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A Study on Failure Characteristics of Double Plastic Hinge of Bridge Tower of Long-span Bridge

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Abstract: Within the engineering context of Runyang bridge, tower bridge of suspension bridge is discussed as the main research object of this paper. A number of nonlinear seismic response analyses have been carried out on long-span suspension bridge, and failure mode and failure characteristic of the bridge tower have been under research. Based on this, incremental dynamic analysis is conducted on the tower bridge of suspension bridge under the action of specific seismic oscillations. Also, the failure process under earthquake action has been studied. Researches show that, with continued excitation from sufficiently large seismic loads, the tower bridge of large-span suspension bridge is bound to form one pair of double plastic hinges at the tower bottom with the maximum bending moment as well as at the tower body with the maximum displacement. When double plastic hinges are formed and the next large seismic pulse arrives, the plastic curvature of cross section of the tower bottom and the tower body will simultaneously change. The stress distributed on the bridge tower makes a gradual shift to the tower body, until the curvature of cross section of the tower bottom exceeds the limit and suffers failure, with no more new plastic hinges generated. The five characteristic responses selected in the incremental dynamic analysis are compared. In terms of the IDA curve, the maximum displacement of bridge tower is set as the X-axis, and the grade of seismic load is chosen as the Y-axis, which effectively illustrates the failure process of bridge tower.

Keywords: *Long-span Bridge; Finite Element Model (FEM); Bridge Tower; Seismic Failure*

1. Introduction

With the rapid development of transportation construction in China, a number of long-span bridges are constructed, and suspension bridge is the main structure of long-span bridge [1]. The horizontal of suspension bridge pylon adopts frame structure that is a high order statically indeterminate structural system, and plastic hinges sequentially form a mechanism[2]. The longitudinal of suspension bridge pylon resembles the single column with elastic restraint at the top. Once the bridge tower fails, it means that the whole suspension bridge collapses [3]. Therefore, the failure of tower column at the longitudinal under earthquake action might be control mode.

In this paper, within the engineering context of Runyang bridge, a full-bridge model is established for the Runyang suspension bridge, and the failure characteristics of double plastic hinges of long-span suspension bridge are under study. With the help of IDA method [4-5], increment dynamic analysis has been conducted on the bridge tower of suspension bridge with specific seismic excitations, and the dynamic characteristics responses of suspension bridge tower are further investigated.

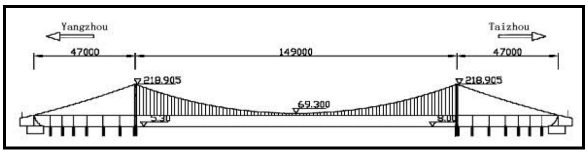
2. The dynamic calculation model of the long-span bridge

2.1. The overview of long-span bridge

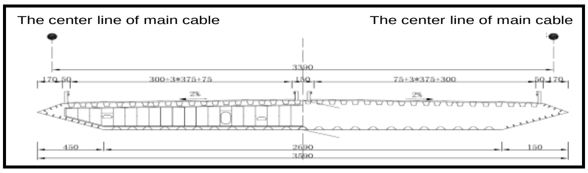
Runyang Yangtze River Highway Bridge is a large-scale bridge project that connects Zhenjiang and Yangzhou and crosses the Yangtze River. The long-span suspension bridge located at the south branch is referred to as Runyang suspension bridge. The long-span cable-stayed bridge lies at the north branch, and the approach bridge situated between the north branch and the south branch is a continuous beam bridge.

Runyang suspension bridge adopts a simply supported single span system, with a main span of 1490m. Both ends of the main girder adopt the sliding bearing support on the bottom end rail of the main tower. Main girder applies a closed streamlined flat steel box girder, with a width of 33.9m and a beam depth of 3m at the centerline. The lateral space between the two main cables is 33.9m, consisting of parallel high-strength galvanized steel cables. The area of each main cable is 0.4735m2, and the longitudinal space of the suspender connecting the main cable and the main girder is 16m.

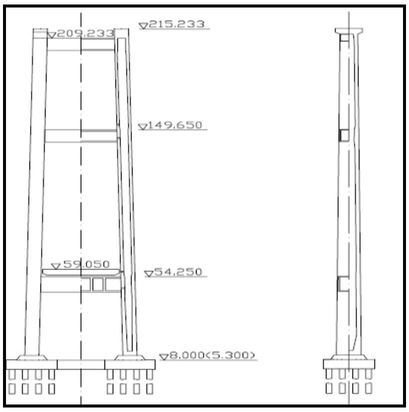
Tower body employs the portal frame with concrete structure. The distance between two tower columns at the tower top is 34.3m, and it is 41.34m at the tower bottom, with linear changes. The south tower is 207.23m high, and the north tower is 209.93m high (excluding the height of saddle). There are upper, middle and lower beams in each tower. The structure model diagram of Runyang suspension bridge is shown in Figure 1.



(a) Vertical View



(b) Standard Cross-section Diagram of Steel Box Girder



(c) The Elevation and Side Schematic Diagram of Bridge Tower of Runyang Suspension Bridge

Figure 1.*Structure Model of Runyang Suspension Bridge (Unit: cm; Elevation: m)*

2.2. The dynamic calculation model of Runyang suspension bridge

In this paper, a large-scale general-purpose finite element software—ADINA has been adopted to establish a computational model for Runyang suspension bridge and to conduct dynamic analysis. In the computational model, the bending moment-curvature beam element is used to simulate the tower column, with the consideration of nonlinearity and large displacement effects of the materials [6]; elastic beam element is used to simulate the main girder and the cross girder of bridge tower; three-dimensional TRUSS rod element is applied to simulate the main cable and the suspender of the suspension bridge and Ernst formula is employed to revise the modulus, taking sag[7] into account; the main girder adopts the backbone model, using the principal-subordinate relationship to simulate the connection between the main girder and the suspender, as well as the connection between the main cable and the tower top; solidification is used between the bridge tower and the ground, as well as the main cable and the ground. Model boundary and connection conditions are illustrated in Table 1, and the dynamic calculation is shown in Figure 2.

Table 1: *Boundary and connection condition*

Degree of Freedom / Location	x	y	z	θ_x	θ_y	θ_z
King tower and the ground	1	1	1	1	1	1
Main girder and main tower	0	Principal-subordinate	Principal-subordinate	Principal-subordinate	0	0
Main cable and tower top	Principal-subordinate	Principal-subordinate	Principal-subordinate	Principal-subordinate	Principal-subordinate	Principal-subordinate
Main cable and the ground	1	1	1	1	1	1

Note: In this table, x stands for the longitudinal direction, and y is the horizontal direction, and z is vertical direction. "0" is freedom, and "1" means solidification.

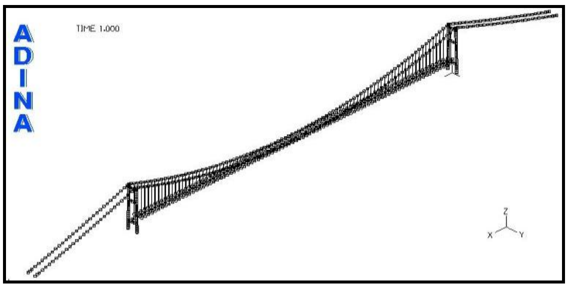


Figure 2.*Finite Element Model*

3. Failure characteristics of double plastic hinge of the bridge tower of suspension bridge under seismic excitation

Because the cross section of long-span suspension bridge increases from top to bottom, the bending capabilities of cross section at the bottom tower and at the middle cross beam are different. Therefore, the cross section of bottom tower might enter the plasticity stage first. But with the persistent excitation of seismic loads, stiffness at the bottom degrades and the force distribution transfers. When the next large seismic pulse arrives, the spot with maximum force transfers to where the stiffness is maximum. Then a plastic hinge forms in the tower. Thus, it is necessary to study the failure mode of double plastic hinge of long-span suspension bridge.

3.1. Inputs of seismic wave

The inputs of seismic oscillation are based on the structural earthquake-resistant design, so the selection of appropriate inputs is the first crucial step in this design. Seismic oscillation is featured by intense randomness, with uncertain occurrence time, space, intensity, spectral components, and waveforms [8]. Time-history analysis is a deterministic analytic process, whose results largely depend on the time histories of selected seismic waves. Seismic response of different time histories might vary substantially. If the input of seismic oscillation z is unreasonable, its conclusions or results would be meaningless. Therefore, this paper selects the actual record of strong earthquakes as the inputs of earthquake excitation, and observes the universal failure characteristics of double plastic hinge of bridge tower of Runyang suspension bridge. Four typical seismic waves on the four types of fields in the documents [9] are selected (as shown in Table 2), where each wave from Class I ~ IV field is chosen, and seismic waves are inputted along the longitudinal direction of the bridge.

***Table 2:** Selected record chart of seismic wave*

Field Class	No.	Record name of seismic wave
I	F2-2	1994, Los AngelesGriffith Observation, Northridge ,360
II	F5-1	1952,Taft, Kern County,N21E
III	F7-2	1940, El Centro-lmp Vall lrr Dist, El Centro,270
IV	F10-1	1984, Parkfield Fault Zone 14, Coalinga,CA,0

Note: The actual record names of seismic waves consist of four parts: the first is seismic time; the second is the name of the station; the third is the name of the earthquake; and the fourth part is the weight of the record.

3.2. Analysis on failure mode of single tower with uniform cross section under seismic excitations

Long-span suspension bridge tower is similar to single column constrained at the top in the longitudinal, and the stiffness system is a series system. Therefore, with the longitudinal seismic wave inputs, bridge tower is reflected as the occurrence and the development of two plastic hinge in partners. To observe the development of plastic hinge, seismic waves are shown in Table 2 in this section, and seismic excitation analysis is carried out on the full-bridge model of Runyang suspension bridge.

To observe the injury characteristics of long-span suspension bridge tower, the paper uses the uniform degree of plasticity development as the benchmark; in other words, the potential plastic hinges of cross sections at the tower bottom and at the tower body generate plastic hinge at the same time. The plastic curvature time-history curve of bridge tower is drawn to observe the plastic development of bridge tower. When the double plastic hinge occurs at the bridge tower for the first time, the corresponding amplitude levels of seismic oscillation (with 0.1g increments) and the maximum displacement are illustrated in Table 3.

***Table 3:** Time difference, the corresponding seismic peak values and the maximum displacement at the first time double plastic hinge occurs at the bridge tower*

Seismic wave	Seismic peak (g)	Maximum displacement (m)	Types of plastic development
F2-2	0.6	-1.02E+00	Type1
F5-1	1.5	1.06E+00	Type1
F7-2	0.5	9.46E-01	Type2
F10-1	0.8	-8.52E-01	Type3

At the first time the double plastic hinge occurs at the bridge tower, the dynamic responses of key sections and plastic hinge sections at the tower bottom and at the tower body are shown in Table 4, where the curvature ductility factor is the ratio of the maximum curvature and the yield curvature.

***Table 4:** Dynamic response of key cross section the first time double plastic hinge occurs at the bridge tower*

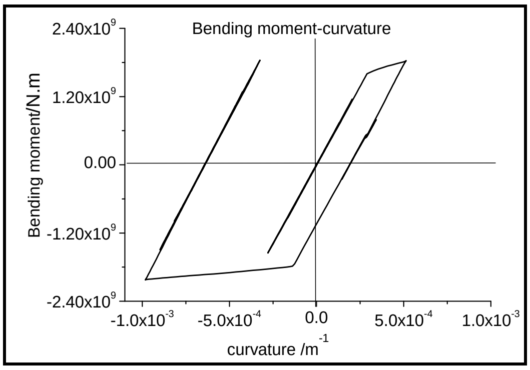
Seismic wave	Maximum curvature (m^{-1})		Maximum plastic curvature (m^{-1})		Curvature ductility	
	Tower bottom	Tower center	Tower bottom	Tower center	Tower bottom	Tower center
F2-2	-9.82E-04	4.01E-04	-6.30E-04	3.22E-05	2.79	1.09
F5-1	1.90E-03	-3.76E-04	1.55E-03	-1.24E-05	5.39	1.03
F7-2	4.65E-04	-4.14E-04	1.45E-04	-3.19E-05	1.45	1.08
F10-1	-1.23E-03	4.67E-04	-8.78E-04	5.23E-05	3.52	1.13

As indicated by the dynamic response of Runyang suspension bridge in the case of gradually increasing seismic loads, the findings are as follows. When the grade of seismic wave is low, the dynamic response of bridge tower is linear elastic; when the seismic load increases, slight plasticity occurs at the tower bottom; when the seismic load continues to increase, three different development patterns appear in the plastic development of bridge tower, as shown in table 4. In case of type 1, plastic hinge first forms at tower bottom, then plasticity occurs in the tower center along with the development of plastic hinges at tower bottom. Under the condition of type 2, the plastic hinge generates at the tower bottom and at the tower center simultaneously, and they develop together. In case of type 3, plastic hinge first forms at the tower center, then plasticity occurs in the tower bottom with unsimultaneous development of plastic hinge at the tower center.

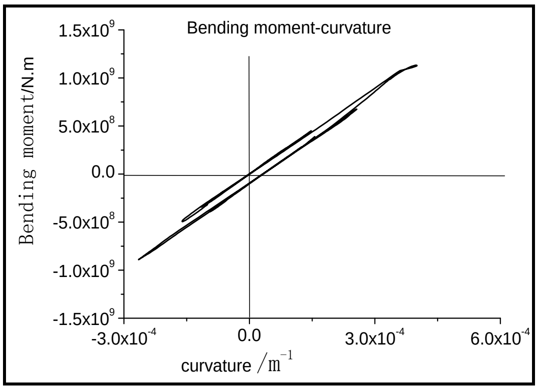
In this section, two types of typical seismic oscillation—F2-2 and F7-2 are selected. Under the seismic excitation of these two representatives oscillations, detailed data of dynamic characteristics at the north tower are listed for elaboration.

3.2.1 Analysis on dynamic characteristics of bridge tower under the excitation of seismic wave F2-2 with amplitude of 0.6g

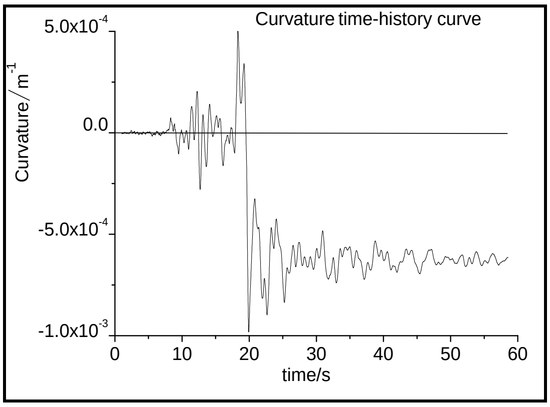
Under the excitation of seismic wave F2-2 with an amplitude of 0.6g, bending moment- curvature curves at the maximum displacement of the tower bottom and the tower body are shown in Figure 3 (a) and Figure 3 (b); curvature time-history curves are illustrated in Figure 3 (c) and Figure 3 (d); plastic curvature time-history curves are demonstrated in Figure 3 (e) and Figure 3 (f); and displacement time-history curves of the tower center are shown in Figure 3 (g).



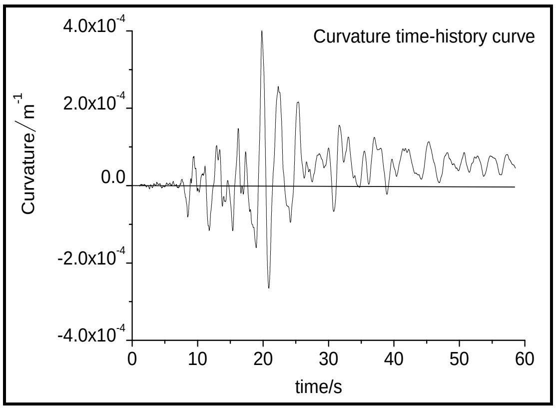
(a) Bending moment-curvature curve at tower bottom



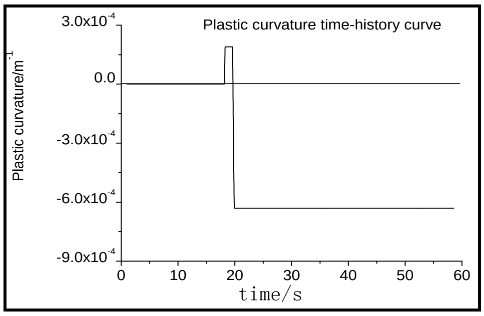
(b) Bending moment-curvature curve at the maximum displacement of tower body



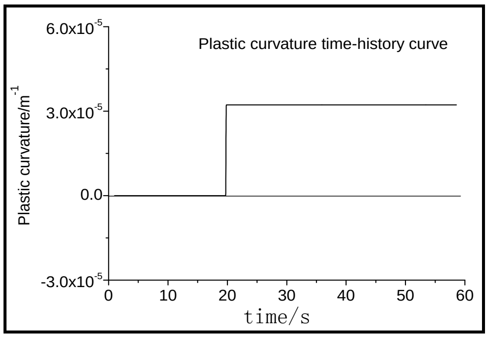
(c) Curvature time-history curve at tower bottom



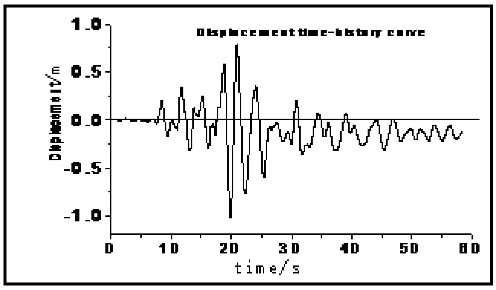
(d) Curvature time-history curve at the maximum displacement of tower body



(e) Plastic curvature time-history curve at tower bottom



(f) Plastic curvature time-history curve at the maximum displacement of tower body



(g) Displacement time-history curve at the maximum displacement

Figure 3.*Response curves of bridge tower under the excitation of seismic wave F2-2*

According to the seismic response of bridge tower under the excitation of seismic wave F2-2 with an amplitude of 0.6g:

(1) As indicated in Figure 3 (a) and Figure 3 (b), both the bottom of bridge tower and the single tower at the largest displacement are in the plastic state. The maximum curvature at tower bottom is $-9.82\times10^{-4}m^{-1}$, and curvature ductility is 2.79. The maximum curvature at tower body is $4.01\times10^{-4}m^{-1}$, and curvature ductility is 1.09.

(2) According to Figure 3 (c) and Figure 3 (d), because the curvature of tower bottom and that of the tower body with the maximum displacement yield, after both curvatures yield, the oscillation vibrates from the initial equilibrium position.

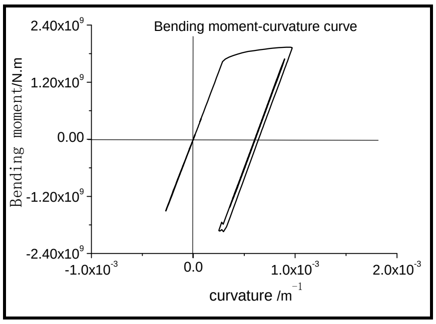
(3) As shown in Figure 3 (e) and Figure 3 (f), plasticity occurs at 18.20s at the tower bottom, with a curvature of $3.04\times10^{-4}m^{-1}$, and the corresponding curvature of the tower body is $-8.65\times10^{-5}m^{-1}$. Plasticity appears at 19.73s at the maximum

displacement of the tower body, with a curvature of $3.60\times10^{-4}m^{-1}$. At this time, with the plastic development at the tower bottom, plastic curvatures at the tower bottom and at the tower center make synchronized changes.

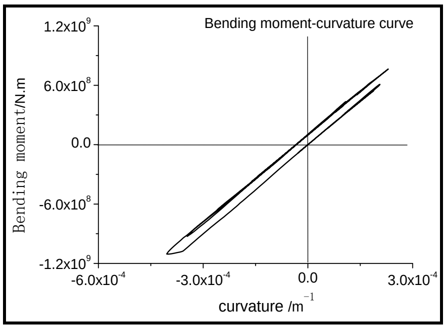
(4) Because the response of bridge tower under the excitation of seismic wave is relatively small, the horizontal displacement of tower body merely deviates.

3.2.2. Analysis on dynamic characteristics of bridge tower under the excitation of seismic wave F2-2 with amplitude of 0.5g

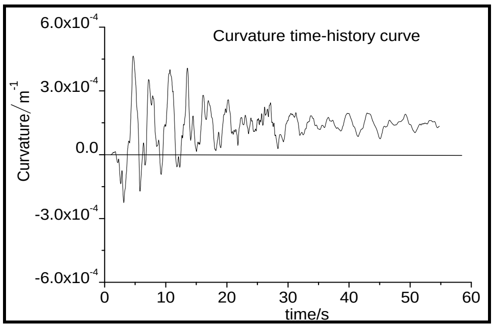
Under the excitation of seismic wave F7-2 with an amplitude of 0.5g, bending moment-curvature curves at the tower bottom and at the maximum displacement of tower body are shown in Figure 4 (a) and Figure 4 (b); curvature time-history curves are illustrated in Figure 4 (c) and Figure 4 (d); plastic curvature time-history curves are demonstrated in Figure 4 (e) and Figure 4 (f); and displacement time-history curves of the tower center are shown in Figure 4 (g).



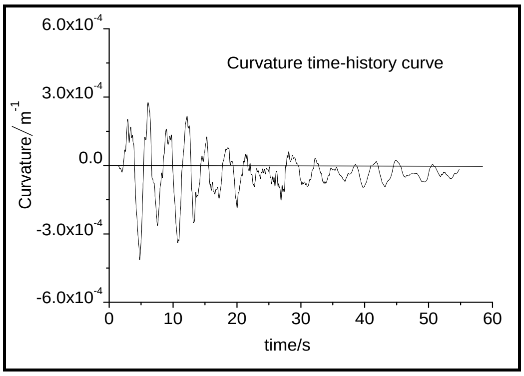
(a) Bending moment-curvature curve at tower bottom



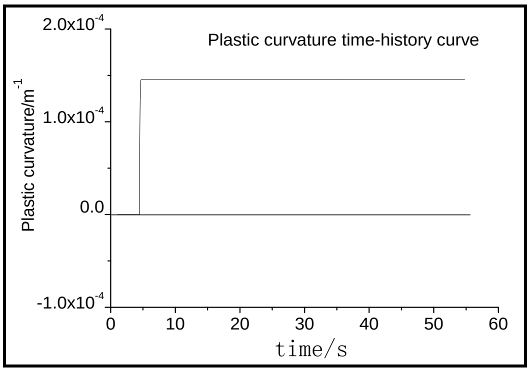
(b) Bending moment-curvature curve at the maximum displacement of tower body



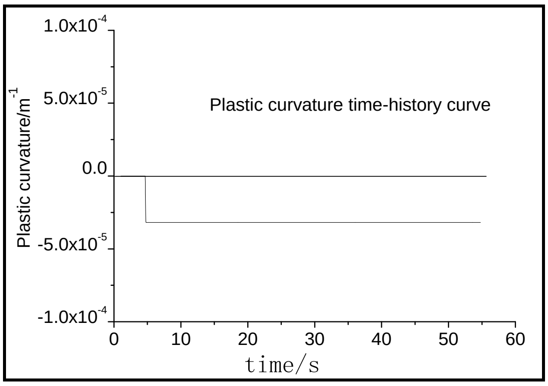
(c) Curvature time-history curve at tower bottom



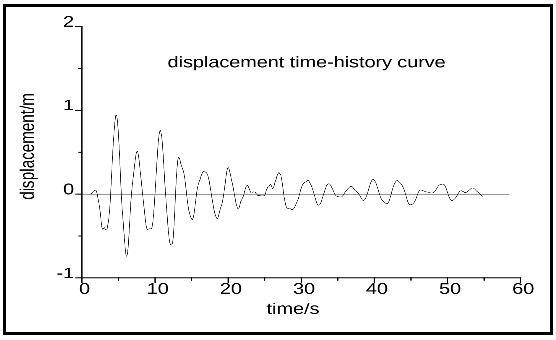
(d) Curvature time-history curve at the maximum displacement of tower body



(e) Plastic curvature time-history curve at tower bottom



(f) Plastic curvature time-history curve at the maximum displacement of tower body



(g) Displacement time-history curve at the maximum displacement

Figure 4.*Response curve of bridge tower under the excitation of seismic wave F7-2*

According to the seismic response of tower bridge under the force of seismic wave F7-2 with an amplitude of 0.5g:

(1) As indicated in Figure 2.2 (a) and Figure 2.2 (b), both the bottom of bridge tower and the single tower at the largest displacement are in the plastic state. The maximum curvature at tower bottom is $4.65\times10^{-4}m^{-1}$, and curvature ductility is 1.45. The maximum curvature at tower body is $4.14\times10^{-4}m^{-1}$, and curvature ductility is 1.08.

(2) According to Figure 2.2 (c) and Figure 2.2 (d), because the curvature of tower bottom and that of the tower body with the maximum displacement yield, after both curvatures yield, the oscillation vibrates from the initial equilibrium position.

(3) As shown in Figure 2.2 (e) and Figure 2.2 (f), plasticity occurs at 4.46s at the tower bottom, with a curvature of $3.09\times10^{-4}m^{-1}$, and the corresponding curvature of the tower body is $-2.89\times10^{-4}m^{-1}$. Plasticity appears at 4.68s at the maximum displacement of the tower body, with a curvature of $-3.66\times10^{-4}m^{-1}$. In other words, plastic curvatures at the tower bottom and at the tower body simultaneously change.

(4) Because the response of bridge tower under the excitation of seismic wave is relatively small, the horizontal displacement of tower body merely deviates.

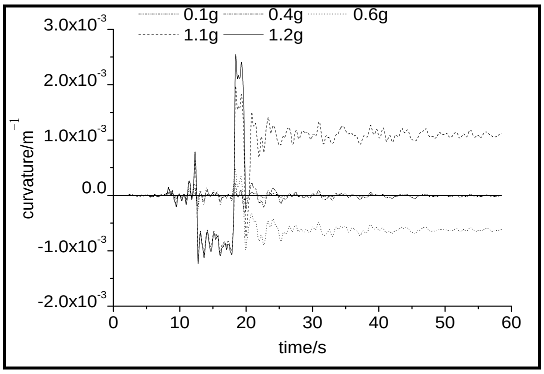
4. Incremental dynamic analysis

Incremental dynamic analysis (IDA) is a dynamic parameter analytical method of structural performance under the impact of seismic oscillation, which is calculated by the accelerometer of seismic oscillation multiplied by a series of accelerometer adjustment factor respectively, forming a set of seismic oscillations with different intensities. Under the function of this group of seismic loads, nonlinear dynamic time-history analysis is conducted respectively. With the help of IDA curve that studies structure performance parameters and accelerometer adjustment factors, the whole failure process of the structure with seismic load effects is under study.

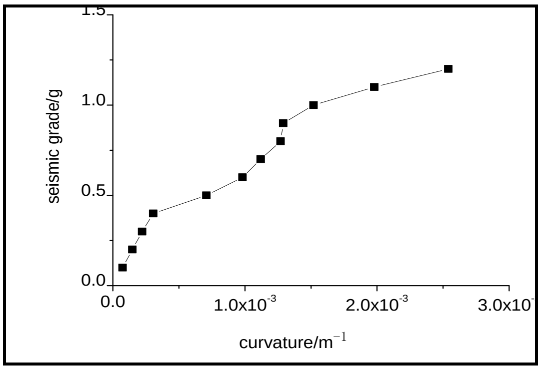
This section selects and studies the process of how the typical seismic oscillation F2-2 fails the bridge tower. The step length of the seismic oscillation amplitude is set as 0.1g. Table 3.1 illustrates the dynamic response characteristics value of potential plastic hinge cross-section under the excitation of a group of F2-2 seismic oscillation with different intensities. The characteristics responses of Runyang suspension bridge tower are shown in Figure 5 to Figure 9. Figure 5 compares the curvature time-history curves at the tower bottom and at the tower center under the seismic oscillations with different intensities. Figure 11 compares the plastic curvature time-history curves at the tower bottom and at the tower center under the seismic oscillation with different intensities. Due to the large number of seismic oscillations with different intensities, it is impossible to compare all the intensities in one chart. Therefore, some representative intensities are selected, such as 0.1g (elasticity), 0.4g (plasticity occurs at the tower bottom), 0.6g (plasticity occurs both at the tower bottom and at the tower center), 1.1g (plasticity occurs both at the tower bottom and at the tower center) and 1.2g (the section at the tower bottom fails).

Table 5: *Dynamic response characteristics value of potential plastic hinge cross-section under the excitation of seismic oscillation F2-2*

The grade of seismic wave (g)	Extreme values of tower body (at the maximum displacement of bridge tower)			Extreme values at tower bottom		Status of bridge tower
	Displacement (m)	Curvature (m^{-1})	Plastic curvature (m^{-1})	Curvature (m^{-1})	Plastic curvature (m^{-1})	
0.1	-1.73E-01	6.69E-05	0	-7.36E-05	0	plastic
0.2	-3.47E-01	1.34E-04	0	-1.48E-04	0	plastic
0.3	-5.21E-01	2.01E-04	0	-2.22E-04	0	plastic
0.4	-6.97E-01	2.69E-04	0	-3.06E-04	-1.14E-05	single plastic hinge
0.5	-8.72E-01	3.30E-04	0	-7.08E-04	-3.74E-04	single plastic hinge
0.6	-1.02E+00	4.01E-04	3.22E-05	-9.82E-04	-6.30E-04	double plastic hinge
0.7	-1.09E+00	4.89E-04	1.02E-04	-1.12E-03	-7.58E-04	double plastic hinge
0.8	-1.14E+00	6.12E-04	2.13E-04	-1.27E-03	-9.33E-04	double plastic hinge
0.9	-1.18E+00	7.40E-04	3.35E-04	-1.29E-03	-9.56E-04	double plastic hinge
1.0	-1.16E+00	8.19E-04	4.12E-04	1.52E-03	1.18E-03	double plastic hinge
1.1	1.15E+00	8.57E-04	4.51E-04	1.98E-03	1.64E-03	double plastic hinge
1.2	1.40E+00	8.82E-04	4.77E-04	2.54E-03	2.21E-03	Cross-section of bridge bottom fails

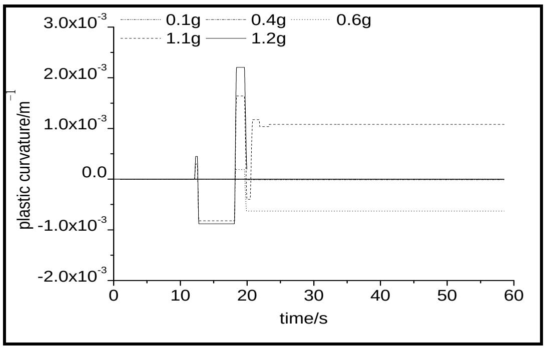


(a) Curvature time-history curve at tower bottom

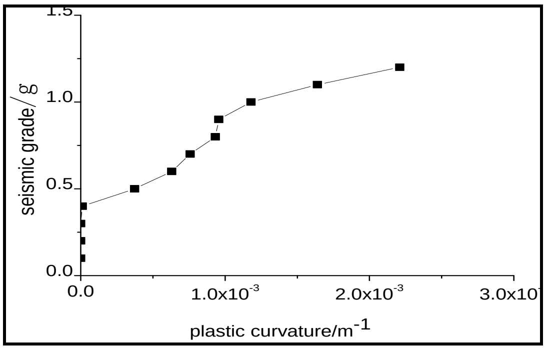


(b) Maximum curvature IDA curve at tower bottom

Figure 5.*Curvature response of the tower bottom under the excitation of seismic wave F2-2*

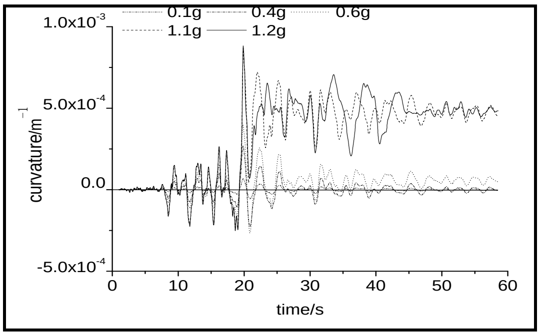


(a) Plastic curvature time-history curve at tower bottom

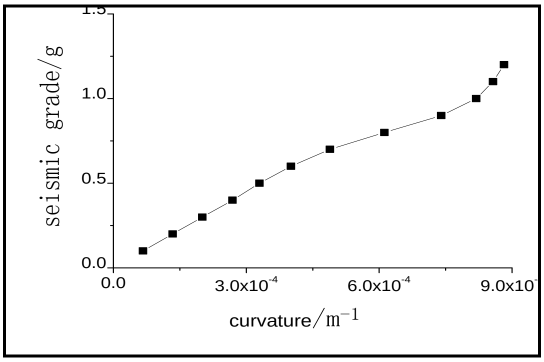


(b) Maximum plastic curvature IDA curve at tower bottom

Figure 6.*Plastic curvature response of the tower bottom under the excitation of seismic wave F2-2*

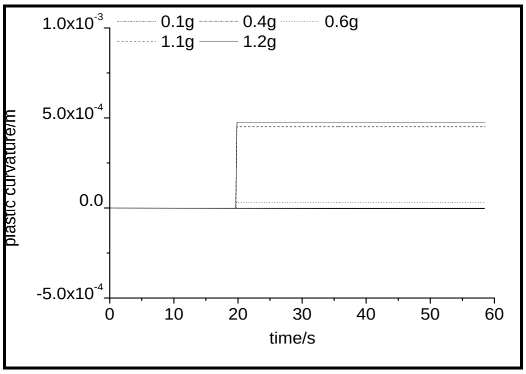


(a) Curvature time-history curve at tower body

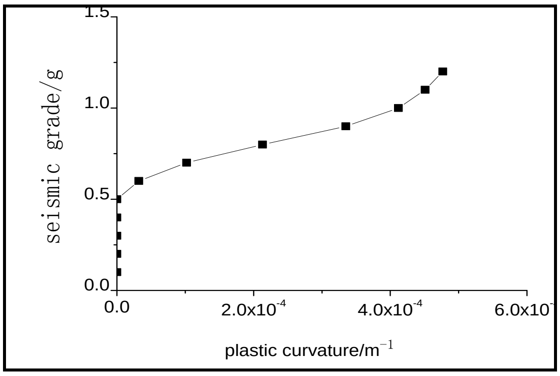


(b) Maximum curvature IDA curve at tower body

Figure 7.*Curvature response of the tower center under the excitation of seismic wave F2-2*

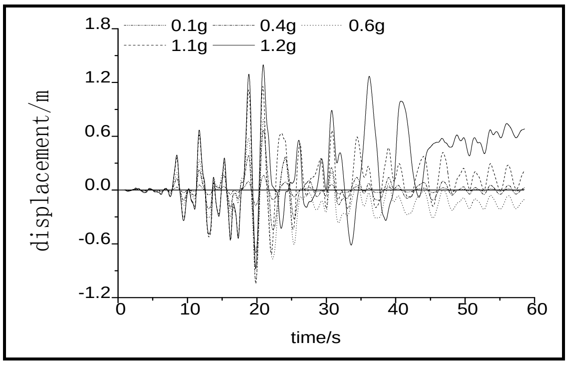


(a) Plastic curvature time-history curve at tower body

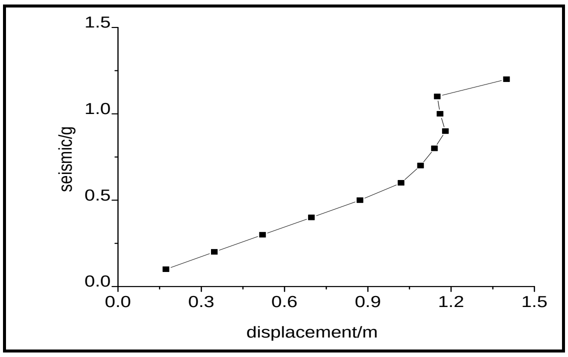


(b) Maximum plastic curvature IDA curve at tower body

Figure 8.*Plastic curvature response of the tower center under the excitation of seismic wave F2-2*

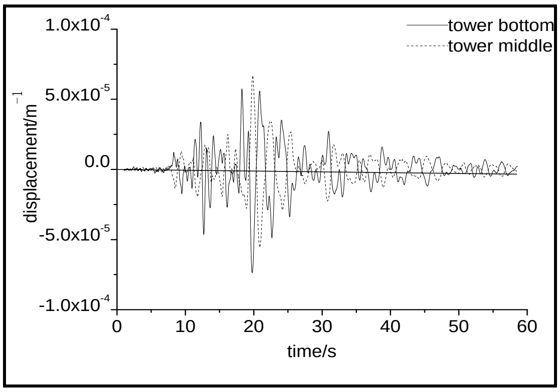


(a) Displacement time-history curve at tower body

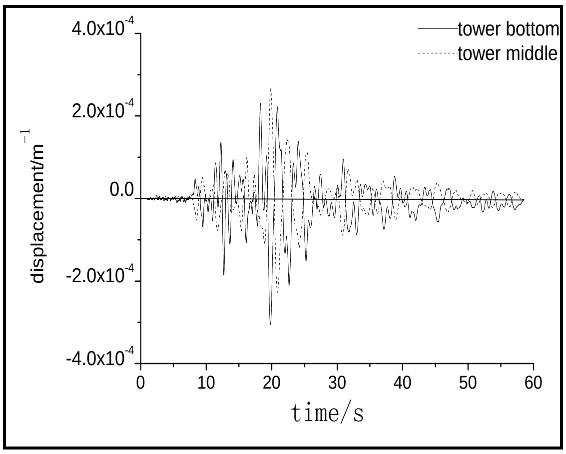


(b) Maximum displacement IDA curve at tower body

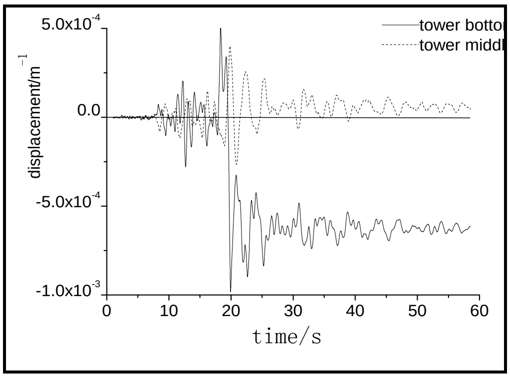
Figure 9.*Displacement response of the tower center under the excitation of seismic wave F2-2*



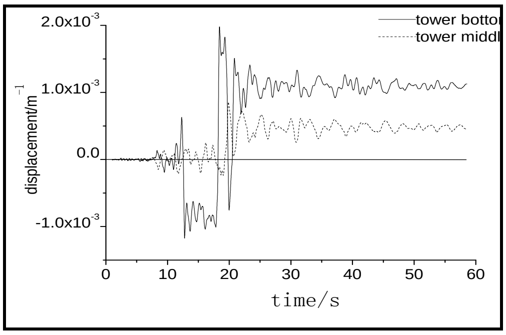
(a) The excitation of 0.1g seismic wave F2-2



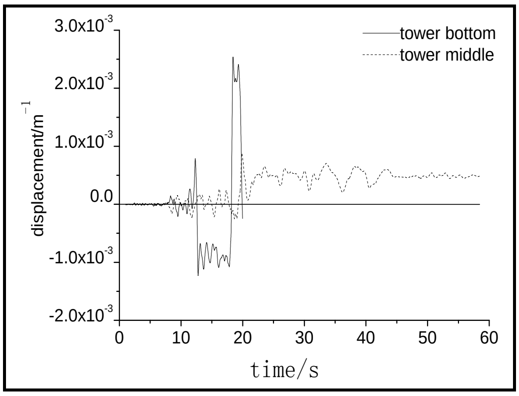
(b) The excitation of 0.4g seismic wave F2-2



(c) The excitation of 0.6g seismic wave F2-2

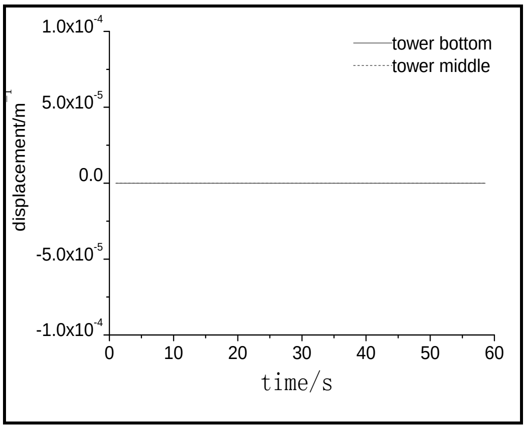


(d) The excitation of 1.1g seismic wave F2-2

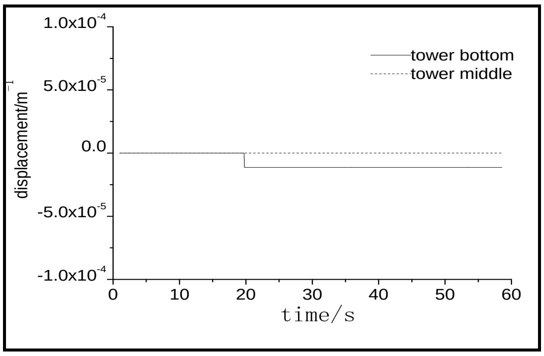


(e) The excitation of 1.2g seismic wave F2-2

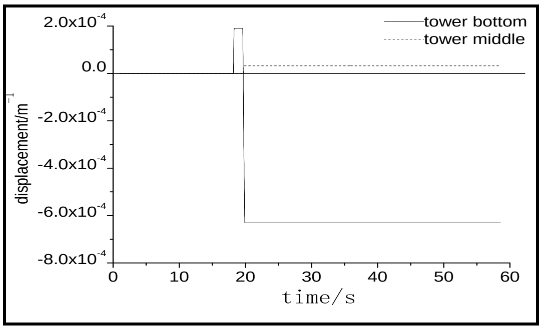
Figure 10.*The comparison of curvature time-history curve at the tower bottom and the tower center under the excitation of seismic wave F2-2*



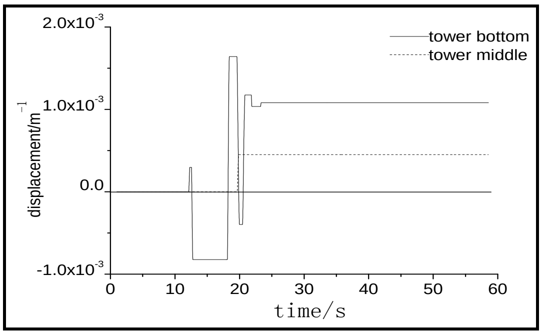
(a) The excitation of 0.1g seismic wave F2-2



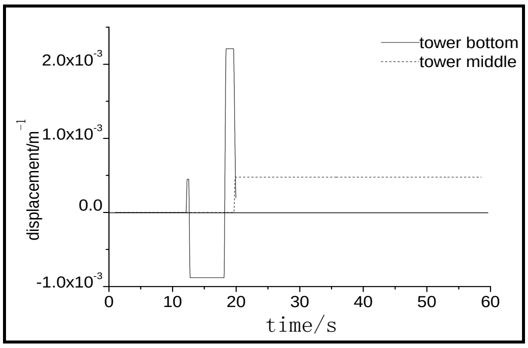
(b) The excitation of 0.4g seismic wave F2-2



(c) The excitation of 0.6g seismic wave F2-2



(d) The excitation of 1.1g seismic wave F2-2



(e) The excitation of 1.2g seismic wave F2-2

Figure 11.*The comparison of plastic curvature time-history curve at the tower bottom and the tower center under the excitation of seismic wave F2-2*

According to the seismic response of bridge tower under the excitation of seismic wave F2-2 with different intensities:

(1) When the curvature of the tower bottom section exceeds the curvature limit, namely when the section of tower bottom fails, an obvious turning occurs at the IDA curve whose x-coordinate is the maximum displacement of the bridge tower, as shown in Figure 9. Thus, the failure of long-span suspension bridge tower is caused by the loss of carrying capacity of the tower bottom section.

(2) Figure 5 to Figure 9 compare the selected five characteristic responses, namely the curvature at the tower bottom, the plastic curvature at the tower bottom, the curvature at the tower center, the plastic curvature at the tower center and the maximum displacement of the bridge tower, as well as the IDA curve drawn with characteristic response as the x-coordinate. As seismic oscillation amplitudes increase, the IDA curve slope of curvature at the tower bottom and plastic curvature shows the trend of decrease, while that of curvature at the tower center and plastic curvature tends to increase. Therefore, with the failure of the tower bottom section, the distributed force of the bridge tower has gradually transferred to the tower body. And when the curvature of bottom section exceeds the curvature limit, there is an evident turning point on the IDA curve at the maximum displacement, characterizing the failure of bridge tower.

(3) According to Figure 10, with the increase of seismic oscillation amplitudes, after the cross sections of the tower bottom and the tower center yield, vibration deviates from its equilibrium position farther and farther.

(4) As shown in Figure 11, with the persistent excitation of seismic oscillation and the increase of seismic oscillation amplitudes, the cross-sectional plastic curvature at the tower bottom and at the tower center almost always change at the same time.

5. Conclusions

Within the engineering context of Runyang suspension bridge, a full-bridge model of Runyang suspension bridge is established. A number of seismic analysis has been carried out, and the main conclusions are as follows:

(1) With persistent excitation of sufficiently large seismic loads, the tower bridge of large-span suspension bridge is bound to form one pair of double plastic hinge at the tower bottom with the maximum bending moment as well as at the tower body with maximum displacement, which is the structural characteristic of bridge tower, and is determined by the single column constrained by the top in the longitudinal.

(2) According to the time two potential plastic hinge cross sections of Runyang bridge tower enter plasticity, in most cases, the section of tower body at the largest displacement slightly lags behind the bottom section, or cross sections at the tower bottom and at the tower body arrive at plasticity simultaneously. This is mainly due to the yield moment at the tower bottom is greater than that at the tower body. However, the bending moment at the tower bottom is larger than that at the tower body. Therefore, there is no significant time sequence concerning the time cross sections at the tower bottom and at the tower body reach plasticity.

(3) In most cases, after double plastic hinge of the suspension bridge tower is formed, and when the next large seismic pulse arrives, with persistent excitation of seismic loads, the plastic curvatures of the cross section at the tower bottom and at the tower body would simultaneously change. The distributed stress on the bridge tower makes a gradual shift to the tower body, until the curvature of the bottom section exceeds the curvature limit and fails, with no more new plastic hinges generated.

(4) Five characteristic responses are compared in the incremental dynamic analysis, namely the curvature at the tower bottom, the plastic curvature at the tower bottom, the curvature at the tower body, the plastic curvature at the tower body and the maximum displacement of the bridge tower. IDA curve uses the maximum displacement of the bridge tower as x-coordinate, and the grades of seismic loads as the y-coordinate, which effectively reflects the failure process of the bridge tower.

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