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Analysis of rigid frame bridges with different high piers' dynamic behavior and seismic fragility

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Abstract. In order to study the seismic performance of different high pier bridges, the (106+200+106) m rigid frame bridge is taken as the engineering background. According to the RC pier of this bridge and the principle of same bearing capacity and pier-top's stiffness, two new types of high piers are optimal designed: one is CFS-1 pier, the other is CFS-2 pier. A finite element model is established by OpenSees to analyse seismic responses of three bridges with Incremental Dynamic Analysis method. IDA curves are drawn and the damage exceedance probability is calculated under different damage conditions. The dynamic response laws and vulnerability of rigid frame bridges at different levels peak ground acceleration with different structure form's high piers are obtained. CFS-1 and CFS-2 piers' seismic performance is better than RC pier. The results of this study could serve as reference and guidance for similar engineering design.

Key words. Double limb thin-walled high pier, concrete-filled steel high pier, double corrugated steel web, Incremental Dynamic Analysis, IDA curve, damage exceedance probability.

1. Introduction

In recent years, many long-span continuous rigid frame bridges with high piers are built in high seismic intensity area. Usually, the pier section forms are rectangular hollow pier and double limb thin-walled hollow pier [1]. And thin-walled hollow pier has a lot of characteristics, for instance, its complexity in structure, high axial compression ratio at the bottom of piers, and it has complex construction, multi-processes, long construction period, difficulty of control and so on. At the same time in bridge vibration, with the increase of pier height, the weight of high pier and higher mode shape becomes more and more significant for the seismic response of the whole structure. Therefore, the seismic design and analysis theory of the high pier bridges are quite different from that of middle and low pier bridges.

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In the foundation of referring to a large number of documents, the future development trends of long-span continuous rigid frame bridge with high pier are as follows:

- The weight of superstructure will be lighter.
- The span and the total length of superstructure will continue to increase.
- Many curved bridges will be built.
- The pier will be higher and higher.
- Some new types of pier and superstructure structures will continue to appear.
- New materials and composite structures will be applied to bridge completely and the durability will be fully reflected in structural design.
- The construction will be faster and more convenient and so on.

Based on the development trend of continuous rigid frame bridge with high piers, the (106+200+106) m long-span continuous rigid frame bridge is taken as the engineering background of this paper. The pier height of the main bridge is 166.405 m and 166.205 m and the pier section form is double limb rectangular hollow pier. The layout of the bridge is shown in Fig. 1. Its single limb section size is shown in Fig. 2.

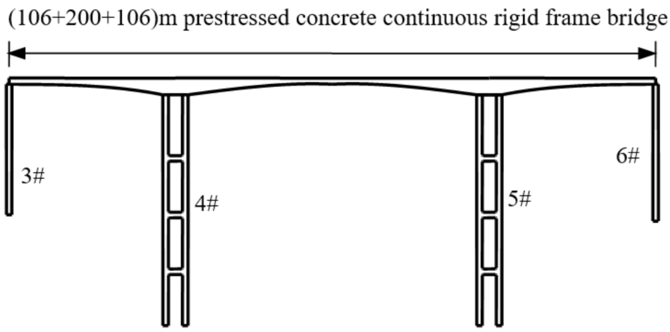

Fig. 1. Single limb section size

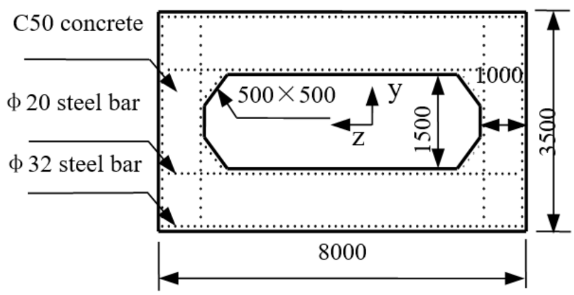

Fig. 2. Layout of main bridge (dimensions in mm)

Considering the convenience of high pier construction and advanced anti-corrosive techniques for steel members, the design life requirements can be met. According to the double limb rectangular hollow pier structure (RC pier) of this bridge and the principle of same bearing capacity and pier-top's stiffness, two new types of high piers are optimal designed in [2], as is shown in Figs. 3 and 4. Based on the research, this paper explores the dynamic characteristics and the vulnerability of different high piers under different levels of ground motion of rigid frame bridges.

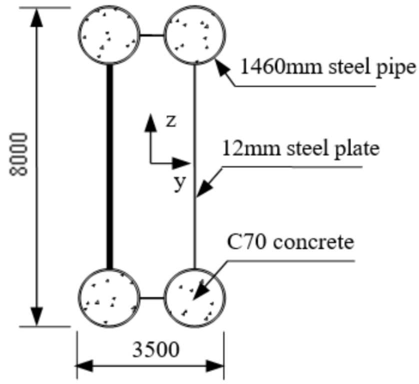

Fig. 3. CFS-1 pier section (dimensions in mm)

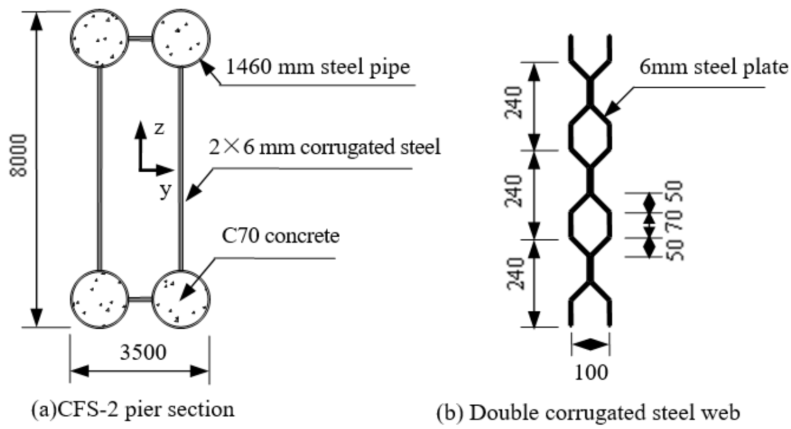

Fig. 4. CFS-2 pier section (dimensions in mm)

2. Methodology

2.1. Research on dynamic characteristics of bridges

According to double limb rectangular hollow pier (RC pier) and two new types of piers by optimal design, the dynamic analysis model of bridge is proposed. The dynamic characteristics of three continuous rigid frame bridges with different types of high piers are analyzed by multiple Ritz vector method. And the dynamic characteristics of the first 45 orders are analyzed. The cycle and mode characteristics of the first 5 orders are shown in Table 1.

Table 1. Dynamic characteristics of continuous rigid frame bridge with different high piers

Order number	RC pier		CFS-1pier		CFS-2pier	
	Cycle T/s	Characteristics of modes	Cycle T/s	Characteristics of modes	Cycle T/s	Characteristics of modes
1	5.725	Symmetrically horizontal bending of the main beam	11.620	Longitudinal floating of bridge	11.501	Longitudinal floating of bridge
2	5.042	Longitudinal floating of bridge	6.236	Symmetrically horizontal bending of the main beam	6.232	Symmetrically horizontal bending of the main beam
3	2.770	Counter-symmetrically horizontal bending of the main beam	2.480	Counter-symmetrically horizontal bending of the main beam	2.479	Counter-symmetrically horizontal bending of the main beam
4	1.420	Symmetrically horizontal bending of the main beam and high piers, the opposite direction	1.816	Symmetrically vertical bending of the main beam, inner longitudinal bending of high piers	1.804	Symmetrically vertical bending of the main beam, inner longitudinal bending of high piers
5	1.390	Symmetrically vertical bending of the main beam, inner longitudinal bending of high piers	1.636	Longitudinal bending of high pier in the same direction (4# pier)	1.620	Longitudinal bending of high pier in the same direction (4# pier)

2.2. Research on the seismic performance of bridges

By using the seismic record of PEER, three seismic waves that have similar geologic conditions and site types of the bridge site are selected for analysis [3–4]. The maximum value of three seismic waves is taken as the calculation result. Details of the seismic wave are shown in Table 2.

Table 2. Seismic waves

Year	Seismic event	Recording platform	PGA (g)
1992	Big Bear-01	Desert Hot Spring	0.225 26
1999	Chi-Chi, Taiwan	CHY025	0.159 22
1992	Landers	Desert Hot Spring	0.170 87

A finite element dynamic model is established to analyze seismic responses of bridge with Incremental Dynamic Analysis method [5–7] and the dynamic time-history curve of the whole bridge is obtained.

Due to the high pier of research object, the maximum horizontal displacement of pier may occur at pier's top, pier's middle height or pier's other positions under earthquake. The pier's position of the largest displacement is found through comparative analysis. The peak ground acceleration of seismic wave is loaded from 0.1 g step by step to 1.2 g. Three different structure forms of high piers are RC pier, CFS-1 pier and CFS-2 pier. Their displacement time-history at pier's top and pier's middle height are shown in Fig. 5.

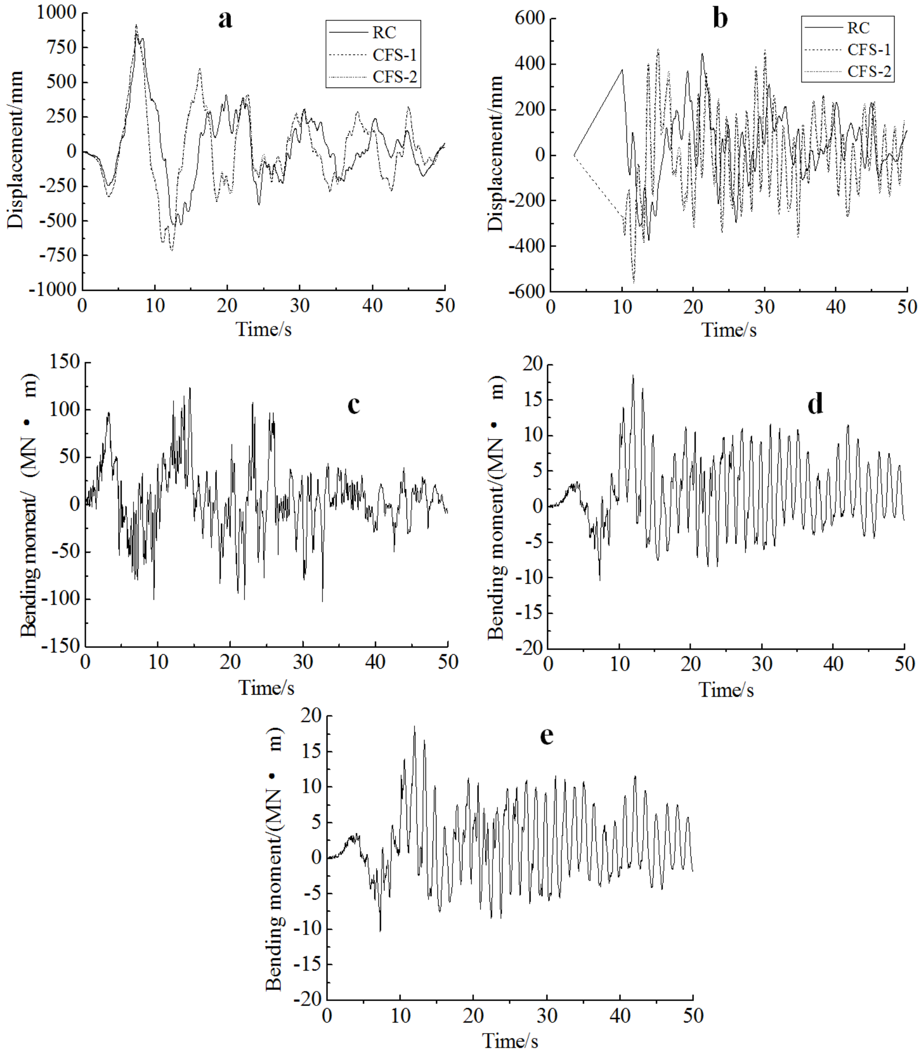

Fig. 5. Displacement time-history and bending moment time-history curves: **a**–PGA is 1.2 g at pier's top, **b**–PGA is 1.2 g at pier's middle height, **c**–PGA is 1.2 g (RC pier), **d**–PGA is 1.2 g (CFS-1 pier), **e**–PGA is 1.2 g (CFS-2 pier)

From the resultant figure, when the peak ground acceleration is small, the maximum displacement of pier's top and middle height of CFS-1 pier and CFS-2 pier is much larger than that of RC pier, while the maximum displacement of pier's top and middle height of CFS-1 pier are similar to those of CFS-2 pier. With the increase of the peak ground acceleration, the maximum displacement of pier's top and middle height increase gradually and their maximum displacements are close to the same.

In the case of the same peak ground acceleration, the moments that three types of high piers' reach maximum displacements of pier's top are different. The moments of maximum displacement of pier's middle height are also different. When the peak ground acceleration is different, the moment of reaching maximum displacement of pier's top is different for one type high pier and the time of maximum displacement of pier's middle height is also different. The displacement of pier's middle height lags behind the ground motion about a few seconds.

The peak ground acceleration of seismic wave is loaded from 0.1 g step by step to 1.2 g. The comparison of bending moment time-history of three types of piers' bottom is shown in Fig.. Three types of piers are double limb rectangular hollow pier(RC pier), concrete-filled steel tube high pier connected by steel plate (CFS-1 pier) and concrete-filled steel tube high pier connected by double corrugated steel web.

From the resultant figure it can be seen that in the case of the same peak ground acceleration, the moments when three types of high piers' reach maximum bending moment of pier's bottom are different. When the peak ground acceleration is different, the moment that maximum bending moment of pier's bottom is different for one type of high pier. At the same time, when the peak ground acceleration is same, the bending moment of pier's bottom of double limb thin-walled high pier is much larger than that of concrete-filled steel tube. And with the increase of the peak ground acceleration, the discrepancy is getting larger for their bending moment of pier's bottom.

3. Result analysis and discussion

Vulnerability is analyzed by IDA method and regression capability requirement method.

Under a given ground motion intensity, the conditional probability that the structural seismic demand D is equal to or greater than the conditional probability of seismic capacity C are as follows:

$$F_{\mathrm{r}} = P[D \geq C|IM]\,. \tag{1}$$

Earthquake intensity IM and earthquake demand D meet the relation

$$\ln D = b \cdot \ln(IM) + \ln a\,. \tag{2}$$

Logarithmic standard deviation of Structural seismic demand is

$$\beta_{D|IM} = \sqrt{\frac{\sum(\ln(d_i) - \ln(aIM_i^b))^2}{N-2}}\,. \tag{3}$$

Vulnerability function is given as

$$P[D \geq C|IM] = \Phi \cdot \frac{\ln \frac{D}{C}}{\sqrt{\beta^2_{D|IM} + \beta^2_{\mathrm{c}}}}\,. \tag{4}$$

The vulnerability function is further deduced as follows:

$$P[D \geq C|IM] = \Phi\Big(\frac{\ln(IM) - \frac{\ln C - \ln a}{b}}{\frac{\sqrt{\beta^2_{D|IM} + \beta^2_{\mathrm{c}}}}{b}}\Big) \tag{5}$$

Symbol a is the IDA curve fitting coefficient, b is the IDA curve fitting coefficient, D_i is the ith earthquake demand peak, IM_i is the ith ground motion peak, and β_c is the structural seismic capacity logarithmic standard deviation.

3.1. Damage index

Select the appropriate control section to calculate the maximum curvature distribution of the RC pier and CFS pier along the high direction of the pier (as is shown in Fig. 6) for the seismic vulnerability analysis of the two types of piers [8–11].

The XRTACT software is used to establish the RC pier and CFS pier cross-section model to analyze the bending moment curvature, and the curvature values of different damage states are summarized in Table 3.

Table 3. Damage index corresponding to the curvature of piers m^{-1}

Damage status	RC pier	CFS pier
Slight Damage	0.000651	0.001014
Secondary Damage	0.000812	0.001582
Serious Damage	0.00371	0.00429
Collapse	0.01099	0.04776

It can be seen from Fig. 7 that under the action of seismic wave, the maximum curvature of CFS pier and RC pier is at the bottom of the pier. This means that the high-order vibration mode will affect the pier of the high pier, but the bottom of the pier is most likely to reach to the plastic stage. Therefore, the pier bottom section is selected as the control section for analysis. The damage state of the pier is divided: basic intact, slight damage, secondary damage, serious damage and collapse. The damage state of the pier is shown in Table 4.

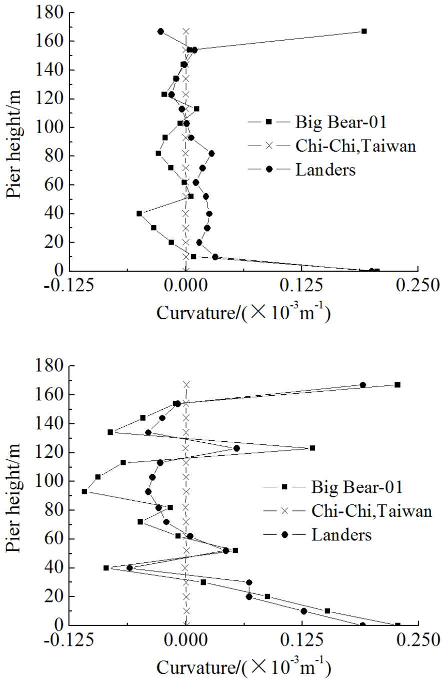

Fig. 6. Maximum curvature envelope of pier along the high direction of the pier: up–RC pier, bottom–CFS pier

Table 4. Pier damage status

Damage status	Damage description	Curvature range
Basically intact	Partial cracking of concrete	$\Phi \leq \Phi_1$
Slight damage	The lateral and longitudinal reinforcement for the first time to yield to the outer steel pipe	$\Phi_1 < \Phi \leq \Phi_y$
Secondary damage	The control section forms a plastic hinge and needs to be repaired	$\Phi_y < \Phi \leq \Phi_c$
Serious damage	Pier strength began to degenerate, difficult to repair	$\Phi_c < \Phi \leq \Phi_u$
Collapse	Core concrete crushed	$\Phi > \Phi_u$

Note: Φ_1 is the curvature of the first yielding of the inner or outer edge of the pipe; Φ_y is the equivalent yield curvature; Φ_c is the curvature at the ultimate bearing capacity; Φ_u is the ultimate curvature.

3.2. Vulnerability curve

The IDA method is used to analyze the RC pier, CFS-1 pier and CFS-2 pier. The IDA curve is drawn and fitted. The data of the regression analysis are obtained, and the damage probability of the control section is calculated. Finally, get the vulnerability curve. The comparison of the vulnerability curves of three kinds of high-pier structures, RC pier, CFS-1 pier and CFS-2 pier, is shown in Fig. 7.

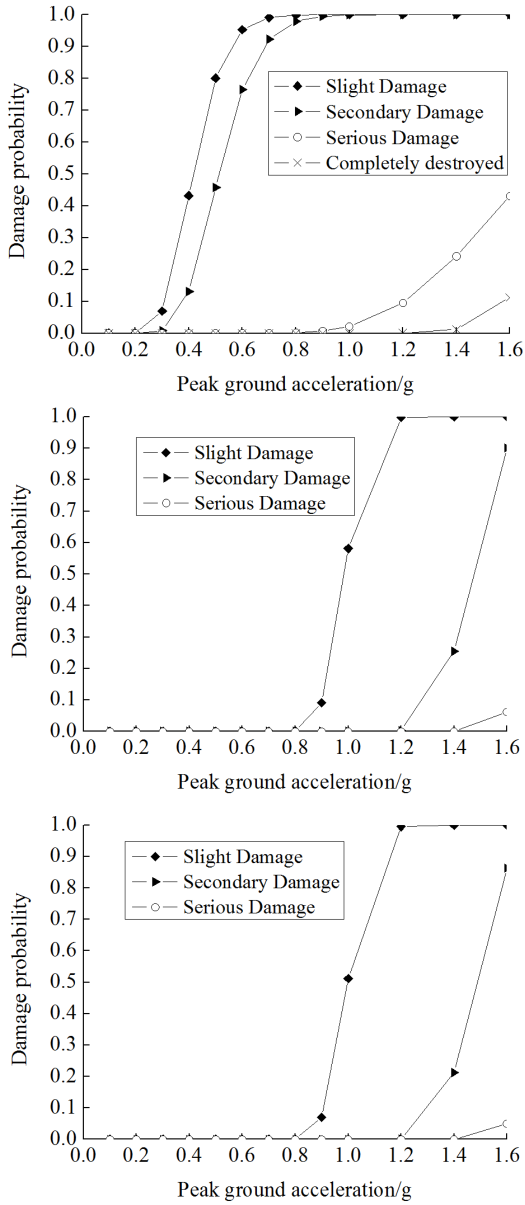

Fig. 7. Comparison of vulnerability curves of three kinds of high piers structures: up–RC pier vulnerability curves, medium–CFS-1-pier vulnerability curves, bottom–CFS-2-pier vulnerability curves

It can be seen from the comparison chart that when the ground motion is the same, the probability of slight damage of the RC pier is more than 90 % and the peak ground acceleration is 0.6 g. The peak ground acceleration of CFS-1 pier and CFS-2 pier is 1.2 g under the same damage probability. So when the peak ground acceleration value is between 0 and 1.2 g, RC-pier are more susceptible to occurred light damage. When the secondary damage probability is more than 90 %, the peak ground acceleration value of the RC pier is 0.7 g, while the peak ground acceleration value of CFS-1 pier and the CFS-2 pier is 1.6 g. So the seismic performance of CFS pier is better than the RC pier.

When the peak ground acceleration is 1.6 g, the secondary damage and serious damage probability of the CFS-2 pier are slightly smaller than those of the CFS-1 pier, so the contribution of double corrugated steel web is bigger than that of the ordinary steel plate.

4. Conclusion

Based on the analysis of dynamic characteristics, seismic response and vulnerability of three kinds of high-pier continuous rigid frame bridges, the following conclusions are given:

1. The dynamic characteristics of the first two orders of three different high-pier continuous rigid frame bridges are different,the cycle of the RC pier is much smaller than that of the CFS-1 pier and the CFS-2 pier. The natural vibration cycle of the other stages is not much difference. The first-order vibration mode of the RC pier is the symmetrically horizontal bending of the main beam, and the second-order mode is the longitudinal floating of bridge. And the first-order vibration mode of the CFS-1 pier and the CFS-2 pier is just the opposite.The seismic response of the RC piers is different from that of the CFS-1 pier and the CFS-2 piers, indicating that the seismic response of the different high-pier structures is different.

2. The value of the peak ground acceleration is loaded from 0.1 g step by step to 1.2 g. The maximum displacement of the pier top and the maximum bending moment of the pier bottom are increased. When the peak ground acceleration is the same, the maximum displacement of the pier top is different from the moment of the maximum bending moment.

3. When the ground motion is the same, although the maximum displacement of pier top of the CFS-1 pier and the CFS-2 pier is larger than that of RC pier, bending moment of the pier bottom is much smaller than the RC pier. It is indicating that the CFS-1 pier and the CFS-2 pier are more flexible than the RC pier.

4. As high piers, the flexibility of CFS-1 pier, CFS-2 pier and RC pier are better, so the probability of serious damage and collapse under the earthquake is small. However, under the same ground motion, the CFS-1 pier and CFS-2 pier are more secure than the rectangular hollow RC pier.

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