Nonlinear Seismic Response Analysis and Seismic Performance Evaluation of A Reinforced-Concrete Cable-Stayed Bridge Tower Using IDA Method

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ABSTRACT :

Nonlinear Seismic Response analysis and Seismic performance evaluation for cable-stayed bridge are very important, especially for the tower of it. Based on the recent researches, A simplified single tower finite element model was developed to modelling the seismic response under the longitudinal excitation in this paper. In order to investigate the nonlinear seismic response of the tower, a fiber flexural element was adopted and the P-delta Effect was considered in present study. On the other hand, the incremental dynamic analysis (IDA) of this Tower has been done herein, in order to evaluate the seismic performance of the tower more reasonably. The results show the damage zone (or the plastic zone) for this tower mainly concentrated in three regions. According to IDA analysis and the summary curves of it, seismic damage state evaluation can be more reasonable which has more probabilistic characteristic. Finally, some suggestions on the definition of the global ductility factor for a complex structure like the tower of cable-stayed bridge were given herein.

KEYWORDS: Nonlinear Seismic Response, Seismic Performance Evaluation, Cable-Stayed Bridge, Reinforced-Concrete Tower, IDA

1. INTRODUCTION

In 1990's, The 3D nonlinear seismic response analysis of cable-stayed bridge was studied by Aly S. Nazmy and Ahmed M. Abdel-ghaffar [1990]. In that paper the author paid more attention to the geometric nonlinearities, such as cable-sag effect, axial force-bending moment interaction and the large displacements. The conclusion showed that for the cable-stayed bridge with the longer center span (>=610m) including the geometrical nonlinearities would reduce the response significantly under uniform earthquake excitation. It also showed the dead load deformed state was the basis of the reasonable nonlinear seismic response analysis. With the development of the computer science, more possible effects of nonlinearities can be included in the analysis. For instance, Wei-Xin Ren [1999] has considered the material nonlinearity due to the stiffening steel girder yielding. The 2D Plane beam element was adopted in order to reduce the computing time, and the maximum equivalent plastic strain ratio was proposed to evaluate the elastic-plastic seismic damage for the local element in his research. However, his research still assumed the cable, and reinforced-concrete tower remained in elastic state under the earthquake action.
As the performance-based design philosophy were followed, the seismic performance of the tower of long-span cable supported bridges need evaluating and designing in details, as well as, the damage states for different performance levels from fully operational to collapse should be clarified. Kazuo Endo, [2004] and Shehata Eldabie ABDEL RAHEEM, [2003] have done some researches at the tower considering the material nonlinearity. In their researches the steel tower was modeled by shell and fiber elements, the strength and damage progress characteristics were obtained, and the acceptable ductility capacity for the steel tower structure exceeding the elastic limit was proposed. A fiber beam-column element is a good tool to investigate the spread of plastic zone in the tower under the great earthquake ground motion. So far, the nonlinear seismic response of RC tower in cable supported bridge has not been carried out in details, which will be studied in this paper.
On the other hand, the incremental dynamic analysis (IDA) which was carried out by Dimitrios Vamvatsikos

and Allin Cornell [2003] gave us a good thought to study the structure performance under seismic loads, especially for the nonlinear model, which can't be predicted well by Nonlinear static pushover analysis. Therefore, in this paper fiber elements were used to model the tower and the IDA method was adopted to investigate the seismic performance of the tower.

2.COMPUTING MODEL AND NOLINEARITY CONSIDERATION

2.1. Computing Model

The tower of a cable-stayed bridge can be divided into to two parts, the anchorage zone and the pylons. Due to the high circumferential compression stress and the complex constitutive model for the anchorage zone of the tower, the elastic beam elements were used in this study. For the pylons, which made from high strength concrete reinforced by steel bars, the fiber beam-Column elements supported by the Open System for Earthquake Engineering Simulation (Opensees) [2007] were used. Three different material constitutive models were used in a section, the confined Mander model [1988] (modeling confined concrete), the unconfined Mander model (modeling cover concrete), and the Menegotto Pinto model (modeling the longitudinal steel bars). A typical cable-stayed bridge tower with the height of 235m and its critical section used in this paper are shown in Fig.2.1. The constitutive model of Mander model and the constitutive model of the steel bar are illustrated in Fig. 2.2. The parameters of the model are shown in Table 2.1.

In longitudinal seismic response analysis, a simplified tower model suggested by Yan Hai-quan [2007] has been modified and applied in this paper. In order to consider the inertia force of the main girder more reasonably, the main girder and the cables are modeled as an oscillator linked to the highest anchorage point of the tower. The weight of the main girder is taken as the mass of the oscillator, the longitudinal stiffness of the oscillator is calculated by the period of the floating mode of the system made up of cables and the beam. Eqn. 2.1 is used to calculate this stiffness K1.

$$K_1 = \left(\frac{2\pi}{T_F}\right)^2 \times m_b \qquad (2.1)$$

For the soil is stiff enough, and the tower is fixed at the bottom, so the soil-structure interaction is not included herein.

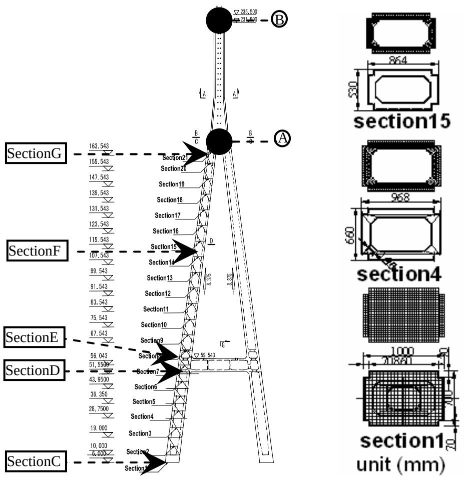

Fig. 2.1 FEA model division of the tower

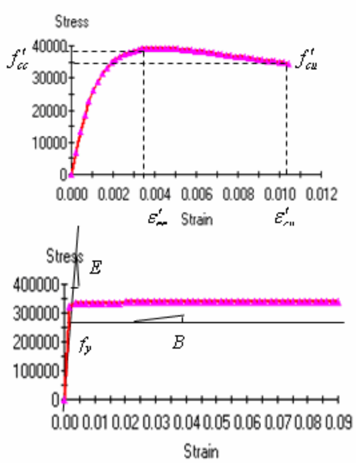

Fig. 2.2 Constitutive model of the confined concrete and steel bar

Table 2.1 Material parameters of section 15

Material	f'cc(kPa)	ε 'cc	f'cu(kPa)	ε 'cu	Material	f'y (kPa)	E	B
Confine concrete	3.838E+04	0.004	3.374E+04	0.011	Confine concrete	3.35E+05	2.00e8	0.0003
Cover concrete	3.240E+04	0.002	2.668E+04	0.004				

2.2 Nonlinearity Consideration

The material nonlinearities are considered as above. In order to consider the geometric nonlinearity of the tower, the nonlinear history analysis including P-delta effect has been done after the dead load analysis in this paper.

3. NONLINEAR SEIMIC DEMAND ANALYSIS

To investigate the seismic performance of the tower, ten ground motion records have been adopted, which are taken out from the pacific earthquake engineering research center (PEER) national ground acceleration (NGA) database. To reduce dispersion of the seismic response, the fields where these earthquakes occurred all belong to the type I, according to the *Specifications of Earthquake Resistant Design for Highway Engineering* in China. The epicenter distance of these waves are less than 20kM as shown in table 3.1 The elastic acceleration response spectra and displacement spectra of the ten waves can be seen in Fig. 3.1. On the other hand, for IDA analysis, recent researches have shown that taking the Sa(T_1, ξ) as an Intensity Measure(IM) for scaling, can be more effective than PGA as an IM, in reducing the dispersion of the seismic response of the structure. Therefore, for this study the Sa (T_1, 0.03) has been scaled from 0.001g to 0.035g, and the incremental step of 0.0029g is adopted. The damping ratio 0.03 is used here mainly considering that the main girder and cables are made of steel.

Table 3.1 Ground motion records used for IDA analysis

Record ID	Earthquake	Station	Record/ Component	PGA (g)	Tp	Sa (T_1,0.03)	M	EpiD (km)
wave1	Helena, Montana 1935/10/31 18:38	2022 Carroll College	HELENA/ A-HMC180	0.15	0.140	0.0003	6.000	6.310
wave2	Helena, Montana 1935/10/31 18:38	2022 Carroll College	HELENA/ A-HMC270	0.173	0.280	0.00065	6.000	6.310
wave3	Helena, Montana 1935/10/31 19:18	2229 Helena Fed Bldg	HELENA/ B-FEB000	0.047	0.080	0.00002	6.000	6.310
wave4	Helena, Montana 1935/10/31 19:18	2229 Helena Fed Bldg	HELENA/ B-FEB090	0.041	0.060	0.00002	6.000	6.310
wave5	San Francisco 1957/03/22 19:44	1117 Golden Gate Park	SANFRAN/ GGP010	0.095	0.260	0.0001	5.280	11.130
wave6	San Francisco 1957/03/22 19:44	1117 Golden Gate Park	SANFRAN/ GGP100	0.112	0.220	0.0002	5.280	11.130
wave7	Central Calif 1960/01/20 03:26	1028 Hollister City Hall	CTRCALIF/ B-HCH181	0.041	0.300	0.00012	5.000	8.010
wave8	Central Calif 1960/01/20 03:26	1028 Hollister City Hall	CTRCALIF/ B-HCH271	0.063	0.260	0.00016	5.000	8.010
wave9	Hollister 1961/04/09 07:25	1028 Hollister City Hall	HOLLISTR/ C-HCH181	0.072	0.320	0.0002	5.500	18.920
wave10	Hollister 1961/04/09 07:25	1028 Hollister City Hall	HOLLISTR/ C-HCH271	0.075	0.420	0.0004	5.500	18.920

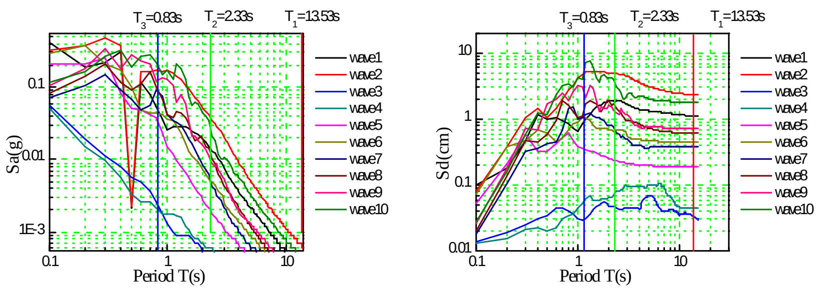

Fig. 3.1 Elastic response spectra of the ten ground motion records

For evaluating the damage of the tower, deformations of the critical position are recorded during every time history analysis. The curvature of each integrator point of the element is obtained. Due to the space limitation, only seismic responses excited by wave2, wave3, wave7, wave9 are shown in Fig. 3.2. The displacements in the longitudinal direction of each point are recorded as shown in Fig. 3.3. It should be stated that, in order to show the increasing trend of the curvature of the tower as the intensity level increases, the curvatures are normalized by insuring tower bottom curvature to be positive. The displacements of the tower are normalized by insuring tower top displacement to be positive.

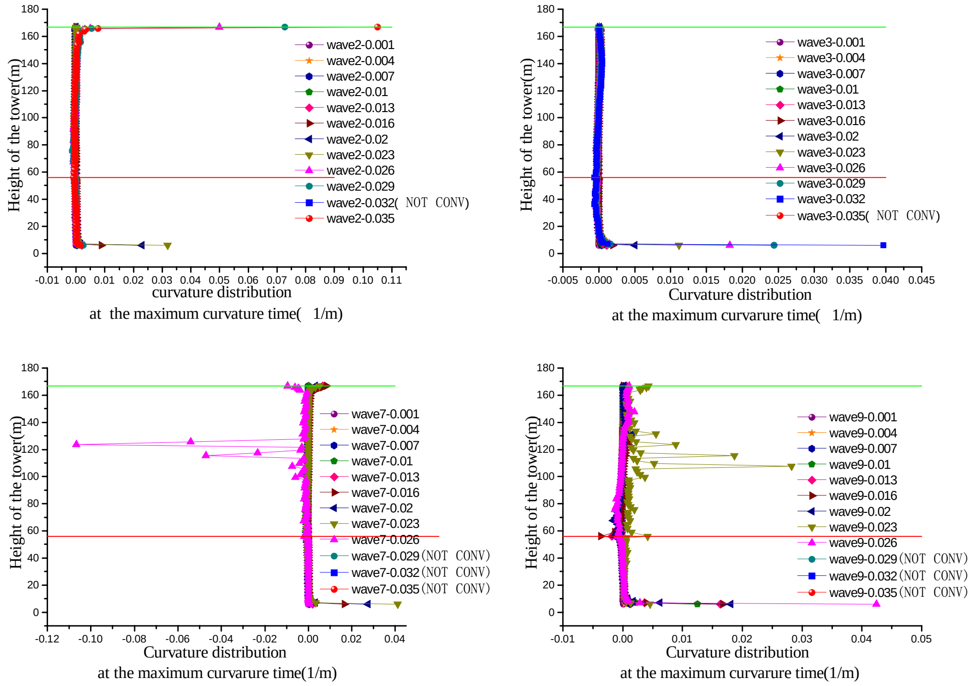

Fig. 3.2 Curvature distribution of the tower at the maximum curvature time

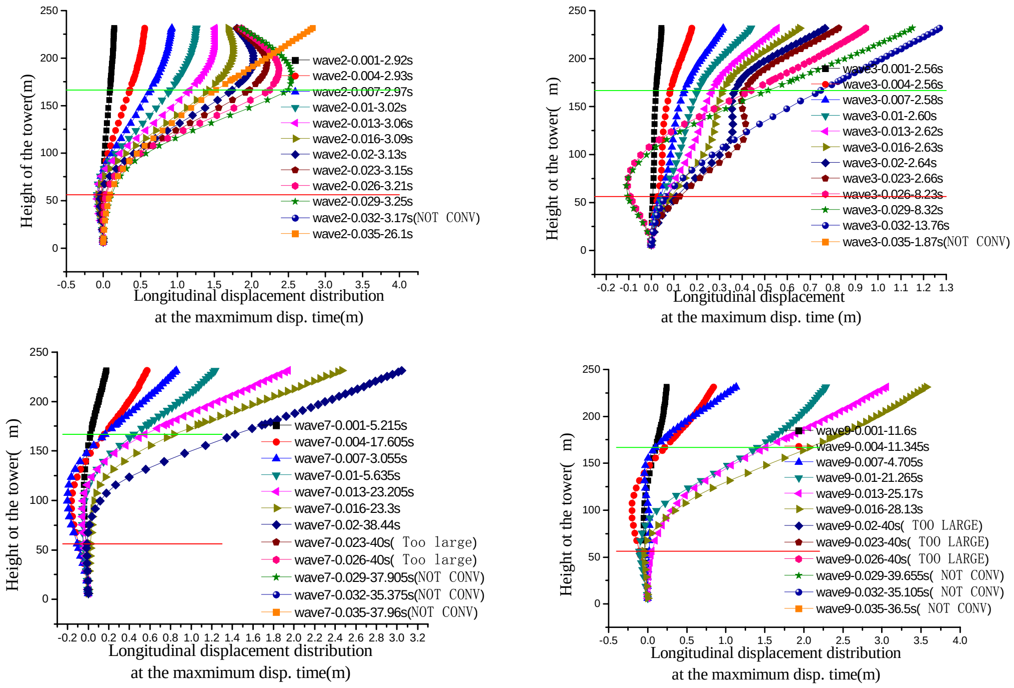

Fig. 3.3 Longitudinal displacement distribution of the tower at the maximum disp. time

From above graphs the conclusion can be obtained that, for different ground motion records or for different scales for a single ground motion record, the damage zone (plastic zone) does not occur in a fixed place. But there still exist some rules as the following.

1. Under longitudinal ground motion excitation, the damages of the tower mainly concentrate on three zones: the two pylon legs' bottoms (SectionC), the two pylon legs' tops (SectionG), the middle zones of the two pylon legs' upper parts (SectionF).
2. The maximum displacement occurs in two critical positions, the intersection point of the two pylons (PointA), and the top of the tower (PointB).
3. The damages caused by Wave1, wave2, mainly concentrates on the pylon leg bottom (SectionC), when the Sa(T_1) of the ground motion is below 0.013g. As the intensity increases, the maximum curvatures of the tower occur at section C, section D section E and section G because of the apparent increasing contribution of high order modes. When the Sa is above 0.023g, the plastic curvature mainly concentrates on the SectionC and SectionG, and the value is nearly the ultimate curvature, which may cause the collapse of the tower.
4. The damages caused by Wave3, wave4, wave5, wave6 mainly concentrates on the pylon leg bottoms, even when the Sa(T_1) is above 0.01g, which will cause the yield of critical section in some important part. The reason may be that the spectra shapes of these four ground motion are smoother than other ground motions.
5. Under Wave7, Wave8, wave9 and wave10 with IMs below 0.01g, the damages mainly occurs at sectionC and sectionG. When IMs are above 0.02g, (this intensity would cause some section exceeding their ultimate curvatures), the maximum curvature occurred sometimes at the pylon leg middle, because this curvature level is much higher than the one caused by the lower intensity level. This kind of damage can be thought as the dynamic instability.

4.DAMAGE STATE DEFINITION

To analysis the seismic performance of the tower, different damage state or (limit state) can be defined with two major measures. One is the flexural curvature at the critical section. The other is the displacement of the tower at the critical point. For the former, the axial forces as the maximum curvature occurred has been adopted to calculate the different damage state of the different section, which are decided by the material strain levels in different states. The material strain level defined by Arzoumanidis S [2005] in Taoma new suspension bridge tower is modified in consideration of some recent research result of bridge concrete piers. The material strain levels for different states are listed in table 4.1.The critical section, includes SectionC, SectionD, SectionE, SectionF, SectionG, as shown in Fig. 2.1. Although the different axial forces are used to calculate the curvature for different intensity level of different ground motion records, only 50% fractile curvature are shown in figures for clarity.

Table 4.1 Damage states and strain levels for the critical sections

Damage states	Damage description	Damage measures of strain levels
No damage	there's no crack on the concrete, the steel bars don't yield	$\varepsilon_s \le \varepsilon_y = 0.001675$
Slight damage	the spalling of the cover concrete does not occur, and the cracks were less than 1cm strain.	$0.001675 < \varepsilon_s \le \varepsilon_h = 0.015;$ $\varepsilon_c \le 2\varepsilon_{co} = 0.004$
Repairable damage	the damage on the section were limited to the repairable state economically and technically	$0.015 < \varepsilon_s \le 0.55\varepsilon_{su} = 0.0495;$ $0.004 < \varepsilon_{cc} \le 0.75\varepsilon_{ccu} = 0.007875$
Extensive damage	the damage on the section cannot be repaired; the tower doesn't lose bearing capacity.	$0.55\varepsilon_{su} < \varepsilon_s \le \varepsilon_{su} = 0.09;$ $0.75\varepsilon_{ccu} < \varepsilon_{cc} \le \varepsilon_{ccu} = 0.0105;$
Complete damage	the tower collapsed	$\varepsilon_s > \varepsilon_{su} = 0.09;$ $\varepsilon_{cc} > \varepsilon_{ccu} = 0.0105;$

The drift ratio measures of two critical points (as shown in Fig.2.1) are adopted as defined by Ahmed Ghobarah

[2001], which is show in Table 4.2.

Table 4.2 Damage states measured by drift ratio for the critical node

Damage state	Drift ratio at point B At the bottom of the anchorage zone	Drift ratio at point A At the top of the tower
No damage	U_A<=0. 2%	U_B<=0. 2%
Slight damage	U_A<=0. 5%	U_B<=0. 5%
Repairable damage	U_A<=1. 5%	U_B<=1. 5%
Extensive damage	U_A<=2. 5%	U_B<=2. 5%
Complete damage	U_A>2. 5%	U_B>2. 5%

5. SEIMIC PERFORMANCE EVALUATION USING IDA METHOD

5.1 SEISMIC DAMAGE STATE EVALUATION USING IDA METHOD

Based on the above definition of damage states, seismic performance was studied by IDA analysis as shown in Fig.5.1 ~ 5.2.

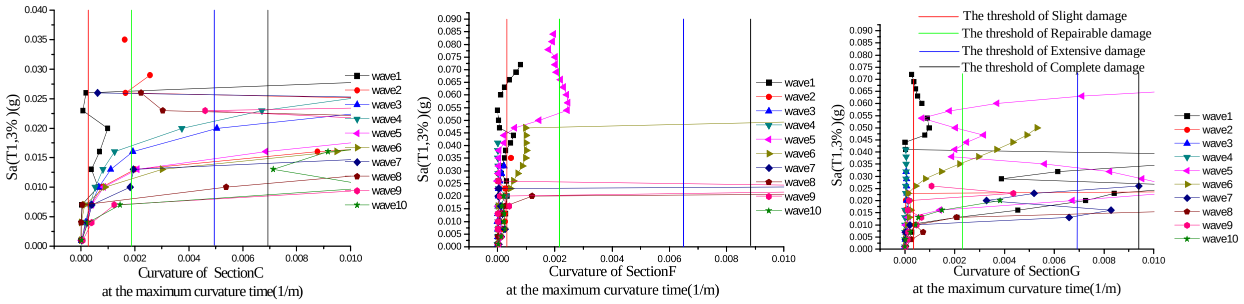

Fig. 5.1 IDA curves and the capacity of critical section at the time of the maximum curvature

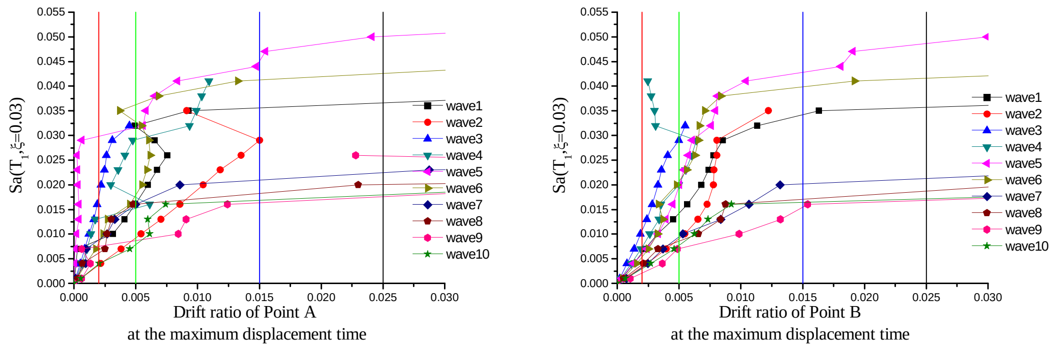

Fig. 5.2 IDA curves and the capacity of critical Point at the time of the maximum displacement

From figure**,** some remarkable characteristics of the seismic performance of the whole tower can be conclude:

1. As the increasing of the seismic intensity, either for curvature or for displacement of the critical point, the dispersion grows larger.
2. At the time of maximum curvature occurs in the tower, the main plastic zone mainly concentrates on sectionC, section F, sectionG.
3. The displacement curve of PointA and PointB also indicate some plastic performance of the tower, the slope of them is gentler than that of the curvature.

To better summarize the seismic performance of the tower, the statistic analysis method has been used herein. The 50%, 84%, 16% fractile of the IDA curves of the maximum curvature of the whole tower and the maximum drift ratio of the whole tower are illustrated in Fig. 5.3.

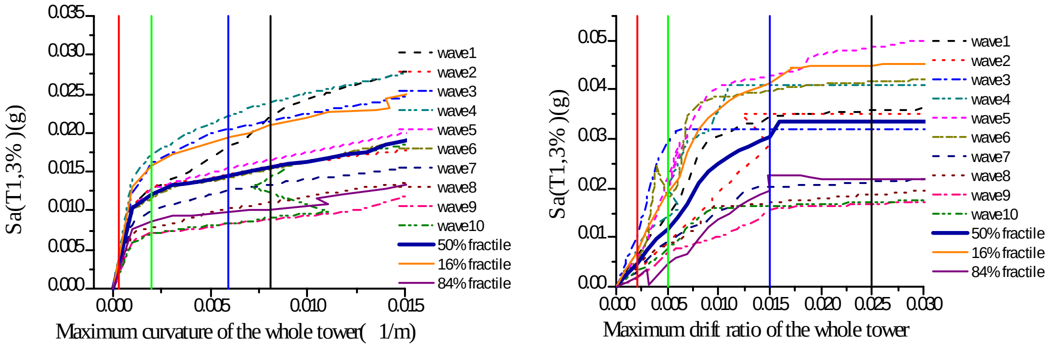

Fig. 5.3 Summary of IDA curves of the Maximum drift ratios and Maximum curvatures of the tower

From the point view of the section damage of the tower, the figures shows below Sa=0.003g tower will keep in no damage state at a probability of 50%, below Sa=0.0119g the tower will keep in slight damage state at a probability of 50%, above Sa=0.0144g the tower will go into extensive damage, and the tower will collapse when Sa is over 0.0155g at a probability of 50%. The same intensity measures for different damage states of the drift ratios of the tower are 0.005g, 0.0118g, 0.03g and 0.035g with the same order.

Compared with the drift ratio as the damage measure, taken the curvature as the damage measure to evaluate the damage state will be more rigorous, especially for the state of repairable damage state and extensive damage state. Because the curvature damage measure has more supporting proof from experiment of bridge piers, it has more reliability than the drift ratio.

5.2 DUCTILIY CAPACITY DICUSSION

To investigate the ductility of the whole tower under different ground motion excitations based on summary of the maximum curvature IDA curves, the yield curvature are assigned by the turning point of the 50% fractile lines. At this point, the curvature equals to 0.001(1/m), earthquake intensity equals to 0.01g, normalized by the two threshold value. The R-μ relationships is plotted in Fig. 3.5., from which it can be seen if the tower goes into plastic state at large ground motion level, the maximum available ductility factor can be thought as 8, the strength reduction factor of the tower can be thought as 1.5. Using the same earthquake intensity level, which is 0.01g to normalizing the yield drift ratio, the yield value of the drift ratio will be 0.004. It can be seen the maximum available ductility factor of the drift ratio will be only 1.25. It is clear that the drift ratio ductility (or displacement ductility) is defined by the yield state and ultimate state of the section. This method still needs discussion in details. So for a complex structure system, the global displacement ductility capacity is difficult to define, the relationship between this ductility and the curvature ductility is difficult to define as well. So the section curvature ductility is more appropriate to be the measure of global ductility than the displacement ductility, because it has more reliability than the latter. On the other hand, the curvature ductility may be more useful for seismic design.

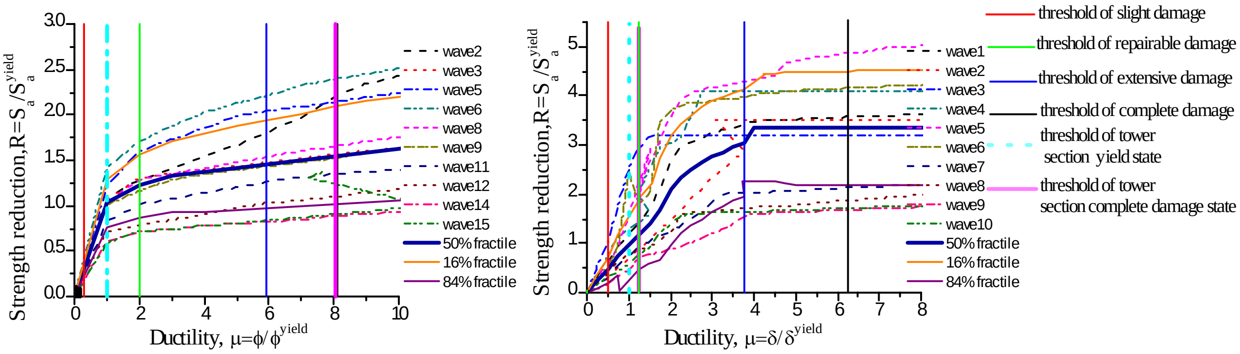

Fig. 5.4 R-μ relationship of the Maximum drift ratio and Maximum curvature of the tower

6. CONCLUSIONS

From all the IDA analyses of the reinforced concrete tower of a cable-stayed bridge in above, nonlinear seismic demand and capacity are investigated, some conclusions can be drew as follows:

1. Under longitudinal ground motion excitation, the damages of the tower mainly concentrate on three regions: the two pylon legs' bottoms (Section C), the two pylon legs' tops (Section G), the middle zones of the two pylon legs' upper parts (Section F). The maximum displacement will occurred in two critical positions, the intersection point of the two pylons (Point A) and the top of the tower (Point B).
2. As the increasing of the seismic intensity, either for curvature or for displacement of the critical point, the dispersion grows larger. At the time of maximum curvature occurs in the tower, the plastic zone mainly concentrates on sectionC, section F, sectionG. The displacements of PointA and PointB also contain some plastic performance of the tower, the slope of which is gentler than that of the curvature.
3. The IDA method is a precise tool for seismic damage state evaluation which has more probabilistic characteristic. From the point view of the section damage of the tower, when Sa is below 0.003g, the tower will keep in no damage state at the probability of 50%, below 0.0119g the tower will keep in slight damage state at a probability of 50%, above 0.0144g the tower will go into the extensive damage state, the tower will collapse when Sa is over 0.0155 at a probability of 50%. The same intensity measures for different damage states of the drift ratios of the tower are 0.005g, 0.0118g, 0.03g, 0.035g at the same order.
4. From the view of ductility, the section curvature ductility is more appropriate to be the measure of global ductility than the displacement ductility, because it has more reliability than the latter, especially for such complex structure as the cable-stayed bridge tower. For the tower studied herein, the maximum available ductility factor reaches 8, the strength reduction factor of the tower is 1.5.

Acknowledgements

This research is supported by Science & Technology Program Foundation For Transportation Development In West China under Grant No. 200631882225.

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