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# Shake Table Studies of Seismic Damage and Failure Mode of a Single Tower Cable-stayed Bridge

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## *Abstract:*

To investigate seismic damage and failure mode of a single pylon cable-stayed bridge, Puqian bridge with 'A" shape concrete pylon is selected and a 1:20 scale model is designed, constructed and tested on shake tables at Tongji University, China. Concentrating on floating system, the performance of the bridge model was evaluated under increasing levels of seismic excitation in longitudinal direction. Testing results showed that (1) first crack appeared at the bottom section of the tower leg and extended nearly to form a cycle;(2) cracks gradually appeared above the bottom and spread up to half height of the tower at nearly equal distance;(3) final cracks appeared just above the crossing section of tower legs and concrete spalling and exposed longitudinal bars were observed. At the last several stages, the displacement at tower top abruptly increased, and the deck experienced large longitudinal and vertical vibration, and large variation of cable tension force was observed.

*Keywords: Shake Table tests, damage and failure mode, single-tower cable-stayed bridge*

## 1. Introduction

In recent decades, cable-stayed bridges have gained much popularity throughout the world due to its aesthetically appearance, structural efficiency and short construction period. However, characterized by large flexibility and low structural damping [1], cable-stayed bridges are quite sensitive to large amplitude oscillations like earthquakes, and some structural damage was observed during past earthquakes. The first reported damage of cable stayed bridges is Shipshaw cable-stayed bridge during 1988 Saguenay Earthquake [2, 3]. An anchorage plate underneath the deck failed due to stress increments induced by the earthquake and the abutment concrete was damaged due to pounding of deck against abutments. Much more severe damage was observed under 1998 ChiChi Earthquake in Chi-Lu cable stayed bridge [4]. The damage included: 1) flexural hinging of the superstructure and the pylon at the superstructure pylon connection, 2) flexural hinging of the pylon above the pile cap and above the superstructure to pylon connection with concrete spalling of the tower above the roadway, 3) pounding damage at the end span supports, 4) anchorage failure of a cable stay. The seismic damage of the structure and the following loss of life and property have raised the necessity to study seismic fragility of cable stayed bridges under earthquakes.

Endo et al. [5] studied the damage behavior of the steel tower structures of a long span suspension bridge under large earthquakes. Based on pushover analysis in the transverse direction, this paper revealed that the first yielding developed at the first strut, and then shifted to the second and third struts, finally the plastic region extended to the tower leg on compressive side. W X Ren [6] studied the elastic-plastic behavior of steel girder of a cable stayed bridge under different earthquakes and showed that elastic-plastic effects tend to reduce the seismic responses but largely depending on the characteristics of input earthquake records. At the meanwhile, seismic controllable damage to the towers of cable stayed bridges has recently been accepted to dissipate energy under extreme earthquake, as is the case of Rion-Antirion in Greece [7]. H Li [8] conducted damage analysis of the towers for Rigid System, Floating System and Passive Energy Dissipation System, revealing that plastic hinge may form at the bottom of tower.

However, above studies mainly focused on the numerical analysis. Test verification is needed as complementary to excite structures in such a way that they are subjected to conditions representing true earthquake ground motions. X Z Duan [9] firstly conducted a 1/20-scale tower model of a cable-stayed bridge at Tongji University. Test results showed that 1) several cracks appeared and extended at the bottom of tower legs and at the middle of tower legs near the lower strut, 2) The bottom of the tower was seriously damaged with concrete spalling and steel bars fractured and exposed approximately 30 cm of the plastic region. But due to lack of fully-designed cable and deck systems in the model, the applicability of the test results needed to be further verified. This is the purpose of a 1/20 scale full cable-stayed bridge model which was designed and tested on shake tables by R L Wang at Tongji University, Shanghai, China [10]. By exciting the model transversely, the test result shows the damage characteristics of cable-stayed bridges in transverse direction included: 1) the severe damage was observed at the upper strut, with several steel bars fractured at both ends, 2) the repairable damage was observed at tower legs at the bottom and the middle part, with concrete cover spalling and exposure of steel bars, 3) the minimal damage was observed at the lower strut and the both sides of the side bents, with only slightly concrete spalling.

This paper, as a companion paper of R L Wang's, presents another 1:20 full-scale model of a cable-stayed bridge which was excited longitudinally on shake tables also at Tongji University, China. The design, instrumentation, and loading protocol of the test model are described. The test results and observed damage were presented in order to study potential plastic region and possible failure mode of cable-stayed bridges in longitudinal direction.

## 2 Bridge model

### 2.1 Prototype Bridge

Lying in the northeast of Hainan province of China, Puqian sea-crossing bridge spans the Puqian sea area to connect Wenchang City with Haikou City. The main bridge is a symmetric cable-stayed bridge with two 230 spans. Fig. 1 show the schematic elevation of the bridge along with A -shaped RC single tower and deck cross section. As seen in the figure, the tower is 60m wide at the foundation level and 150m high. The strut is at elevation 34m and two skewed legs intersect at elevation 106m. The stay cable arrangement is a two-vertical-plane system and totally 68 cable strands are distributed to support an aerodynamically shaped closed box steel deck which is 37.5 wide and 3.2m high.

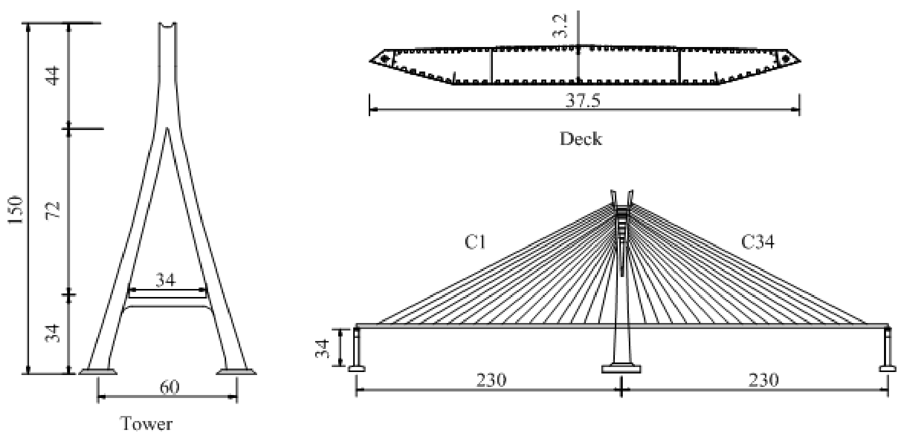



Fig. 1 Schematic of the Puqian Cable-stayed Bridge [m]

### 2.2 Test model

The shake-table bridge model is a 1:20 scale geometric model of a real cable-stayed bridge. With this scale, the total height of the model from the base of the footings to the top of the tower was 7 m, and the total length was 23 m. Three shake tables, all with the dimension of 6*4m and 500mm maximum stroke, were used to support the test model, as shown in Fig. 2. The acceleration scale factor was set to 1 in order to simulate the gravity effect.

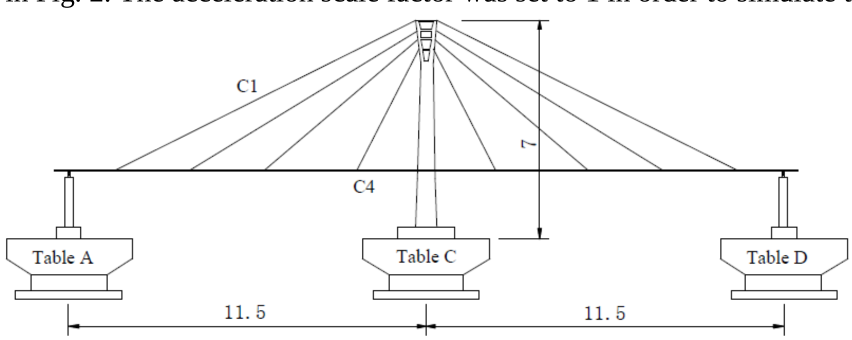



Fig.2. Bridge model and arrangement of shake tables [m]

Buckingham π theorem of dimensional analysis [11] was performed to design the tower and bents .Micro-concrete, the Φ6 steel bars and the galvanized wires were used to substitute the prototype concrete, longitudinal bars and reinforcement stirrups respectively. The elastic modulus of Micro-concrete was properly designed to be 1/3 of the prototype material. Summary of main scale factors adopted can thus be calculated as listed in Table 1.

Table 1 Summary of main scale factors for the model

| **quantity** | length | acceleration | time | modulus | Density | model weight |
|---|---|---|---|---|---|---|
| **scale factor** | 1:20 | 1:1 | 1:√20 | 1:3 | 6:1 | 1:133 |

However, if strictly following geometric scaling factor of 1/20, the minimum thickness of tower plate would be 40mm, making it too difficult to manufacture the model. To solve the construction problem, the approach is to enlarge the thickness of the tower to 80mm while strictly following the scaling factor of flexural stiffness. The same design approach was used to simplify deck construction, which adopts a regular steel box section with the plate thickness of 10mm. To further simplify construction, 34 pairs of cables for the prototype were condensed to 8 pairs for the test model. Based on the principles of equivalent cable forces and the dynamic characteristics from the prototype, the cross sectional area of each cable was calculated 7.85×10-5mm. Teflon sliding bearings are distributed to the tower and bents to support the deck vertically with little friction effect in longitudinal movements. The dimensions of the tower, bents, Teflon sliding bearings and the deck are shown in Fig. 3.

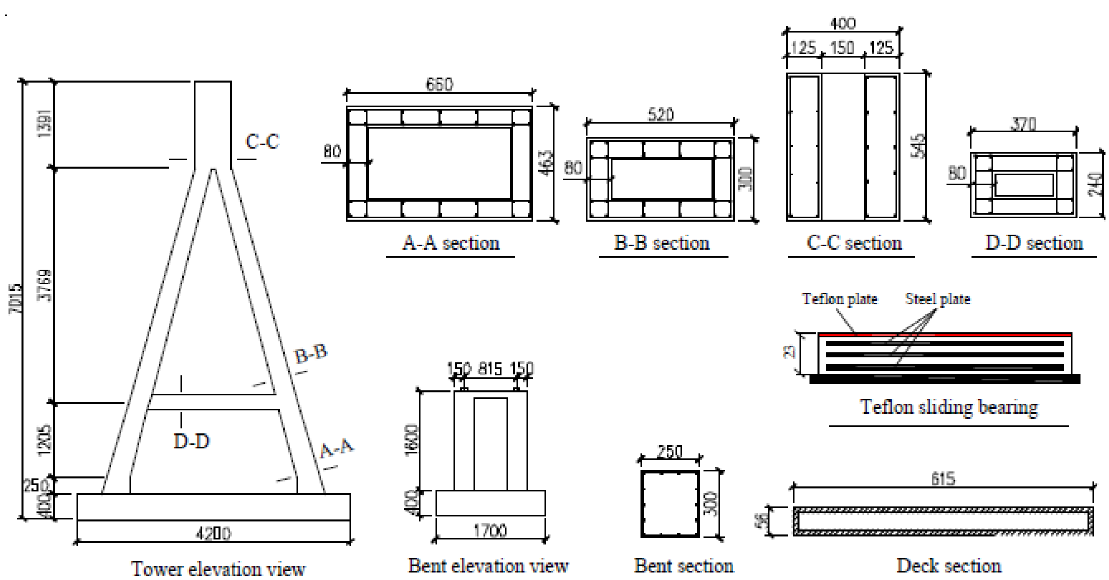



Fig.3. Details of the tower, bents and deck [mm]

From Table 1, it can be seen that the density scale factor is 6:1, so additional mass is needed to increase the density of the structure and to produce a realistic dead load and inertial force. Table 2 lists the self-weight and the additional mass attached to the model.

The assembled bridge model is shown in Fig. 4. As seen in Fig. 4, two gate-style frames are used above transition piers to restrain vertical movement of end span so that end span uplift effect is prevented [4].

Table 2 Self-weight and additional mass of the model

| part | self-weight(kg) | additional mass(kg) | total mass(kg) |
|---|---|---|---|
| tower | 3336 | 15200 | 18536 |
| deck | 2453 | 6216 | 8669 |
| bents | 1440 | 4200 | 5640 |

Fig. 4 Assembled bridge model

## 3 Testing Protocol

The east component of the 1999 Chi-Chi Earthquake measured at TCU052 station is used to simulate the test model in the longitudinal direction. The time axis of the prototype motion was compressed by 0.2236 (1/√20) to account for the scale effect of the test model. Fig. 5 shows the input records with a compressed time axis and scaled amplitude (PGA=0.1g). During the test, the Chi-Chi waves were applied with the increasing PGA from 0.05 to 0.85g in the longitudinal direction. In order to determine changes in structural period so as to evaluate the stiffness degradation of the model, banded white noise excitation was applied at beginning of the test and between each earthquake test after PGA>0.25g.

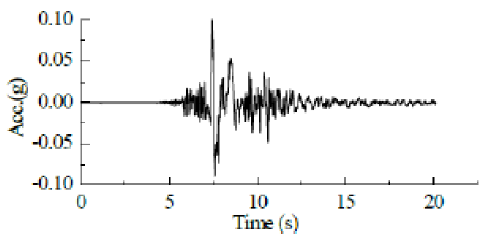
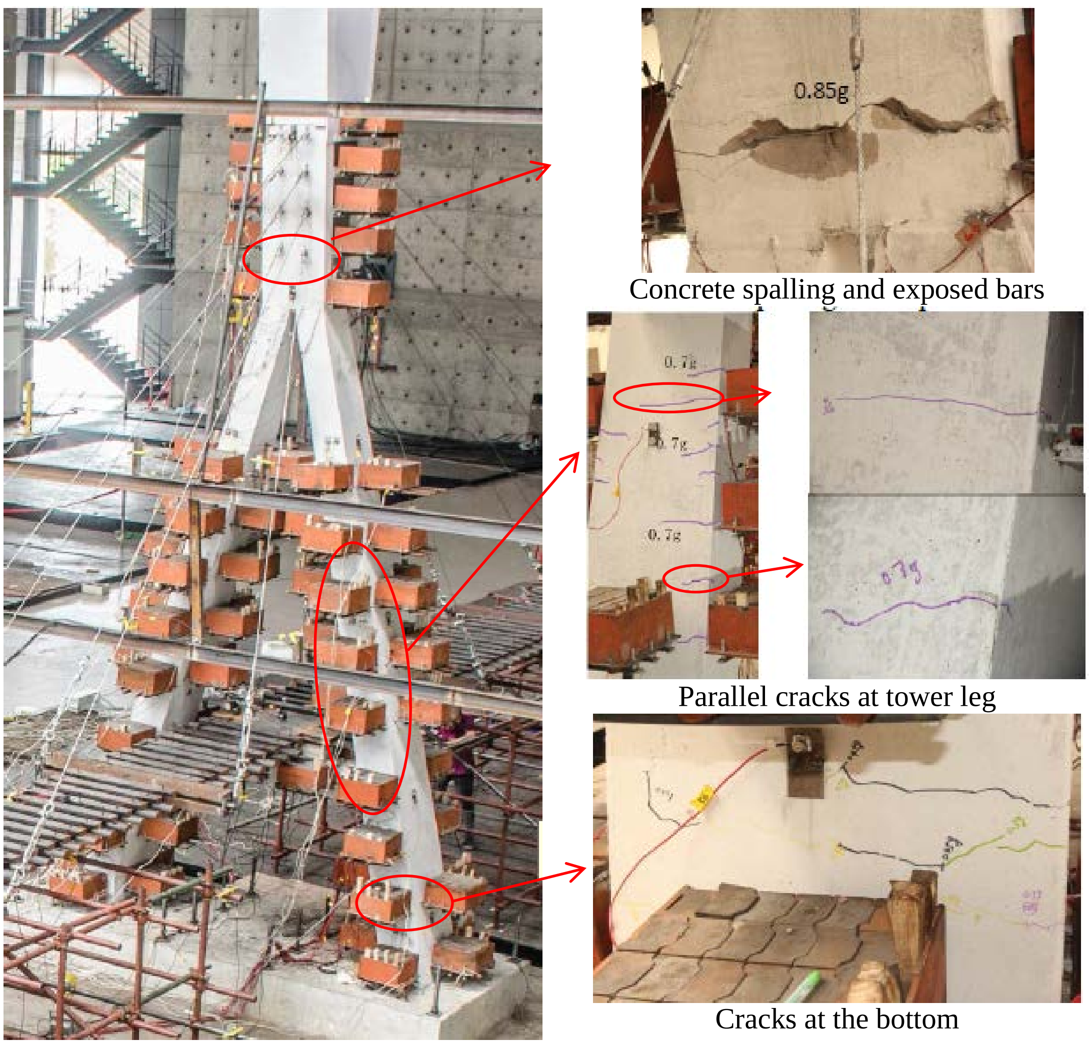



Fig.5. Input motions with compressed time axis and scaled amplitude

# 4 Test results

## 4.1 Observed damage

During the test, the development of cracks at RC tower sections subjected to increasing PGA was observed as follows: 1) PGA=0-0.25g, no damage was observed; 2) PGA=0.3g, first crack appeared at the bottom of tower leg; 3) PGA=0.4~0.5g, the first crack extended to form a circle with several parallel cracks appeared just above the bottom of tower leg; 4) PGA=0.6~0.7g, several horizontal cracks appeared from the bottom to nearly half height of the tower leg and were distributed in parallel at a nearly equal distance of tower width (around 35cm); 5) PGA=0.75g, penetrating cracks appeared at the upper tower just above the section where two skewed legs intersect; 6) PGA=0.8~0.85g, considerable concrete spalling and exposed longitudinal bars were observed at the section just above intersection. Fig. 6 shows the observed seismic damage observed of the bridge model after PGA=0.85g.



Concrete spalling and exposed bars

Parallel cracks at tower leg

Cracks at the bottom

Fig. 6 Observed seismic damage of the bridge model after PGA=0.85g

## 4.2 Modal period

Based on tower top and table acceleration results from white noise excitations, model shapes and periods of the test model were determined utilizing plots of the Fast Fourier Transform. Results revealed that first mode of the test model is longitudinal sliding of the deck and the second mode is longitudinal vibration of the tower coupled with vertical vibration of the deck. Fig. 7 shows the changes of the first and second modal period of the test model after applying Chi-Chi wave with varying PGA. As shown in Fig. 7, the second modal period gradually increased as the damage of the tower occurred and accumulated, while the first modal period slightly increased only when PGA=0.75g. After the test, the period for the first and second longitudinal mode was 3% and 10% longer than that before the test respectively, showing the stiffness degradation of the test model.

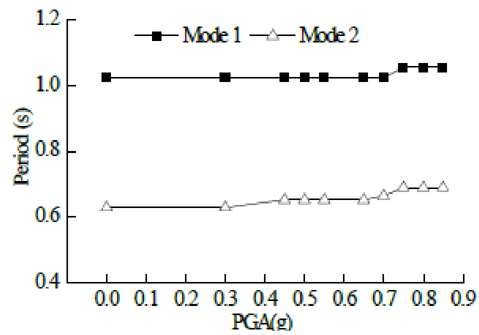
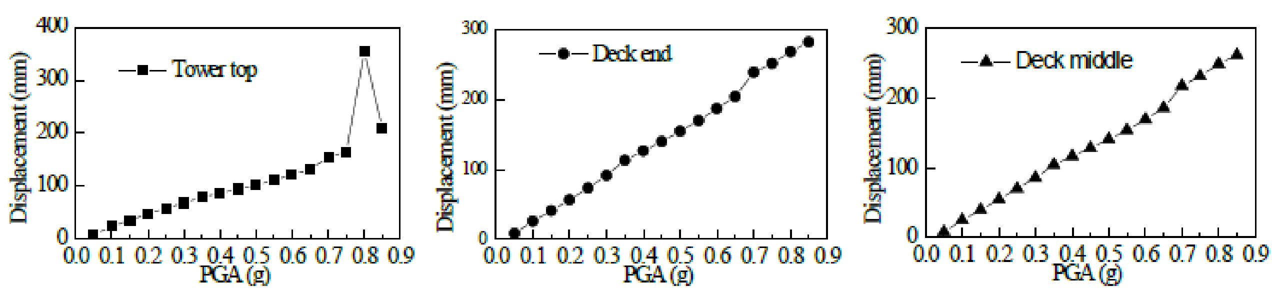



Fig. 7 Period shifts of first and second longitudinal modes

## 4.3 Displacement response

Fig. 8 presents measured maximum longitudinal displacement at tower top, deck end and vertical displacement at deck middle. The displacement at tower top increased almost linearly as PGA increased when PGA≤0.75g. But when concrete spalling appeared at PGA=0.8g, the tower top suddenly went through much larger displacement response, almost twice of that for PGA=0.75g. Both deck longitudinal and vertical displacement linearly increased with increasing PGA, revealing that cracks or concrete spalling of the tower had negligible influence on deck displacement response. And it can also be seen that deck underwent large vertical deformation during the test, almost equal to deck longitudinal response.




(a) Displacement at tower top (b) Displacement at deck end (c) Vertical displacement at deck middle

Fig. 8 Measured maximum displacement of the model

Fig. 9 shows the normalized tower longitudinal displacement at several points. One obtains that 1) when PGA≤0.70g, contribution from each part of the tower to the tower top displacement remained unchanged as PGA increased though cracks appeared at the bottom and middle tower. 2) When PGA=0.75g, the contribution from the upper part of the tower above intersection height increased as penetrating cracks appeared at the upper tower. 3) When PGA=0.8, the contribution from the upper part of the tower above intersection height become 75%, which explains why tower top displacement abruptly increased at this PGA level.

4.4 Cable response

Fig. 10 shows tension force envelope of Cable1~Cable4. When PGA<0.70g, the maximum and minimum tension forces of Cable1~Cable4 changed in a linear manner as PGA increased. However, when PGA≥0.7g, the minimum tension force of Cable3 and Cable4 almost reached 0, and no longer decreased. Under such condition, the minimum tension force of Cable1 and Cable2 slightly increased as PGA increased while maximum tension force of all cables decreased a little. During the test, the maximum tension force of Cable1, Cable3 and Cable4 reached more than twice of the initial cable force, showing large variation of cable tension forces during the test.

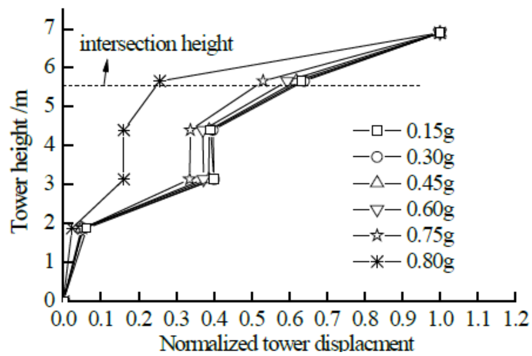
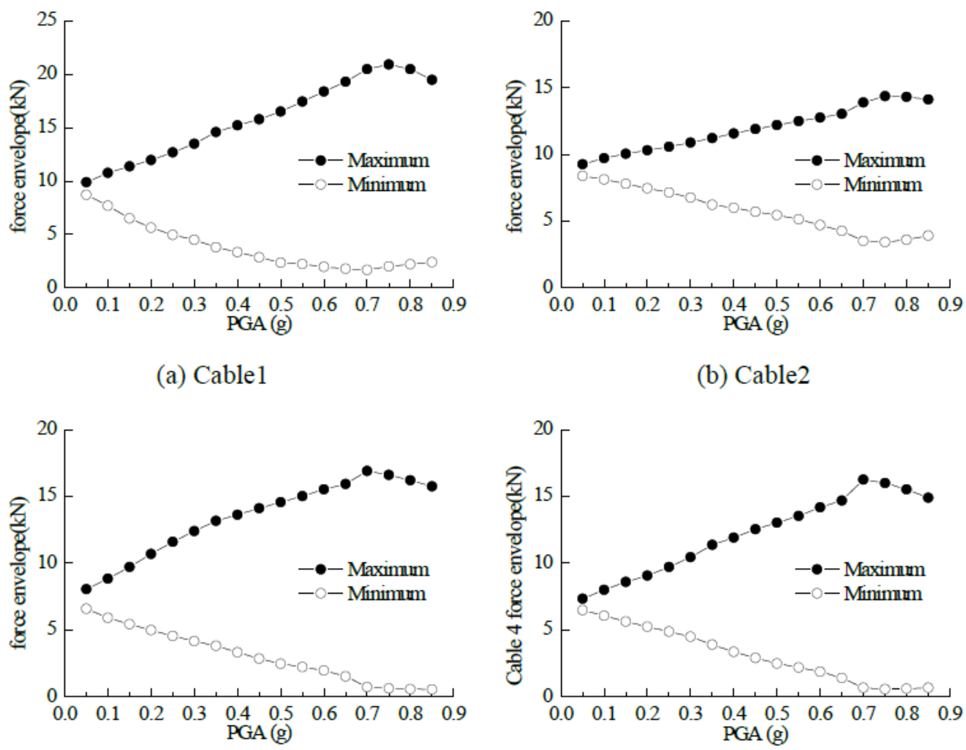



Fig. 9 Normalized tower displacements at some PGA levels




(a) Cable1 (b) Cable2

(c) Cable3 (d) Cable4

Fig. 10 Cable force envelope

## 5 Conclusion

The main objective of this paper was to describe the design and testing of a cable-stayed bridge model by the author, concerning the experimental study of potential plastic region and possible failure mode of cable-stayed bridges in longitudinal direction. From the set of results presented, the following conclusion can be drawn:

(1) The damage of the bridge mainly concentrated on the tower including: 1) several cracks appeared at the bottom region of the tower leg, 2) several horizontal cracks appeared from the bottom to nearly half height of the tower leg and were distributed at a nearly equal distance, 3) concrete spalling and exposed longitudinal bars were observed at the section just above intersection.

(2) The stiffness of the bridge degraded as damage of the tower occurred and accumulated, leading to elongation of vibration period of the model.

(3) Tower experienced large inelastic displacement when concrete spalling occurred and large longitudinal and vertical displacement of the deck were observed.

(4) Cable tension force may experience large variation of tension force, with minimum tension force dropping to zero while maximum tension force might be double of static tension force. However, when minimum tension force became zero, maximum tension force no longer increased.

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