and curvature envelopes along the height of pylon columns with increment of PGA for Tcu076 EW record, respectively. In these figures, the symbol L and U represent the elevation of the lower strut and the upper strut, respectively.

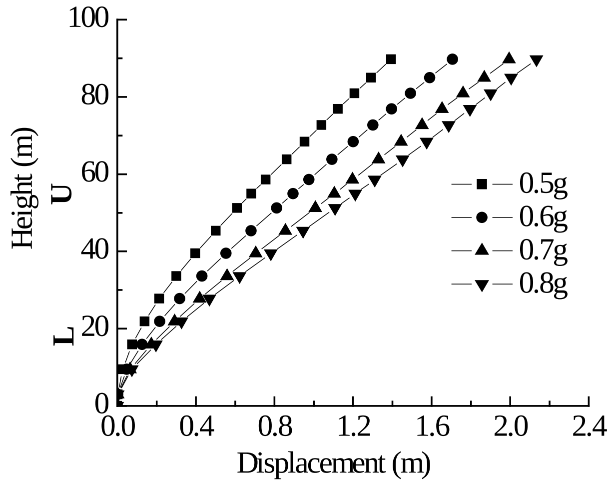


Figure 5.1. Deformation envelopes along the height of pylon columns with increment of PGA

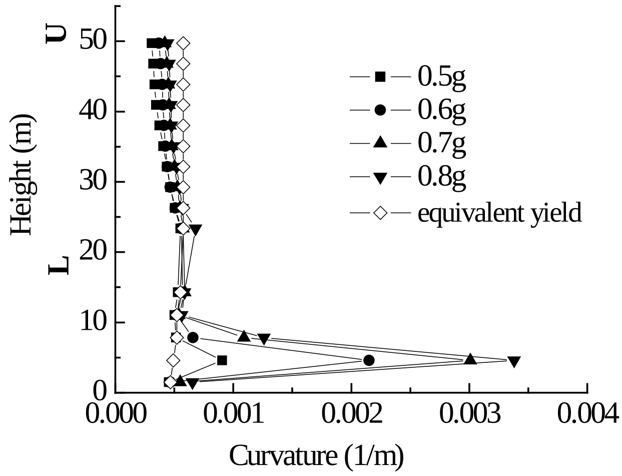


Figure 5.2. Curvature envelopes along the height of pylon columns with increment of PGA

In Fig. 5.1., with increment of PGA, the deformation of the pylon columns increases gradually, but the deformed shapes of columns are generally in consistent. In Fig. 5.2., the first yielding developed at around the bottom of pylon at PGA of 0.5g, and then the plastic region was generated with the increment of curvature at PGA of 0.6g. When PGA was up to about 0.7g, the plastic region extended to the whole lower pylon columns, and further to the middle columns at PGA of around 0.8g. It also can be observed 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. 5.3. 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% at PGA of 0.8g. Therefore, there is no strong nonlinear behavior in the pylons yet.

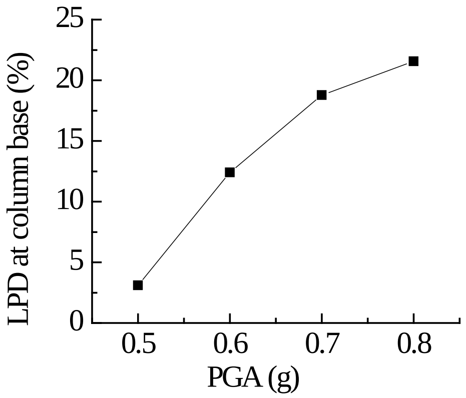


Figure 5.3. Variation trend of the level of plastic development at the column base with increment of PGA

Furthermore, fig. 5.4. shows the relationship between the level of plastic development at the column base and the displacement at the top of pylons. With the development of nonlinear behavior development of pylons, the displacement at the top of pylons increases gradually and almost linearly.

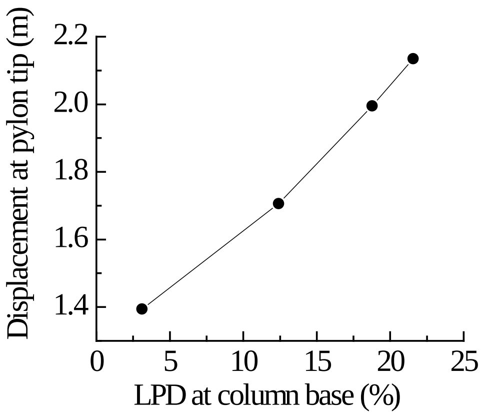


Figure 5.4. Relationship between level of plastic development at the column base and displacement at pylon tip

From the discussion above, according to the current design practice in China, under the strong ground motions, the pylon cannot remain elastic anymore, but after the yielding, the columns can conserve certain plasticity margin with a gradual increment of a pylon displacement.

5.3. The Effect of Pylon Yielding on the Bridge Seismic Response

In order to 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 the consideration of the nonlinear behavior of pylon for Tcu076 EW record was carried out.

A comparison of deformed shape of pylon columns is showed in Fig. 5.5.. At PGA of 0.5g, the results in two cases are the same. With increment of PGA, the differences between the deformed shapes of pylon columns in two cases are generated, but very small. Table 5.3. gives the effect of pylon yielding on the bridge key seismic induced displacement. The result indicates that there is not a remarkable effect.

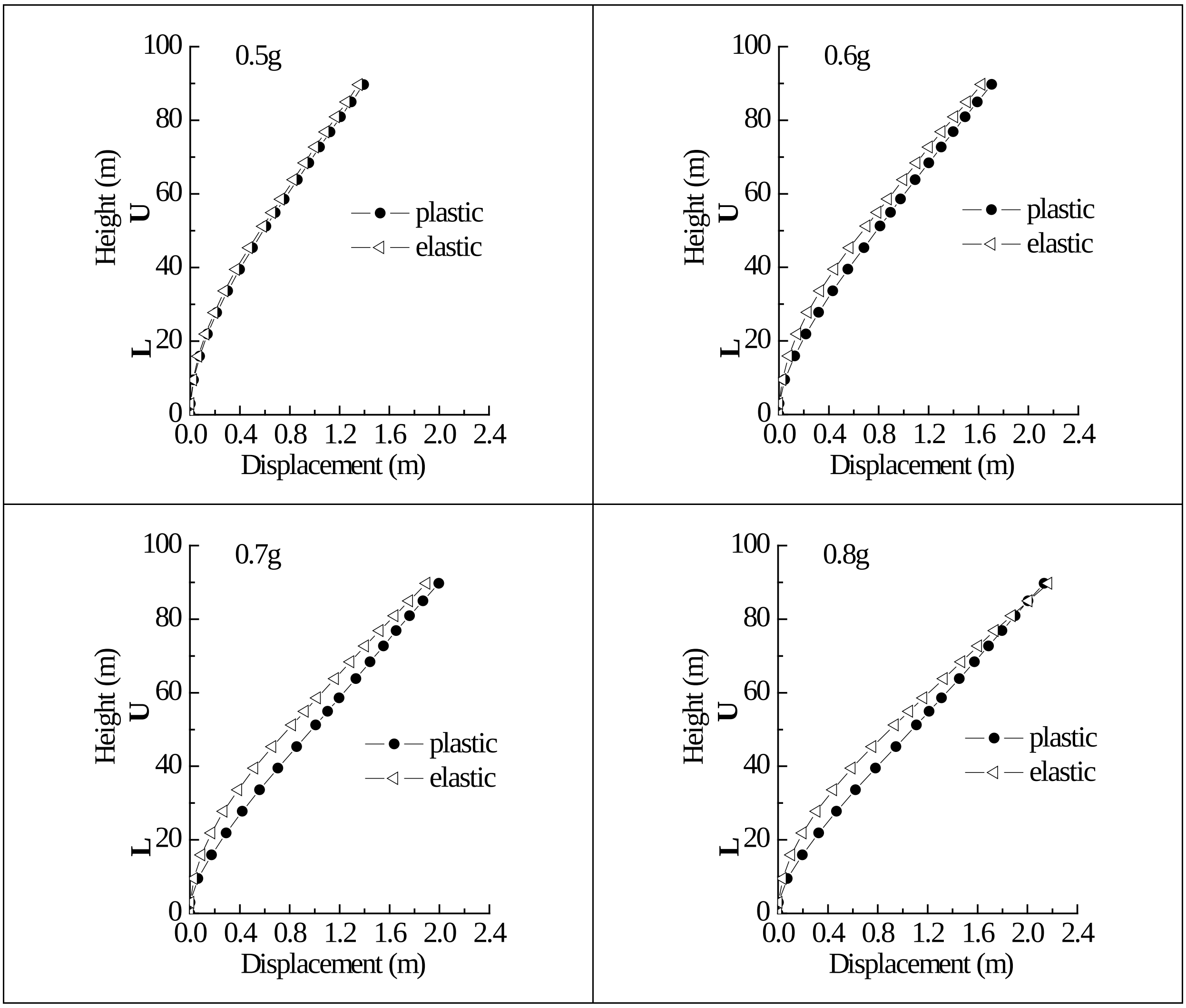


Figure 5.5. Comparison between deformed shape of pylon columns with and without the consideration of the nonlinear behavior of pylon

Table 5.3. 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. 5.6. presents the relationship between the level of plastic development 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, the sheer force and bending moment at the bottom of pylon pile cap decrease significantly with the development of nonlinear behaviour of pylons. However, if the column section is enlarged to be elastic, the seismic response at the bottom of pylon pile cap will grow linearly with increment of strong ground motion intensity.

In addition, with the development of nonlinear behavior of pylons, the longitudinal displacement at the pylon tip and the end of girder won't increase too much. Therefore, a concept of pylon limited ductility design can be proposed when subjected to strong ground motions. On the premise of ensuring the seismic safety, the ductility capability of pylon columns can be made use of to some extent to reduce the seismic demand of the substructure, which avoids increasing the number or diameter of the piles and meanwhile remains the same seismic performance of the bridge.

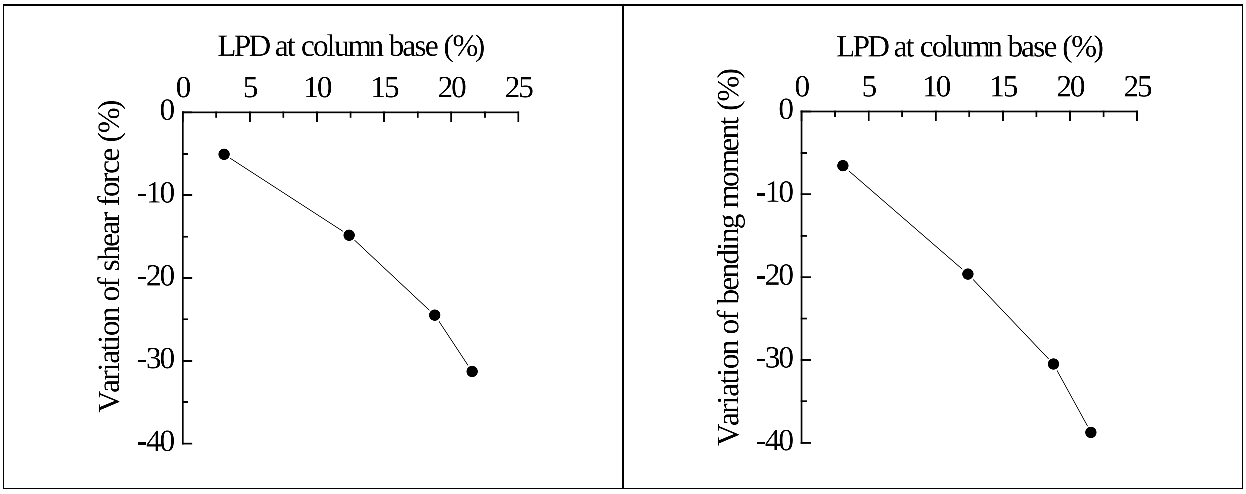


Figure 5.6. Effect of pylon yielding on the seismic induced force at the bottom of pylon pile cap

6. CONCLUSIONS

Under the subject of developing effective seismic design strategy of cable stayed bridges with concrete pylons subjected to strong ground motions, a preliminary research on the seismic performance of cable stayed bridge with the consideration of the nonlinear behavior of concrete pylon excited by longitudinal strong ground motions was conducted herein. Using one typical medium span bridge with concrete pylon in China as the analytical model, a series of nonlinear time history analysis was carried out under strong earthquake inputs with IDA method, and the nonlinear behavior of pylons and piers was considered by the proper plastic hinge elements. Based on the investigation above, the following conclusions can be drawn:

1. According to the current bridge design specifications for the concrete cable stayed bridge pylons, the pylon columns may still yield when subjected to strong ground motions. However, it can conserve certain plasticity margin after the yielding.
2. Due to the yield of pylon, the longitudinal displacement at the pylon tip and the end of girder won’t increase too much compared to the elastic design, but a significant decrease of the seismic response at the pylon base can be observed.
3. If the ductility capacity of the pylon columns can be made use of, the seismic demand of the substructure will be correspondingly decreased.

ACKNOWLEDGEMENT

This research is financially supported by the National Science Foundation of China at the granted no. of (51008225), the Fundamental Research Funds for the Central Universities and the Western Transportation Construction Projects of China at the granted no. of (200631882225).

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